DISCLAIMER

Although the material in this Wisconsin Bridge Manual has been tested by the Bureau of Structures, no warranty, expressed or implied, is made by the Wisconsin Department of Transportation, as to the accuracy of the material in this manual, nor shall the fact of distribution constitute any such warranty, and responsibility is not assumed by Wisconsin Department of Transportation in connection therewith.

1.1 Introduction

The Bridge Manual is for the guidance of design engineers, technicians, and inspection personnel engaged in bridge design, plan preparation, and construction for the Wisconsin Department of Transportation. It is prepared to encourage uniform application of designs and standard details in plan preparation of bridges and other related structures.

This manual is a guide for the layout, design and preparation of highway structure plans. It does not replace, modify, or supersede any provisions of the Wisconsin Standard Specifications, plans or contracts.

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3.1 Specifications and Standards

All bridges in the State of Wisconsin carrying highway traffic are to be designed to the *American Association of State Highway and Transportation Officials (AASHTO) LRFD Design Specifications,* the *American Society for Testing and Materials (ASTM)*, the *American Welding Society (AWS)* and Wisconsin Department of Transportation Standards. The material in this *Bridge Manual* is supplemental to these specifications and takes precedence over them.

All highway bridges are to be constructed according to State of Wisconsin, Department of Transportation, Division of Transportation Systems Development *Standard Specifications for Highway and Structure Construction* and applicable supplemental specifications and special provisions as necessary for the individual project.

All railroad bridges are to be designed to the specifications of the *American Railway Engineering Maintenance-of-Way Association (AREMA) Manual for Railway Engineering* and the specifications of the railroad involved.

3.2 Geometrics and Loading

The structure location is determined by the alignment of the highway or railroad being carried by the bridge and the alignment of the feature being crossed. If the bridge is on a horizontal curve, refer to Figure 3.2-1 to determine the method used for bridge layout. The method of transition from tangent to curve can be found in *AASHTO - A Policy on Geometric Design of Highways and Streets*. Layout structures on the skew when the skew angle exceeds 2 degrees; otherwise detail structures showing a zero skew when possible.

For highway structures, the minimum desirable longitudinal vertical gradient is 0.5 percent. There have been ponding problems on bridges with smaller gradients. This requirement is applied to the bridge in its final condition, without consideration of short term camber effects. Vertical curves with the high point located on the bridge are acceptable provided that sufficient grade each side of the high point is provided to facilitate drainage. Keeping the apex of the curve off of a pier, especially for slab bridges, can be beneficial to reduce ponding at those locations.

The clearances required on highway crossings are given in the Facilities Development Manual (FDM). The recommended clearance for railroad crossings is shown on Chapter 38 Standard for Highway Over Railroad Design Requirements. Proposed railroad clearances are subject to review by the railroad involved.

Highway bridge design live loadings follow the AASHTO LRFD Design specifications using HL93. Chapter 17 provides more detail on applying this load for design. WisDOT requires a specific vehicle design check using the Wis-SPV (Standard Permit Vehicle) which can be found in Chapter 45.

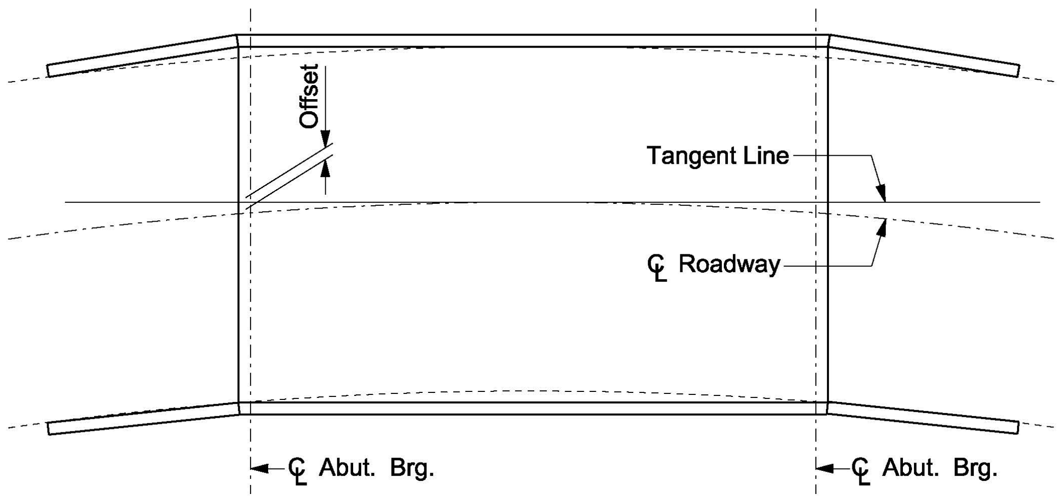
Railroad loadings are specified in the *AREMA Manual for Railway Engineering*.

All new bridges constructed in the State of Wisconsin are designed for the clearances shown in FDM 11-35-1 Attachment 1.8. FDM 11-35-1 Attachment 1.9 covers the cases described in that section as well as bridge widenings. Wires and cables over highways are designed for clearances of 18’-0” to 22’-0”. Vertical clearance is needed for the entire roadway width (critical point to include traveled way, auxiliary lanes, turn lanes and shoulders).

Coordinate early in the design process with the Bureau of Highway Maintenance and Bureau of Structures in determining the appropriate vertical clearance along an OSOW High Clearance Route for new bridges, replacement bridges, bridges with superstructure replacement and overhead utilities. Refer to the FDM 11-10-5.4.3 and 11-35-1.5.1 for additional details along these high routes, including for new and replacement sign structures.

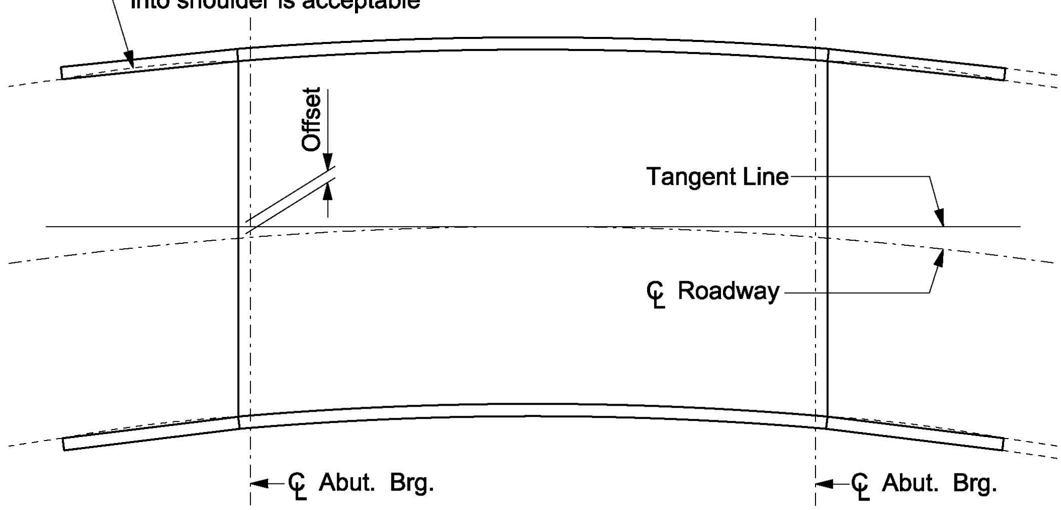
Sidewalks on bridges shall be designed a minimum of 6 feet wide. Refer to the FDM for more details.

The length of bridge approaches should be determined using appropriate design standards. Refer to FDM 3-20-1 for discussion of touchdown points on local program bridge projects.



Case 1
For offsets 0" to 6"

Keep bridge straight. Widen bridge to provide full lane and shoulder width over entire length of bridge (round up to nearest 1"). Align straight wings so inside of wing tip is at edge of shoulder.



Case 2
For offsets over 6"
Curve entire bridge. Do not widen. Align straight wings so inside of wing tip is at edge of shoulder.

Figure 3.2-1
Bridge Layout on Horizontal Curves

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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. See 4.5 for current policy regarding structure aesthetics.

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.

4.2 General Aesthetic Guidelines

Primary features – in relative order of importance:

- Superstructure type and shape, with parapets/railings/fencing being fairly prominent, as well. See Chapter 30 – Railings for further guidance.
- Abutment type and shape, with the wings being most prominent.
- Pier type and shape, with the end elevation being the most notable, especially for a bridge over a highway.
- Grade and/or skews.

Secondary features – in relative order of importance:

- Color
- Pattern and texture
- Ornamentation

Consider the following key points, in relative order of importance, when designing structures:

1. Simplicity
2. Good proportions with an emphasis on thinner members, or members that appear thinner
3. Clear demonstration of how the structure works with recognizable flow of forces
4. Fitting its context/surroundings
5. Good proportions in 3 dimensions
6. Choice of materials
7. Coloring – neutral colors, preferably no more than two. (Chapter 9 – Materials lists *AMS Standard Color Numbers* used most commonly for girders)
8. Pattern and texture
9. Lighting

Consider the bridge shape, relative to the form and function at the location. Use a structural shape that blends with its surroundings. The aesthetic impact is the effect made on the viewer by every aspect of a bridge in its totality and in its individual parts. The designer makes an aesthetic decision as well as a structural decision when sizing a girder or locating a pier.

The structure lines should flow smoothly with as few interruptions as possible. Do not clutter up the structure with distractive elements. If light standards are required, place them in line with the piers and abutments, so the vertical lines blend. Light spacing, however, needs to be coordinated with the Regional electrical engineer. Steel girder bearing stiffeners should be the only vertical stiffeners on the outside face of the exterior girders, although longitudinal stiffeners on the outside face can have an appealing look.

Refer to the WisDOT Traffic Guidelines Manual 2-1-60 for guidance on community sensitive design signing.

4.4 Secondary Features

Color

Color can have a strong visual effect, either positive or negative. Using earth toned colors versus vivid colors is preferred. More neutral colors tend to blend in more with the surroundings. Also, over time earth tones will weather less and not appear as dingy or faded. A bright yellow, for example, will begin to appear dull and dirty soon after application. Avoid red as this color is not UV tolerant and will fade. Concrete stain behaves more like paint and is susceptible to fading and peeling, requiring re-application to avoid an unsightly structure. Stained concrete in need of maintenance looks worse than concrete that was originally left unstained.

Using a maximum of two colors will lend itself to the desired outcome of a clean appearance. On larger structures it may be desirable to use two colors for everything other than the girders, which may be a third color. Remember that plain concrete is a color, too. It should be utilized as much as possible (especially on smaller surfaces) to reduce initial cost and, especially, future maintenance costs.

Utilizing a ribbed, or broken ribbed pattern on a large expanse of plain concrete can give the appearance of color as the patterned section will appear darker than the adjacent plain concrete. This is a good way to add ‘color’ without the future maintenance costs associated with actual stain reapplication.

As much as possible, *AMS Standard Color Numbers* should be used for color selection. A few colors are given in Chapter 9 – Materials, but others may be used. STSP’s should be used as is for staining and multi-colored staining. Specific colors, areas to be applied, etc. should be referenced on the plan sheets.

Pattern and Texture

See 4.5 for current policy regarding structure aesthetics, including patterns and texture.

Large expanses of flat concrete, even if colored, are usually not desirable.

Most bridges are seen from below by people traveling at higher rates of speed. Detail smaller than 4-inches is difficult to discern. The general shape, and perhaps color, will have a greater visual effect than the pattern and/or texture. Sometimes texture is used to represent a building material that wasn’t used for the construction of the structure, as would be the case of rock form liner. While a rock appearance might be appropriate for a smaller bridge over a stream in a small town, it seldom fits the context of a grade separation over a highway or busy urban interchange. Modern bridges should, for the most part, look like they are built out of modern materials appropriate to the current time. Texture consisting of random or ordered geometric forms is generally more preferred over simulating other materials.

On MSE retaining walls it is desirable to keep logos or depictions within a given panel. Matching lines across panels, especially horizontal lines susceptible to differential panel settlement, is difficult. Rock texturing is unconvincing as real stone due to panel joints. A random geometric pattern is a good way to give relief to a wall.

Repetition in pattern rather than an assembly of various patterns or details is more cost effective. For effects that are meant to appear random (e.g. rock), care must be taken in order for the pattern repetition to not appear noticeable.

At all locations on a structure (abutment wings and piers, MSE walls, etc.), form details should be terminated 1'-0" below low water or ground elevations where they will not be visible. See the Standard for Formliner Details.

Designers are cautioned about introducing textures and relief on the inside faces of vehicle barriers. The degree of relief and texture can influence the vehicle response during a crash. See Chapter 30 – Railings for further guidance.

Ornamentation

If signs or medallions are necessary, refer to section 2-1-60 of the *Traffic Guideline Manual*.

Regarding ornamentation in general, more is seldom better.

"In bridge building… to overload a structure or any part thereof with ornaments… would be to suppress or disguise the main members and to exhibit an unbecoming wastefulness. The plain or elaborate character of an entire structure must not be contradicted by any of its parts."

- J.B. Johnson, 1912

4.5 Aesthetics 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. Throughout this process it is important to remember that aesthetics is a concept, not a commodity – it is about a look, not about what can be added to a structure.

WisDOT policy item:

For current statewide policy on aesthetic and/or decorative features (CSS), please see the *Program Management Manual* (PMM). See 4.3 for discussion on primary features such as shape and 4.9 for simple aesthetic concepts. The information below is current WisDOT policy. **Note: Any deviation from the standard details found in the WisDOT Bridge Manual regarding aesthetic features requires prior approval from BOS.**

Aesthetic and/or Decorative Items (non-Participating, or CSS Items)

- *All* formliner is considered CSS. This includes geometric patterns, vertical ribs, rock patterns, custom patterns/designs, etc.
- Stain
- Ornamentation, including city symbols, city names, etc. (City symbols, city names, memorial names, etc. are not allowed on the structures).
- Fencing, railing, or parapets not described below.
- Structure shapes not defined in 4.3 and 4.9 or the standard details.

Note: Future maintenance costs can be substantial when factoring in not only surface preparation and stain/paint, but planning, mobilization and maintenance of traffic required that is entirely attributable to the maintenance project. For example, re-staining of concrete, when all project costs are accounted for, often exceeds $20/SF.

Participating (non-CSS) Items

- Street Names: Street names recessed in the bridge parapet, and stained for visibility, are considered a participating item. The street name is considered an assistance to drivers. Having the name in the parapet removes the sign from the side of the road, which is considered a maintenance problem and safety hazard.

- Protective Fence: Any standard fencing from the Wisconsin Bridge Manual is considered a participating item. Additional costs for decorative fencing requested by the municipality will be included as a non-participating item. Fencing can be either galvanized or a duplex system of galvanized with a colored polymer-coating and/or paint. The polymer coating and/or paint is a nominal cost that provides a longer service life for the fence.

- Bridge Rail: Any standard railing from the Wisconsin Bridge Manual is considered a participating item as long as the railing is required for pedestrian and/or bicyclist protection. There is no discernable difference in cost between any of the standard railings. Paint is a nominal cost that provides longer service life for the railing.

- Bridge Parapet: Any standard parapet from the Wisconsin Bridge Manual is considered a participating item. The Vertical Face Parapet ‘TX’ may be used as a participating item as long as the parapet is required for pedestrian and/or bicyclist protection. There is no discernable difference in cost between the Type ‘TX’ and a shorter, plain concrete parapet with railing that is often used for pedestrian and/or bicyclist protection.

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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.

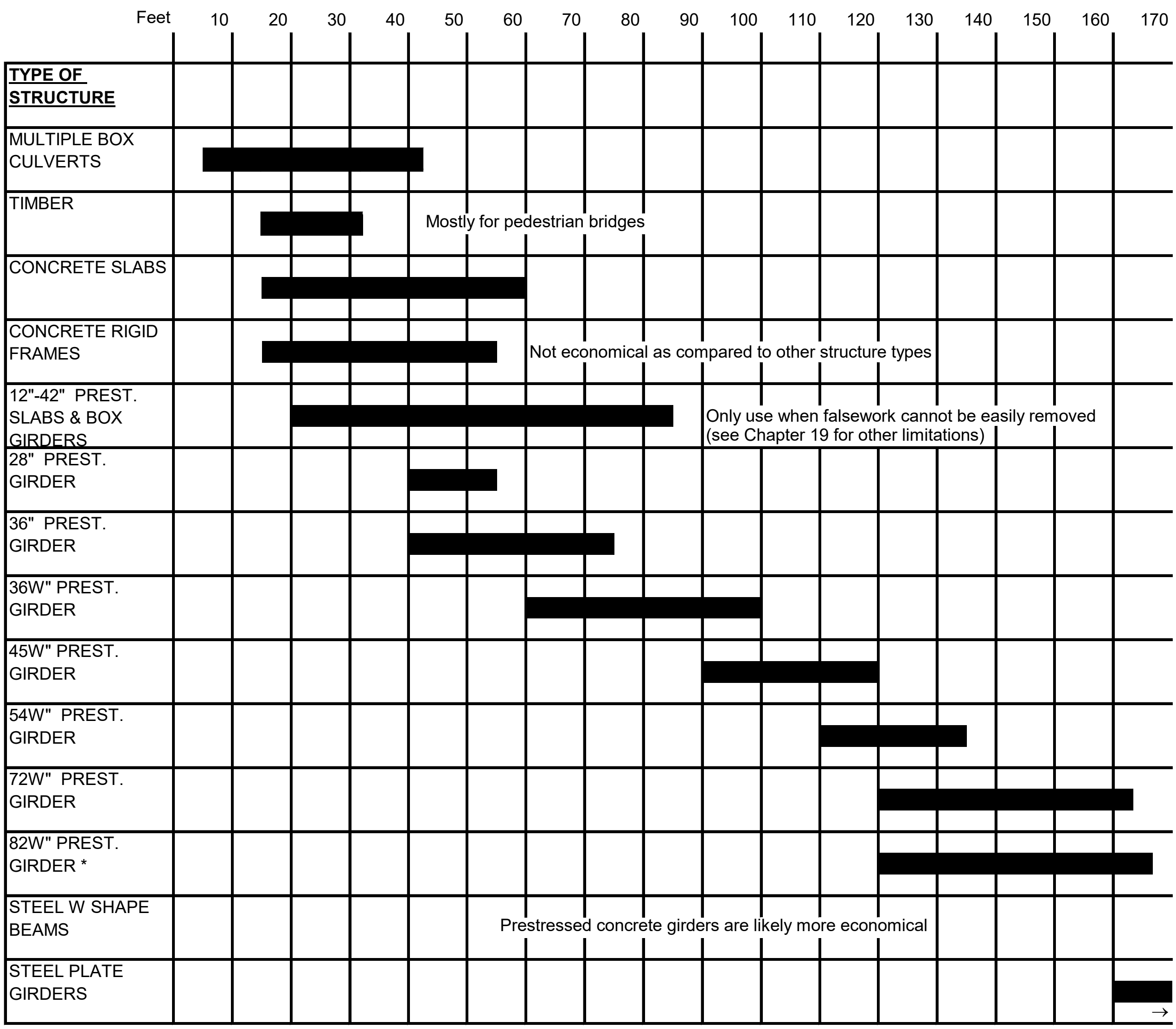
In these reports prestressed girder costs are grouped together because there is a small cost difference between girder sizes. Refer to unit costs. Concrete slab costs are also grouped together for this reason.

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 Facilities Development Manual (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	$206.63
502.3110.S	Expansion Device Modular (structure) (LS)	LF	$1401.52
SPV.0105	Expansion Device Modular LRFD (structure) (LS)	LF	$1947.75

Table 5.3-1
Contract Unit Bid Prices for Structures - 2017

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

5.4 Bid 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.

The square foot costs include all items shown on the structure plan except removing old structure. Costs also include a proportionate share of the project's mobilization, as well as structural approach slab costs, if applicable. However, square footage does not include the structural approach slabs, and is based on the length of the bridge from abutment to abutment. (It is realized that this yields a slightly higher square footage bridge cost for those bridges with structural approach slabs.)

5.4.1 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 5.4-1
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	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 5.4-2
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 5.4-3
Box Culverts

Bridge Type	Cost per Sq. Ft.
Pre-Fab Pedestrian Bridge (B-13-661)	222.06
Pre-Fab Pedestrian Bridge (B-13-666)	240.30
Pre-Fab Pedestrian Bridge (B-17-211)	174.33
Pre-Fab Pedestrian Bridge (B-40-784)	289.02
Concrete Slab Pedestrian Bridge (B-13-656)	105.60
Concrete Slab Pedestrian Bridge (B-13-657)	106.62
Buried Slab Bridge (B-24-40)	182.28
Buried Slab Bridge (B-5-403)	165.57
Buried Slab Bridge (B-13-654)	210.68
Railroad Bridge (B-40-773)	1,151.00
Railroad Bridge (B-40-774)	1,541.00
Inverted T Bridge (B-13-608)	192.75
Inverted T Bridge (B-13-609)	235.01
Inverted T Bridge (B-40-89)	528.81

Table 5.4-4
Miscellaneous 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
Wire Faced MSE Walls	28	160,296	20,554,507	128.17

Table 5.4-5
Retaining Walls

5.4.2 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 5.4-6
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	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-Beams	1	2,078	596,712	82.99	287.16
Trapezoidal Steel Box Girders	1	59,128	9,007,289	121.00	152.34
Pedestrian Bridges	3	35,591	7,436,429	--	208.94

Table 5.4-7
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 5.4-8
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
Wire 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 5.4-9
Retaining Walls

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

Table 5.4-10
Noise Walls

5.4.3 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 5.4-11
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 5.4-12
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
Double Pipe	2	426.20
Triple Pipe	2	1,424.09
Quadruple Pipe	1	2,332.96

Table 5.4-13
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
MSE Panel Walls w/Integral Barrier	4	14,330	1,098,649	76.67
Concrete Walls	2	6,850	712,085	103.96
Wire Faced MSE Walls	3	10,345	1,501,948	145.19
Wire 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

Table 5.4-14
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 5.4-15
Sign Structures

5.4.4 2016 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	19	199,367	26,157,660	57.97	131.20
Reinf. Conc. Slabs (Flat)	36	72,066	10,985,072	63.40	152.43
Reinf. Conc. Slabs (Haunched)	5	22,144	2,469,770	50.63	111.53
Prestressed Box Girders	3	4,550	773,098	80.85	169.91

Table 5.4-16
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	25	343,165	40,412,805	60.62	117.76
Reinf. Conc. Slabs (Haunched)	5	33,268	4,609,286	59.21	138.55
Steel Plate Girders	3	127,080	18,691,714	90.78	147.09
Pedestrian Bridges	1	4,049	846,735	91.35	209.13

Table 5.4-17
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	18	1,694.52
Twin Cell	10	2,850.45
Single Pipe	1	1,268.42

Table 5.4-18
Box Culverts

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	10	10,310	558,347	54.16
MSE Panel Walls	21	112,015	8,681,269	77.50
Modular Walls	5	6,578	419,334	63.75
Soldier Pile Walls	2	13,970	1,208,100	86.48
Steel Sheet Pile Walls	1	3,440	104,814	30.47

Table 5.4-19
Retaining Walls

Sign Structure Type		No. of Structures	Total Lineal Ft. of Arm	Total Costs	Cost per Lin. Ft.
Butterfly (2-Signs)	Conc. Col.	1	25.25	89,102	3,528.80
	1-Steel Col.	1	24.34	44,176	1,814.97
Cantilever	Conc. Col	5	171	384,487	2,248.46
	1-Steel Col.	18	536.25	758,646	1,414.72
Full Span	Conc. Col.	0	--	--	--
	1-Steel Col.	7	430.25	400,125	929.98
	2-Steel Col.	7	590	611,292	1,036.23

Table 5.4-20
Sign Structures

5.4.5 2017 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	24	238,956	33,970,344.86	60.05	142.16
Reinf. Conc. Slabs (Flat)	44	69,095	11,063,299.53	57.75	160.12
Reinf. Conc. Slabs (Haunched)	8	48,434	6,759,897.64	55.41	139.57
Prestressed Box Girders	2	2,530	691,474.35	117.93	273.31

Table 5.4-21
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	28	302,672	37,247,580.94	52.67	123.10
Reinf. Conc. Slabs (Flat)	25	58,076	9,561,823.06	42.14	164.64
Reinf. Conc. Slabs (Haunched)	6	49,160	9,444,012.75	43.73	192.11
Steel Plate Girders	0	--	--	--	--
Pedestrian Bridges	2	12,864	2,141,133.01	53.53	166.44

Table 5.4-22
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	18	1,849.26
Twin Cell	3	3,333.61
Single Pipe	1	1,752.93
Precast	1	2,204.32
Precast Three-Sided	3	8,754.76

Table 5.4-23
Box Culverts

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
CIP Cantilever	17	30,808	3,277,766.33	106.39
CIP Facing (MSE)	3	10,611	1,683,447.67	158.65
MSE Block Walls	6	13,378	1,457,896.15	108.98
MSE Panel Walls	21	137,718	11,789,074.54	85.60
Modular Walls	3	3,643	254,004.30	69.72
Precast Panel and Wire Faced	3	17,270	2,294,507.57	132.86
Soldier Pile Walls	0	--	--	--
Steel Sheet Pile Walls	5	15,056	1,442,741.15	95.82

Table 5.4-24
Retaining Walls

Sign Structure Type		No. of Structures	Total Lineal Ft. of Arm	Total Costs	Cost per Lin. Ft.
Butterfly (1-Sign)	Conc. Col.	0	--	--	--
	1-Steel Col.	4	84.5	221,728.47	2,623.01
Butterfly (2-Signs)	Conc. Col.	0	--	--	--
	1-Steel Col.	6	217.22	417,307.35	1,921.13
Cantilever	Conc. Col	0	--	--	--
Cantilever Full Span	1-Steel Col.	28	825.75	1,165,570.03	1,411.53
	2-Steel Col.	2	199	245,997.03	1,236.17
	Conc. Col.	2	185	349,166.59	1887.39
Full Span	1-Steel Col.	6	466.03	589,773.11	1265.53
	2-Steel Col.	21	1,773.5	1,789,041.14	1008.76

Table 5.4-25
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 Facilities Development Manual (FDM) Section 20-50-5 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.

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.

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:

   <u>Material Properties</u>:

- 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 (45.9.3) 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)

If widening a bridge, provide ratings for both the new and existing superstructure elements. For example, if widening a girder bridge previously designed with Load

Factor Design, provide the LFR rating for the controlling existing girder and the LRFR rating for the controlling new girder.

Hydraulic Data

100 YEAR FREQUENCY

Q_{100} = XXXX C.F.S.
VEL. = X.X F.P.S.
HW_{100} = EL. XXX.XX
WATERWAY AREA = XXX SQ.FT.
DRAINAGE AREA = XX.X SQ.MI.
ROADWAY OVERTOPPING = (NA or add "Roadway Overtopping Frequency" data)
SCOUR CRITICAL CODE = X

2 YEAR FREQUENCY

Q_2 = XXXX C.F.S.
VEL. = X.X F.P.S.
HW_2 = EL. XXX.XX

ROAD OVERTOPPING FREQUENCY (if applicable, frequencies < 100 years)

FREQUENCY = XX YEARS
Q_{XX} = XXXX C.F.S.
HW_{XX}= EL. XXX.XX

(See Chapter 8 – Hydraulics for additional information)

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

6. Rehabilitation structure plans should use the same labeling convention as shown on the original structure plans when practical. Generally, this will include substructure labels (wings, abutments, piers, etc.) and girder numbers. This labeling convention is beneficial for inspection purposes.

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 FDM Chapter 18 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 Other Agencies

This is a list of other agencies that may or may not need to be coordinated with. There may be other stakeholders that require coordination. Consult FDM Chapter 5 for more details on coordination requirements.

- Department of Natural Resources

  A copy of preliminary plans (preliminary layout, plan & profile, and contour map) for stream crossing bridges are forwarded by BOS 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).

- Railroad (FDM Chapter 17)

  Begin communicating as early as possible with the Region Railroad Coordinator.

- Utilities (FDM Chapter 18, Bridge Manual Chapter 32)

  BOS discourages attachment of utilities to a structure. However, if there are no other viable options, private or public utilities desiring to attach their facilities (water, and sewer mains, ducts, cables, etc.) to the structure must apply to the owner for approval. For WisDOT owned structures, approval is required from the Region's Utilities & Access Management Unit.

- Coast Guard (FDM)

- Regions

  A copy of the preliminary plans is sent to the Regional Office involved for their review and use.

- Native American Tribal Governments

- Corps of Engineers

- Other governing municipalities

- State Historic Preservation Office

- Environmental Protection Agency

- Other DOTs

East. See FDM 9-25-5 for additional bench mark information. For multi-directional bridges, locate the name plate on the roadway side of the first right wing or parapet traveling in the highway cardinal directions of North or East. For one-directional bridges, locate the name plate on the first right wing or parapet in the direction of travel. For type "NY", "W", "M" or timber railings, name plate to be located on wing. For parapets, name plate to be located on inside face of parapet.

6.3.3.8 Removing Old Structure and Debris Containment

This section provides guidance for selecting the appropriate Removing Old Structure bid item and determining when to use the "Debris Containment" bid item.

The "Removing Old Structure" bid item is most typically used for complete or substantial removals, as described in 6.3.3.8.2, of grade separation structures. In addition to this Standard Specification bid item, there are three STSP bid items for complete or substantial removal work over waterways: "Removing Old Structure Over Waterway", "Removing Old Structure Over Waterway With Minimal Debris", and "Removing Old Structure Over Waterway With Debris Capture System". The designer should review all of these STSPs and coordinate with the Wisconsin Department of Natural Recourses (DNR) to reach consensus on which STSP to use when removing a particular structure. **The designer should not automatically defer to the recommendation from the initial DNR letter, but should work with WisDOT and DNR environmental coordinators, considering constructability and cost impacts of the items.** For unique or difficult removals, designers should consult with the contracting community to assess costs and the feasibility of a particular removal technique. One of the following Removing Old Structure bid items should be selected for removals over waterways:

- Removing Old Structure Over Waterway is used where it is not possible to remove the structure without dropping it, or a portion of it, into a waterway or wetland; and that waterway or wetland is not highly environmentally sensitive. This special provision is typically appropriate for removing the following structure types: slab spans; voided slabs; cast-in-place concrete girder bridges; earth-filled bridges.

- Removing Old Structure Over Waterway <u>With Minimal Debris</u> is used where it is possible to remove the structure without dropping it, or a portion of it, into a waterway or wetland, and that waterway or wetland is not highly environmentally sensitive. This special provision is typically appropriate for removing all structures types except for the following bridges which are typically covered under Removing Old Structure Over Waterway: slab spans; voided slabs; cast-in-place concrete girder bridges; earth-filled bridges.

- Removing Old Structure Over Waterway <u>With Debris Capture System</u> is typically used when the waterway or wetland is highly environmentally sensitive. Before including this special provision in the contract, consult with the department's regional environmental coordinator to determine if the affected waterway or wetland is highly environmentally sensitive and if this special provision is appropriate.

Debris Containment is used where structure removal, reconstruction, or other construction operations may generate falling debris that might pose a safety hazard or environmental/contamination concern to facilities located under the structure. This item is most typically used where the removal area is located over a railroad.

The Debris Containment item is not used when one of the Removing Old Structure Over Waterway items is used.

6.3.3.8.1 Structure Repairs

Structure repair work could include, but is not limited to, the following bid items:

- Removing Concrete Masonry Deck Overlay
- Removing Asphaltic Concrete Deck Overlay
- Removing Polymer Overlay
- Cleaning Parapets
- Cleaning Concrete Surfaces
- Cleaning Decks to Reapply Concrete Masonry Overlay
- Preparation Decks (type)
- Cleaning Decks
- Joint Repair
- Curb Repair
- Concrete Surface Repair
- Full-Depth Deck Repair

Removal work limited to the above items is already included in the respective bid item specification, therefore a Removing Old Structure bid item not required. Use of Debris Containment should be reviewed for the following conditions:

- For work **over waterways**, a method of protecting the waterway is needed in some cases. Use Debris Containment, **only as needed** based on the extent and location of removal, and environmental sensitivity of the waterway. Debris is expected to be minimal.
- For work **over roadways**, Standard Specification, Sections 104 and 107, addresses safety of the traveling public and damage to all property, so generally no additional specifications are needed. **It is expected that pertinent lanes of the underpass roadway are closed when falling debris is expected from above.** No additional specifications are needed unless specifically requested with sufficient reason, in which

case use Debris Containment **only as needed**, based on the extent and location of removal. Debris is expected to be minimal.

- For work **over railroads**, Standard Specification, Sections 104 and 107, addresses safety of the traveling public and damage to all property, so generally no additional specifications are needed. Exception: containment of debris is required where Full-Depth Deck Repair is expected. Use Debris Containment if Full-Depth Deck Repair is expected, or **only as needed**, based on the extent and location of removal. Debris is expected to be minimal.

6.3.3.8.2 Complete or Substantial Removals

Complete or substantial removals, not covered by one of the bid items listed in 6.3.3.8.1, should use a Removing Old Structure bid item. Substantial removals could include, but are not limited to; decks, parapets, and wingwalls. The appropriate Removing Old Structure bid item should be selected and the need for Debris Containment should be reviewed for the following conditions:

- For work **over waterways**, a method of protecting the waterway is needed if the removal area is located over the waterway. If the removal area is located over the waterway, use Removing Old Structure Over Waterway, Removing Old Structure Over Waterway With Minimal Debris or Removing Old Structure Over Waterway With Debris Capture System. If the removal area is not located over the waterway, use Removing Old Structure. The Debris Containment item is not used for this work.

- For work **over roadways**, Standard Specification, Sections 104 and 107, addresses safety of the traveling public and damage to all property, and Standard Specification, Section 203 Removing Old Culverts and Bridges addresses removal. **It is expected that pertinent lanes of the underpass roadway are closed when falling debris is expected from above.** Use Removing Old Structure. No additional specifications are needed unless specifically requested with sufficient reasoning. Use Debris Containment **only as needed**, based on the significance of the roadway and/or location of removal.

- For work **over railroads**, Standard Specification, Sections 104 and 107, addresses safety of the traveling public and damage to all property, and Standard Specification, Section 203 Removing Old Culverts and Bridges addresses removal. A method of protecting the railroad is needed if the removal area is located over the railroad. Use Removing Old Structure. Use Debris Containment if the **removal area is located over the railroad, or only as needed**, based on the extent and location of removal.

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 Granular Materials

Granular materials can be bid in units of tons or cubic yards. Structure plans should use the TON bid item for Structure Backfill, Granular Backfill, and Base Aggregate Dense 1 1/4-inch, unless directed otherwise by the Region. The Region may consider use of the CY bid item when contractor-provided tickets may be problematic or when the TON bid item is not used elsewhere on the contract. Other cases may also warrant the use of the CY bid item.

For Structure Backfill, Granular Backfill, and Base Aggregate Dense 1 1/4-inch materials use a 2.0 conversion factor (tons/cubic yard) for compacted TON bid items or use a 1.20 expansion factor (i.e. add an additional 20%) for CY bid items, unless directed otherwise. Refer to the FDM when preparing computations using other granular materials (breaker run, riprap, etc.).

Granular quantities and units should be coordinated with the roadway designer. For some structures, backfill quantities may be negligible to the roadway, while others may encompass a large portion of the roadway cross section and be present in multiple cross sections. A long

MSE retaining wall would be an example of the latter case and will require coordination with the roadway designer.

Generally, granular material pay limits should be shown on all structure plans. This information should be used to generate the estimated quantities and used to coordinate with roadway cross sections and construction details. See Standard Detail 9.01 – Structure Backfill Limits and Notes - for typical pay limits and plan notes.

Refer to 9.10 for additional information about granular materials.

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 1 cubic yard.

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. Preparation Decks Type 1 should be provided by the Region. Estimate Preparation Decks Type 2 as 40% of Preparation Decks Type 1. Deck preparation areas shall be filled using Concrete Masonry Overlay Decks, Concrete Masonry Deck Repair, 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 and Debris Containment

For work over roadways and railroads, "Removing Old Structure" is most typically used for complete or substantial removals. For work over waterways, one of the following bid items should be used for complete or substantial removals: Removing Old Structure Over Waterway, Removing Old Structure Over Waterway With Minimal Debris, or Removing Old Structure Over Waterway With Debris Capture System. For work other than complete or substantial removals, a Removing Old Structure bid item may not be required.

Use Debris Containment, **only as needed** based on the significance, extent, or location of the removal.

See 6.3.3.8 for additional information on Removing Old Structure and Debris Containment bid items.

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 Concrete Adhesive Anchors

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. This item is seldom used now that railroad excavations have a unique SPV.

Record this quantity to the nearest square foot for the area from the sheet pile tip elevation to one foot above the retained grade.

6.4.37 Temporary Shoring

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

Measured as square foot from the ground line in front of the shoring to a maximum of one foot above the retained grade. For the estimated quantity use the retained area (from the ground line in front of the shoring to the ground line behind the shoring, neglecting the additional height allowed for measurement).

6.4.38 Concrete Masonry Deck Repair

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 Type 1.

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 19-5-3.2 for bid item usage guidance and quantity calculation guidance. Bid as LB and round to the nearest 5 lbs.

6.4.42 Asphaltic Overlays

Estimate the overlay quantity by using the theoretical average overlay thickness and add ½" for variations in the deck surface. Provide this average thickness on the plan, as well. Use 110 lbs/(square yard - inch) to calculate hot mix asphalt (HMA) and polymer modified asphalt (PMA) overlay quantities.

For HMA overlays use 0.07 gallons/square yard to calculate tack coat quantity, unless directed otherwise.

Coordinate asphaltic quantity assumptions with the Region and roadway designers.

6.5 Production of Structure 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 (including maintenance projects), a completed Structure Survey Reports, preliminary and final plans are submitted to the Bureau of Structures with a copy forwarded to the Regional Office for review and 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 loads on 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.

The list of consultant firms eligible to provide structural design services to WisDOT may be accessed using the link below:

https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/plan-submittal.aspx

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 Subsurface Investigation Report.
Consultant	Submit hydrology report via Esubmit or as an email attachment to the supervisor of the Consultant Review and Hydraulics Unit. Submit 60 days prior to preliminary plan submittal.
	Prepare preliminary plans according to 6.2.
	Coordinate with Region and other agencies per 6.2.3.
	Submit preliminary plans, SSR and supporting documents via e-submit for review and approval of type, size and location.
Structures Design Section	Record project information in HSIS.
	Review hydraulics for Stream Crossings.
	Review Preliminary Plan. A minimum of 30 days to review preliminary plans should be expected.
	Coordinate with other agencies per 6.2.3.
	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, and provide explanation for preliminary comments not incorporated in final plan.
	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 required final design documents via e-submit per 6.5.3.
Structures Design Section	Determine which final plans will be reviewed and perform quality assurance review as applicable.
	For final plans that are reviewed, return comments to Consultant and send copy to Regional Office, including FHWA as appropriate.
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 and send performance evaluation form to Region and Consultant.
Geotechnical Consultant	At time of PSE, transmit gINT boring logs, soils laboratory testing summary and data sheets, and Soil Reports to the emails provided in the Soils and Subsurface Investigations section of Two/Three Party Design Contract Special Provisions.
Bureau of Project Development	Prepare final accepted structure plans for pre-development contract administration.
Consultant	If a plan change is needed after being advertised but before being let, an addendum is required per FDM 19-22-1 and 19-22 Attachment 1.2.
Structures Design Section	Review structure addendum as applicable.
	Sign structure addendum.
Bureau of Project Development	Distributes structure addendum to bidders.
Consultant	If a plan change is required after being let, a post-let revision is required per 6.5.5.
Structures Design Section	Review post-let revision as applicable.
	Stamp post-let revision plan as accepted.
	Delivers revised plan to DOT construction team for distribution.

Table 6.5-1
Approvals, Distribution and Work Flow

6.5.2 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. Hydrology Report
2. Structure Survey Report
3. Preliminary plan, including log borings shown on the subsurface exploration sheet
4. Evaluation of subsurface investigation report
5. Contour map
6. Plan and profile, and typical section for roadway approaches

7. Hydraulic/Sizing Report (see Chapter 8 - Hydraulics) and hydraulic files are required for stream crossing structures

8. County map showing location of new and/or existing structures and FEMA map

9. Any other information or drawings which may influence location, layout or design of structure, including DNR initial review letter and photographs

6.5.3 Final Plan Requirements

The guidelines and requirements for Final Plan preparation are given in 6.3. The Load Rating Summary form and On-Time Submittal form can be found on the Bureau of Structures' Design and Construction webpage. The following files are included as part of the final-plan submittal:

1. Final Drawings

2. Design and Quantity Computations

   For all structures for which a finite element model was developed, include the model computer input file(s).

3. Special Provisions covering unique items not in the Standard Specifications or Standardized Special Provisions (STSP).

4. QA/QC Verification Sheet

5. Inventory Data Sheet

6. Bridge Load Rating Summary Form

7. LRFD Input File (Excel ratings spreadsheet)

8. On-Time Improvement Form

   The On-Time Improvement form is required to be submitted if either of the following situations occur:

   - If the first version of a final structure plan is submitted after the deadline of two months prior to the PSE date.

   - If any version of a final structure plan is re-submitted after the deadline of two months prior to the PSE date. However this form is not required when the re-submit is prompted by comments from the Consultant Review Unit. The form is also not necessary when submitting addenda or post-let revisions.

6.5.4 Addenda

Addenda are plan and special provision changes that occur after the bid package has been advertised to potential bidders. See FDM 19-22-1 for instruction on the addenda process.

6.5.5 Post-Let Revisions

Post-let revisions are changes to plan details after the contract is awarded to a bidder. ESubmit only the changed plan sheets, not the entire plan set. The changes to the plan sheet shall be in red font, and outlined by red clouding. The revision box shall also be filled in with red font. Each sheet shall be 11x17, PE stamped, signed, and dated on the date of submittal.

6.5.6 Local-Let Projects

Local-let projects that are receiving State or Federal funding shall be submitted to and reviewed by the Consultant Review Unit in the same way as a State-let project. Final structure plans accepted and signed by the Consultant Review Unit will be returned to the Designer of Record and to the Region for incorporation into the local contract package.

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:

https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/default.aspx

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

https://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/

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duration. Traditionally, the Design-Bid-Build (DBB) method has been used for project delivery. This involves the design and construction to be completed by two different entities. Project schedules using the DBB method are elongated because the design and construction cannot be completed concurrently. The entire design process must be completed before the bidding process begins. Finally, after the bidding process is completed, the construction can begin.

Other state DOT's have used project delivery methods that can allow for more accelerated overall project delivery. These include Design/Build (D/B) and Construction Manager/General Contractor (CM/GC). The D/B process requires the designer-builder to assume responsibility for both the design and construction of the project. This method increases the risk for the design-builder, and reduces the risk for the owner. Project delivery time can be reduced, since the D/B process allows for the design and construction phases to overlap, unlike the DBB process. There is a specific type of D/B called Low Bid Design Build (LBDB) which has the same structure as the traditional D/B process, except that the lowest bidder wins the project (rather than having a quality component as with the traditional D/B process). Refer to the Facilities Development Manual (FDM) for further discussion on LBDB.

The CM/GC process is a hybrid of the DBB and D/B processes. In CM/GC, both the designer and the contractor have contracts with the owner, and the owner is part of the design team. In this process, a construction manager is selected, and is able to provide input regarding schedule, pricing, and phasing during the design phase. Around the 60% or 90% design completion, the owner and construction manager negotiate a "guaranteed maximum price" for the construction of the project based on the defined scope and schedule. CM/GC allows the owner to remain active in the design process, while the risk is still taken by the general contractor.

Generally, in Wisconsin, projects administered by the Department have been Design Bid Build with minimal use of the Low Bid Design Build method. Refer to the FDM 11-2 for additional discussion on Alternative Contracting (AC) methods.

WisDOT policy item:

Each state has different preferences and constraints to which project delivery method they use, and due to current legislation, CM/GC and traditional D/B are not viable options for the state of Wisconsin. To implement ABC using the DBB process, the contract should either specify to use the ABC method required by the owner, and/or provide opportunity for the contractor to propose ABC alternatives that meet contract requirements.

7.2 ABC Decision-Making Guidance

This section is intended to provide guidance on when to use ABC versus conventional construction. When ABC methods are appropriate, this section will also help determine which ABC method(s) are most practical for a particular project.

Figure 7.2-1 is a Decision Matrix that can be used to determine how applicable an ABC method is for a particular project. Each item in Figure 7.2-1 is described further in Table 7.2-1. Once a total score is obtained from the Decision Matrix, the score is used to enter the Decision Flowchart (Figure 7.2-2). After entering the Flowchart, the user could be directed to the question "Do the benefits of ABC outweigh any additional costs?" This question needs to be evaluated on a project-specific basis, using available project information and engineering judgment. This item is intended to force the user to step back, think about the project as a whole, and decide if an ABC method really makes sense with all the project-specific information considered. The remainder of the flow chart questions will help guide the user toward the ABC method(s) that are most appropriate for the project.

There is an acknowledged level of subjectivity in both the Decision Matrix and in the Flowchart. These tools are intended to provide general guidance, not to provide a specific answer for all projects. The tools present different types of considerations that should be taken into account to help guide the user in the right direction and are not intended to provide a "black and white" answer.

The flowchart item "Program Initiative" can encompass a variety of initiatives, including (but not limited to) research needs, public input, local initiatives, stakeholder requests, or structure showcases. These items should be considered on a project-specific basis.

The flowchart guides users towards specific ABC technologies. However, the user should also recognize the ability and opportunity to combine various ABC technologies. For example, the combination of PBES with GRS-IBS could be utilized.

For additional guidance or questions, contact the Bureau of Structures Development Section Chief.

ADT and/or ADTT (Construction Year)	This is a measure of the total amount of traffic crossing the bridge site. A higher ADT value at a site will help support the use of accelerated bridge construction methods. Use a construction year ADT value equal to the sum of the traffic on the structure and under the structure. For cases where there is a very high ADT on the bridge and very low or no ADT under the bridge, consider using a "slide" method (on rollers or Polytetrafluorethylene (PTFE)/Elastomeric pads) or SPMT's, which can be very cost effective ABC techniques for this situation. For structures with a higher-than-average percentage of truck traffic, consider providing a higher score than indicated solely by the ADT values in the table.
Required Lane Closures/Detours?	This is a measure of the delay time imposed on the traveling public. If conventional construction methods will provide significant delays to the traveling public, provide a high score here. If conventional construction methods will provide minimal delays to the traveling public, provide a low score here. Use the delay times provided in the table as guidance for scoring.
Are only Short Term Closures Allowable?	This is a measure of what other alternatives are available besides accelerated bridge construction. If staged construction is not an alternative at a particular site, the only alternative may be to completely shut down the bridge for an SPMT move, and therefore a high score should be provided here. If there is a good alternative available for staged construction that works at the site, a low score should be provided here.
Impact to Economy	This is a measure of the impact to the local businesses around the project location. Consider how the construction staging, road closures, etc. will impact local businesses (public access, employee access, etc.) A high impact to the economy equates to a high score here. A low impact to the economy equates to a low score here.
Impacts Critical Path of Total Project?	This is a measure of how the construction schedule of the structure impacts the construction schedule of the entire project. If the construction of the structure impacts the critical path of the entire project, and utilizing ABC methods provides shorter overall project duration, provide a high score here. If other project factors are more critical for the overall project schedule and utilizing ABC methods will not affect the overall project duration, provide a low score here.
Restricted Construction Time	This is a measure of how the construction schedule is impacted by environmental and community concerns or requirements. Items to consider are local business access windows, holiday schedules and traffic, special event traffic, etc. If there are significant restrictions on construction schedule, provide a high score here. If there are little to no restrictions on the construction schedule, provide a low score here.

Does ABC mitigate a critical environmental impact or sensitive environmental issue?	This is a measure of how using accelerated bridge construction methods can help mitigate impacts to the environment surrounding the project. Since accelerated methods allow a shorter on-site construction time, the impacts to the environment can be reduced. If the reduced on-site construction time provided by accelerated bridge construction methods mitigates a significant or critical environmental concern or issue, provide a high score here. If there are no environmental concerns that can be mitigated with accelerated construction methods, provide a low score here.
Compare Comprehensive Construction Costs	This is a measure of the complete comprehensive cost difference between conventional construction methods versus using an accelerated bridge construction method. Some costs will increase with the use of accelerated construction methods, such as the cost of the SPMT equipment and the learning curve that will be incorporated into using new technologies. However, some costs will decrease with the use of accelerated construction methods, such as the reduced cost for traffic control, equipment rentals, inspector wages, etc. Many of the reduced costs are a direct result of completing the project in less time. Use the cost comparisons in the table as guidance for scoring here.
Does ABC allow management of a particular risk?	This is an opportunity to add any project-specific items or unique issues that have risk associated with them that are not incorporated into another section in this text. Consider how ABC may or may not manage those particular risks.
Safety (Worker Concerns)	This is a measure of the relative safety of the construction workers between conventional construction methods and accelerated construction methods. The reduced on-site construction time from using accelerated bridge construction methods reduces the exposure time of workers in a construction zone, thus increasing safety. If a significant increase in safety can be seen by utilizing accelerated construction methods, provide a high score here. If utilizing accelerated construction methods does not provide additional safety, provide a low score here. Refer to the FDM for definitions of TMP Types.
Safety (Traveling Public Concerns)	This is a measure of the relative safety of the traveling public between conventional construction methods and accelerated construction methods. The reduced on-site construction time from using accelerated bridge construction methods reduces the exposure time of the traveling public in a construction zone, thus increasing safety. If a significant increase in safety can be seen by utilizing accelerated construction methods, provide a high score here. If utilizing accelerated construction methods does not provide additional safety, provide a low score here. Refer to the FDM for definitions of TMP Types.
Economy of Scale	This is a measure of how much repetition is used for elements on the project, which can help keep costs down. Repetition can be used on both substructure and superstructure elements. To measure the economy of scale, sum the total number of spans that will be constructed on the project. For example, if there are 2 bridges on the project that each have 2 spans, the total number of spans on the project is equal to 4. Use the notes in the table for scoring guidance here.

Weather Limitations for Conventional Construction?	This is a measure of the restrictions that the local weather causes for on-site construction progress. Accelerated bridge construction methods may allow a large portion of the construction to be done in a controlled facility, which helps reduce delays caused by inclement weather (rain, snow, etc.). Depending on the location and the season, faster construction progress could be obtained by minimizing the on-site construction time.
Use of Typical Standard Details (Complexity)	This is a measure of the efficiency that can be gained by using standard details that have already been developed and approved. If standard details are used, some errors in the field can be prevented. If new details are going to be created for a project, the contractors will be less familiar with the details and problems may arise during construction that were not considered in the design phase. Use the notes in the table for scoring guidance here.

Table 7.2-1
ABC Decision-Making Matrix Terms

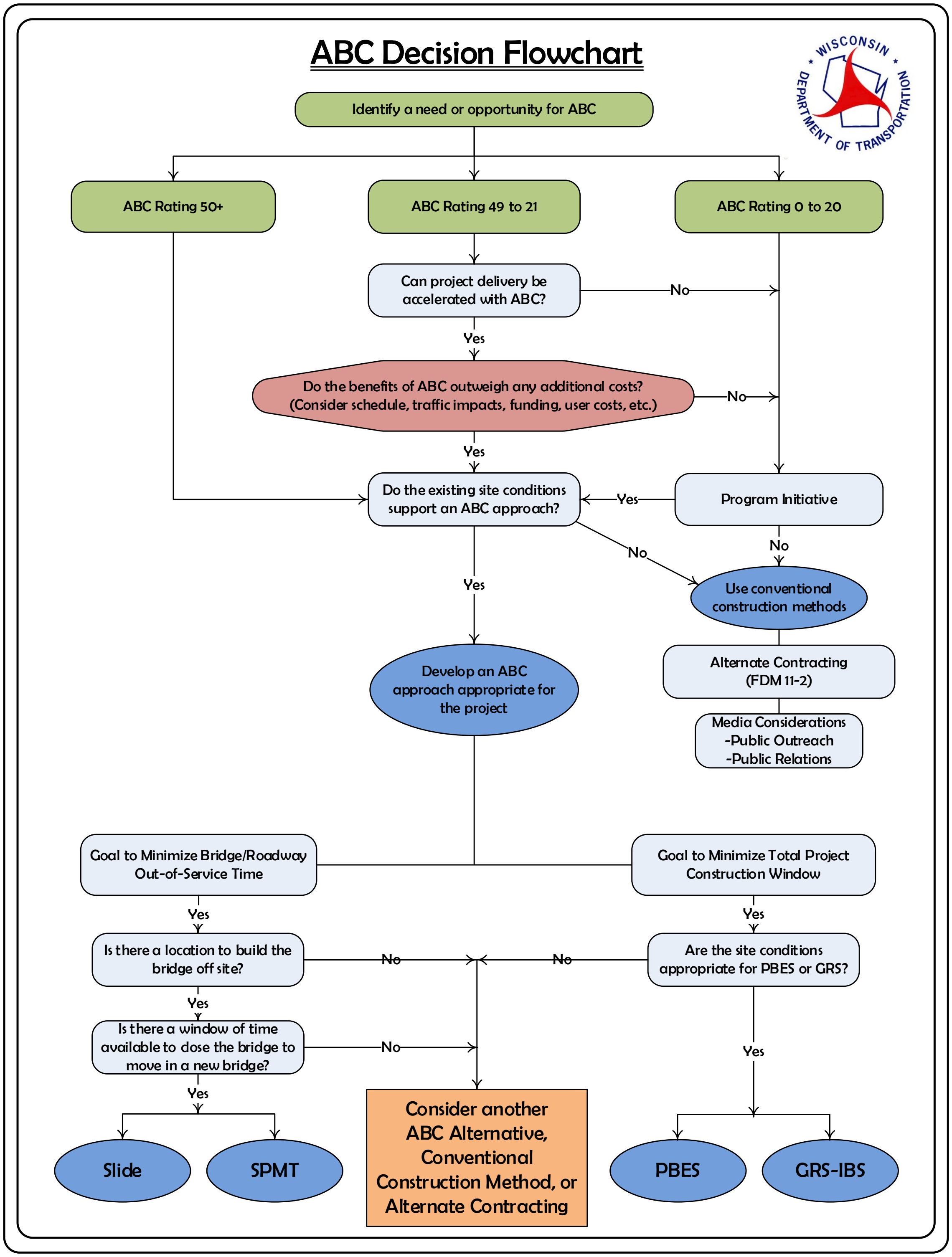

Figure 7.2-2
ABC Decision-Making Flowchart

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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 Facilities Development Manual (FDM) 20-5-15.

8.1.3.3 DOT Facilities Development Manual

Refer to FDM Chapter 10 – Erosion Control and Storm Water Quality, FDM Chapter 11 – Design, FDM Chapter 13 - Drainage, and FDM Chapter 20 - Environmental Documents, Reports and Permits.

8.1.4 Hydraulic Site Report

The "Stream Crossings Structure Survey Report" shall be submitted for all bridge and box culvert projects. When submitting preliminary structure plans for a stream crossing, a hydraulic site report shall also be included. 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), 2-year velocity, 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:

https://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].

8.3 Hydraulic Design of Bridges

Bridge design for roadway stream crossings requires analysis of the hydraulic characteristics for both the “existing conditions” and the “proposed conditions” of the project site. A thorough hydraulic analysis is essential to providing a properly sized, safe and economical bridge design and assessing the relative impact that the proposed bridge has on the floodplain. The following subsections discuss design considerations and hydraulic design procedures for bridges. See 8.6 Appendix 8-A for a checklist of items that need to be considered and included in the Hydraulic/Sizing report for stream crossing structures.

8.3.1 Hydraulic Design Factors

Several hydraulic factors dictate the design of both the bridge and the approach roadway within the floodplain limits of the project site. The critical hydraulic factors for design consideration are:

8.3.1.1 Velocity

Velocity through the bridge opening is a major design factor. Velocity relates to the scour potential in the bridge opening and the development of scour areas adjacent to the bridge. Examination of the “existing conditions” model, existing site conditions, soil conditions, and flooding history will give good insight to acceptable design velocity. Generally, velocities through bridges of less than 10 feet per second are acceptable.

8.3.1.2 Roadway Overflow

The vertical alignment of the approach grade is a critical factor in the bridge design when roadway overflow is a design consideration. The two important design features of roadway overflow is overtopping velocity and overtopping frequency. See 8.3.2.6.2

8.3.1.3 Bridge Skew

When a roadway is at a skew angle to the stream or floodway, the bridge shall also be at a skew to the roadway with the abutments and piers parallel to the flow of the stream. The hydraulic section through the bridge shall be the skewed section normal to the flow of the stream. Generally, in the design of stream crossing, the skew of the structure should be varied in increments of 5 degrees where practical. Improper skew can greatly aggravate the magnitude of scour.

8.3.1.4 Backwater and High-water Elevation

Roadways and bridges are generally restrictions to the normal flow of floodwaters and increase the flood profile in most situations. The increase in the flood profile is referred to as the backwater and the resultant upstream water surface elevation is referred to as the High-Water Elevation (HW).

The high-water elevation or backwater calculations at the bridge are directly related to the bridge size and roadway alignment, which dictates all of the aforementioned hydraulic design factors. A significant design consideration when computing backwater is the potential for increasing flood damage for upstream property owners. The Cooperative Agreement between the Wis. Department of Natural Resources (DNR) and Wis. Department of Transportation (DOT) (see 8.1.3.2) defines the policy for high-water elevation design. That portion of the Cooperative Agreement relating to floodplain considerations is based on the Wisconsin Adm. Rule NR116, “Wisconsin Floodplain Management Program”. It is advisable to thoroughly study both documents as they can significantly influence the hydraulic design of the bridge.

One very subtle backwater criteria which is not addressed under the guidelines of the DNR-DOT Cooperative Agreement, is the backwater produced for flood events less than the 100 year frequency flood. Design consideration should be given to the more frequent flood events when there is potential for increasing the extent and frequency of flood damage upstream.

8.3.1.5 Freeboard

Freeboard is defined as the vertical distance between the low cord elevation of the bridge superstructure and the high-water elevation. A freeboard of 2.0 feet is the desirable minimum for all types of superstructures. However, economics, vertical and horizontal alignment, and the scope of the project may force a compromise to the 2 foot minimum freeboard. For these situations, close evaluation shall be made of the type and amount of debris and ice that would pass through the structure. Freeboard should be computed using the low chord elevation at the upstream face on the lower end of the bridge. The calculated 100-year high water elevation at a cross section that is approximately one bridge length upstream should be used to check freeboard.

It has become common practice that if debris and ice are a potential problem, or adequate freeboard cannot be provided, a concrete slab superstructure is preferred. A girder superstructure may be susceptible to damage when ice and/or debris is a significant problem. Girder structures are more susceptible to damage associated with buoyancy and lateral hydrostatic forces. In situations where the superstructure may be inundated during major flood events, it is recommended that the girders be anchored, tied or blocked so they cannot be pushed or lifted off the substructure units by hydraulic forces. In addition, air vents near the top of the girder webs can allow entrapped air to escape and thus may reduce buoyancy forces. The use of Precast Pretensioned Slab and Box Sections is allowed where desirable freeboard cannot be provided and conventional cast in place slabs cannot be employed. The following requirements should be met:

- Precast Pretensioned Slab and Box Sections may be in the water for the 100-year flood. The designer will be responsible for ensuring the stability of the structure for buoyant and lateral forces.

- If Precast Pretensioned Slab and Box Sections are in contact with water for flood events equal to or less than a 5-year event, the Precast Pretensioned Slab and Box Sections must be cast solid.

- If Precast Pretensioned Slab and Box Sections are in contact with water for flood events equal to or less than a 100-year event, the void in Precast Pretensioned Slab and Box Sections must be cast with a non-water absorbing material.

8.3.1.6 Scour

Investigation of the potential for scour at the bridge site is a design consideration for the bridge opening geometry and size, as well as pier and abutment design. Bridges shall be designed to withstand the effects of scour from a super-flood (a flood exceeding the 100-year flood) without failing; i.e., experiencing foundation movement of a magnitude that requires corrective action. See 8.3.2.7. Generally, scour associated with a 100-year event without significant reduction in foundation factor of safety will accomplish this objective. For situations where a combination of flow through a bridge and over the roadway exist, scour should also be evaluated for flow conditions at the onset of flow over topping when velocity through the bridge may be the greatest.

8.3.2 Design Procedures

8.3.2.1 Determine Design Discharge

See 8.2 for procedures.

8.3.2.2 Determine Hydraulic Stream Slope

The primary method of determining the hydraulic slope of a stream is surveying the water surface elevation through a reach of stream 1500 feet upstream to 1500 feet downstream of the site. Intermediate points through this reach should also be surveyed to detect any significant slope variation.

There are situations, particularly on flat stream profiles, where it is difficult to determine a realistic slope using survey data. This will occur at normal water surface elevation at the mouth of a stream, upstream of a dam, or other significant restriction in the stream. In this case a USGS 7-1/2” quadrangle map and existing flood studies of the stream can be investigated to determine a reasonable stream slope.

8.3.2.3 Select Floodplain Cross-Section(s)

Generally, a minimum of two floodplain valley cross-section(s) are required to perform the hydraulic analysis of a bridge. The sections shall be normal to the stream flow at flood stage and approximately one bridge length upstream and downstream of the structure. A detailed cross-section of one or both faces of the bridge will also be required. If the section is skewed to the flow, the horizontal stationing shall be adjusted using the cosine of the skew angle.

If the downstream boundary condition of the hydraulic model is using normal depth, then the most downstream cross-section in the model should be located far enough downstream from the bridge and should reflect the natural floodplain conditions.

Field survey cross-sections will be needed when a contour map is plotted using stereographic methods. A field survey section is needed for that portion below the normal water surface.

Cross-sections taken from contour maps are acceptable when the information is supplemented with field survey sections and data. Additional sections may be required to develop a proper hydraulic model for the site.

The hydraulic cross-sections should not include slack water portions of the flood plain or portions not contributing to the downstream movement of water.

Refer to FDM 9-55 for a discussion of Drainage Structure Surveys.

8.3.2.4 Assign "Manning n" Values to Section(s)

"Manning n" values are assigned to the cross-section sub-areas. Generally, the main channel will have different "manning n" values than the overbank areas. Values are chosen by on-site inspection, pictures taken at the section, and use of aerial photos defining the extent of each "n" value. There are several published sources on open channel hydraulics which contain tables for selecting appropriate "n" values. See 8.5 References (5) and (6).

8.3.2.5 Select Hydraulic Model Methodology

There are several public and private computer software programs available for modeling open channel hydraulics, bridge hydraulics, and culvert hydraulics. Three public domain computer software programs that are most prevalent and preferred in Wisconsin bridge design work are "HEC-RAS", "WSPRO" and "HY8".

The HEC-RAS program is currently the most widely used methodology for floodplain and bridge hydraulic modeling. HEC-RAS has more options and capabilities than WSPRO when modeling complex floodplains and requires a greater amount of expertise to apply. HEC-RAS should be used where existing HEC-2 data is available from a previous Flood Insurance Study. The WSPRO methodology is tailored specifically for bridge hydraulics with many appropriate default coefficients and analysis options. More information on these two programs is given below. "HY8" is a FHWA sponsored culvert analysis package based on the FHWA publication "Hydraulic Design of Highway culverts" (HDS-5), see 8.5 Reference (13).

1. HEC-RAS

   The hydrologic Engineering Center's River Analysis System (HEC-RAS) is the first of the U.S. Army Corps of Engineers "Next Generation" software packages. It is the successor to the HEC-2 program, which was originally developed by the Corps of Engineers in the early 1970's. HEC-RAS includes several data entry, graphing, and reporting capabilities. It is well suited for modeling water flowing through a system of open channels and computing water surface profiles to be used for floodplain management and evaluation of floodway encroachments. HEC-RAS can also be used for bridge and culvert design and analysis and channel modification studies.

   For a complete treatise on the methodology of the program, see 8.5 reference (7), (8) and (9). The HEC-RAS program and supporting documentation can be downloaded from the U.S. Army Corps of Engineers web site: http://www.hec.usace.army.mil/software/hec-ras/. A list of vendors for HEC-RAS is also available on this web site.

2. WSPRO

   "Water Surface Profiles (WSPRO)" is a computer program developed by the U.S. Geological Survey under contract with Federal Highway Administration. WSPRO was specifically oriented toward hydraulic design of highway bridges although it is equally suitable for water surface profile computations unrelated to highway and bridge design.

   The program uses bridge backwater computations based on analyses documented in the USGS publication entitled Measurement of Peak Discharge at Width Contractions by Indirect Methods, see 8.5 reference (10).

   For a complete treatise on the methodology of the program, see 8.5 reference (11) and (12). The WSPRO program and supporting documentation can be downloaded from the following FHWA web site, or can be obtained through "McTrans" or "PcTrans". See 8.7 Appendix 8-B.

   http://www.fhwa.dot.gov/engineering/hydraulics/software.cfm

3. HY8

   HY8 is a computer program that uses the FHWA culvert hydraulic approaches and protocols as documented in the publication "Hydraulic Design Series 5: Hydraulic Design of Highway Culverts" (HDS-5). See 8.5 reference (13). HY8 can perform hydraulic computations for circular, rectangular, elliptical, metal box, high and low profile arch, as well as user defined geometry culverts. FHWA recently released a new Windows based version of the HY-8 culvert program. The methodology used by HY8 is discussed in 8.4.2.4. This program can be downloaded from the FHWA web site: http://www.fhwa.dot.gov/engineering/hydraulics/software.cfm.

8.3.2.6 Develop Hydraulic Model

First, a hydraulic model shall be developed for the "existing conditions" at the bridge site. This shall become the basis for hydraulic design of "proposed conditions" for the project and allows for an assessment of the relative hydraulic changes associated with the proposed structure. Special attention should be given to historic high-water and flood history, evidence of scour (high velocity), roadway overtopping, existing high-water, and compatibility with existing Flood Insurance Study (FIS) profiles. When current information and/or estimates of site conditions or flows differ significantly from adopted regulatory information (FIS), it may be necessary to compute both "design" and "regulatory" existing and proposed conditions.

There are a number of encompassing features of a steady state (flow is constant) hydraulic model for a roadway stream crossing. They include the natural adjacent floodplain, subject structure, any supplemental structures, and the roadway. Accurate modeling and calculations need to account for all potential conveyance mechanisms. Generally, most modern step-backwater methodologies can incorporate all of the above elements in the evaluation of hydraulic characteristics of the project site.

8.3.2.6.1 Bridge Hydraulics

The three most common types of flow through bridges are free surface flow (low flow), free surface (unsubmerged) orifice flow and submerged orifice flow. The latter two are also referred to as pressure flow. All of the above flow conditions may also occur simultaneously with flow over the roadway.

There are situations in which steep stream slopes are encountered and the flow may be supercritical (Froude No. > 1). This is a situation in which theoretically no backwater is created. For critical and supercritical flow situations the profile calculation would proceed from upstream to downstream. If this situation is encountered, the accuracy of the hydraulic model may be suspect and it is questionable whether the bridge should impose any constrictions on the stream channel. Sufficient clearance should be provided to insure that the superstructure will not come in contact with the flow.

Generally, in Wisconsin, most natural stream flow is in a sub-critical (Froude No. < 1) regime. Therefore, the water surface profile calculation will proceed from downstream to upstream.

Sample bridge hydraulic problems using HEC-RAS can be found in the HEC-RAS Applications Guide[9].

8.3.2.6.2 Roadway Overflow

One potential element in developing a hydraulic model for a stream crossing is roadway overflow. It is sometimes necessary to compute flow over highway embankments in combination with flow through structure openings. Most automated methodologies will incorporate the division of flow through a structure and over the road in determination of the solution. The WSPRO methodology will conduct the “combined flow” solution and internally determine and adjust the coefficient of discharge for both the structure and roadway weir section. Other methodologies, such as HEC-RAS, rely on user defined coefficients for both the structure and roadway flow solutions. The discharge equation and coefficients for flow over a highway embankment are given in this section.

The geometry and flow pattern for a highway embankment are illustrated in Figure 8.3-4. Under free flow conditions critical depths occur near the crown line. The head (H) is referred to the elevation of the water above the crown, and the length (L), in direction of flow, is the distance between the points of the upstream and downstream embankment faces (edge of shoulder). The length (B) of the embankment has no influence on the discharge coefficient.

The weir discharge equation is:

$$Q = k_t \cdot C_f \cdot B \cdot H^{3/2}$$

Where:

Q	=	discharge
C_f	=	coefficient of discharge for free flow conditions

B = length of flow section along the road normal to the direction of flow

H = total head = h + h_v

k_t = submergence factor

The length of overflow section (B) will be a function of the roadway profile grade line and depth of over-topping (h). Coefficient (C_f) is obtained by computing h/L and using Figure 8.3-1 or Figure 8.3-2, for paved or gravel roads.

The degree of submergence of a highway embankment is defined by ration ht/H. The effect of submergence on the discharge coefficient (C_f) is expressed by the factor k_t as shown in Figure 8.3-3. The factor k_t is multiplied by the discharge coefficient (C_f) for free-flow conditions to obtain the discharge coefficient for submerged conditions. For roadway overflow conditions with high degree of submergence, HEC-RAS switches to energy based calculations of the upstream water surface. The default maximum submergence is 0.95, however that criterion may be modified by the user.

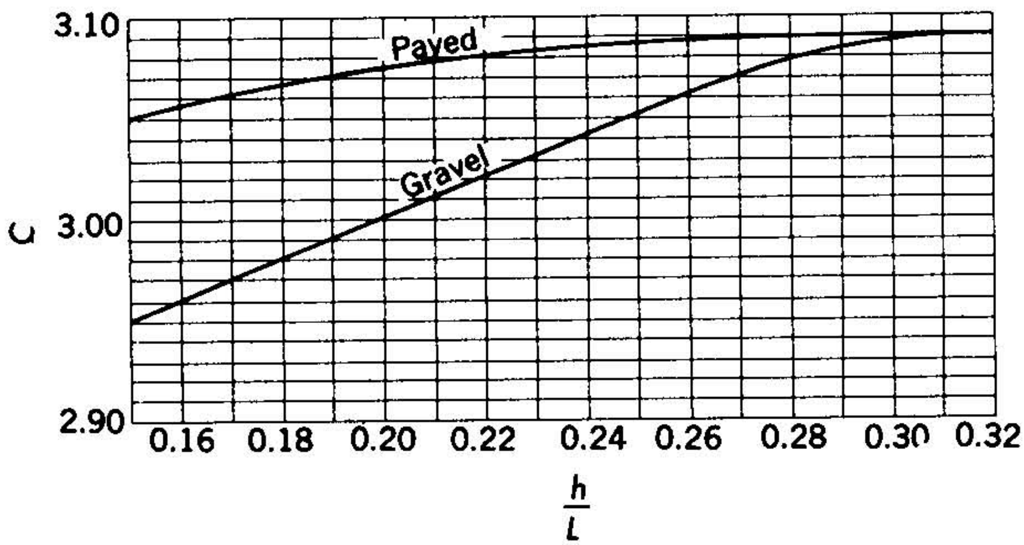

Figure 8.3-1
Discharge Coefficients, C_f, for Highway Embankments for H/L Ratios > 0.15

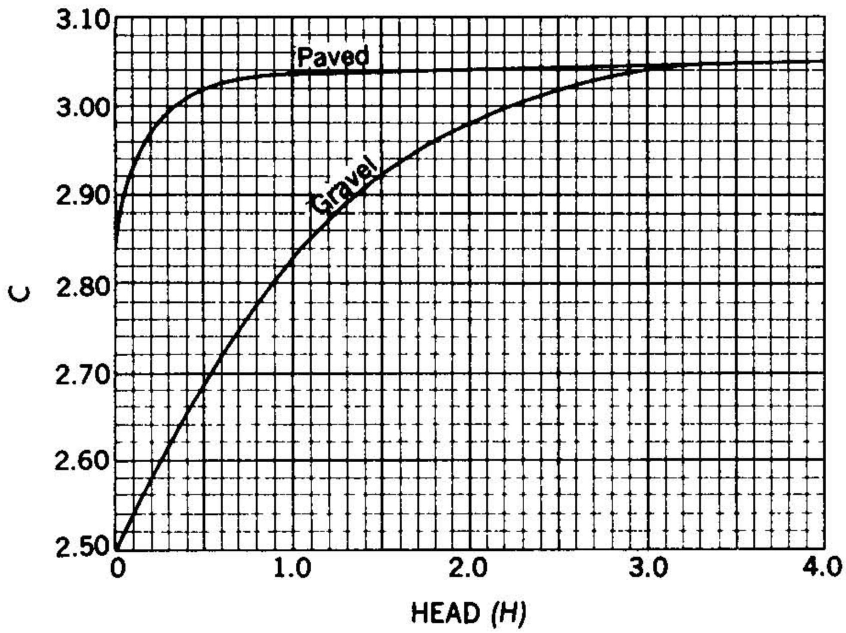

Figure 8.3-2
Discharge Coefficients, C_f, for Highway Embankments for H/L Ratios < 0.15

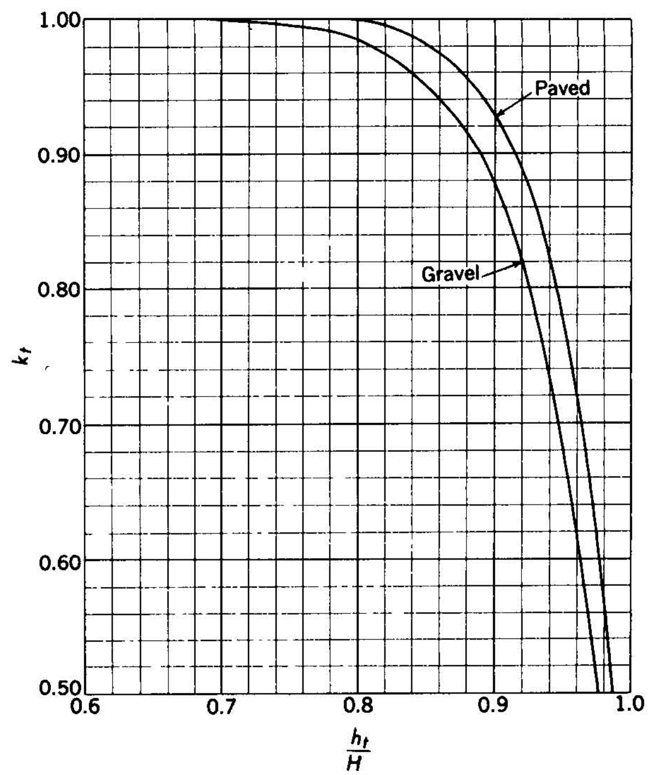

Figure 8.3-3
Definition of Adjustment Factor, k_t, for Submerged Highway Embankments

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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:

https://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.

9.8 Painting

All highway grade separation structures require steel girders to be painted because unpainted steel is subject to additional corrosion from vehicle salt spray. Additional discussion on painting is presented in Chapter 24 – Steel Girder Structures and Chapter 40 – Bridge Rehabilitation. The current paint system used for I-girders is the three-coat epoxy system specified in Section 517 of the *Standard Specifications*. Tub girders utilize a two-coat polysiloxane system, which includes painting of the inside of the tubs.

Recommended paint colors and *AMS Standard Color Numbers* for steel girders in Wisconsin shall be in accordance with AMS Standard 595A and are:

White (For Inside of Box Girders)	#27925
Blue (Medium Sky Blue Tone)	#25240
[1] Brown (Similar to Weathering Steel)	#20059
Gray (Light Gray)	#26293
Green (Medium Tone)	#24260
Reddish-Brown (Red Brick Tone)	#20152
Gray (Dark Gray-DNR Request)	#26132
Black	#27038

Table 9.8-1
Standard Colors for Steel Girders

[1] Shop applied color for weathering steel.

AMS Standard 595A can be found at www.ams-std-595-color.com/

All steel bearing components which are not welded to the girder or do not have a Teflon or bronze surface, and anchor bolts shall be galvanized. In addition to galvanizing, some bearing components may also be field or shop painted as noted in the Standards for Chapter 27 – Bearings.

All new structural steel is blast cleaned including weathering steel. It has been shown that paint systems perform well over a longer period of time with proper surface preparation. The blast cleaned surface is a very finely pitted surface with pit depths of 1 ½ mils.

Corrosion of structural steel occurs if the agents necessary to form a corrosion cell are present. A corrosion cell is similar to a battery in that current flows from the anode to the cathode. As the current flows, corrosion occurs at the anode and materials expand. Water carries the electrical current between the anode and cathode. If there is salt in the water, the current travels much faster and the rate of corrosion is accelerated. Oxygen combines with the materials to help form the anodic corrosion cell.

The primary reason for painting steel structures is for the protection of the steel surface. Appearance is a secondary function that is maintained by using compatible top coatings over epoxy systems. Regarding appearance with respect to color retention, black is good, blues and greens are decent, and reddish browns are acceptable, but not the best. Reds are highly discouraged and should not be used.

Paint applied to the steel acts as a moisture barrier. It prevents the water from contacting the steel and then a corrosion cell cannot be formed. When applying a moisture barrier, it is important to get an adhering, uniform thickness as well as an adequate thickness. The film thickness of paint wears with age until it is finally depleted. At this point the steel begins to corrode as moisture is now present in the corrosion cell. If paint is applied too thick, it may crack and/or check due to temperature changes and allow moisture to contact the steel long before the film thickness wears down.

The paint inspector uses a paint gauge to randomly measure the film thickness of the paint according to specifications. Wet film thickness is measured and it is always thicker than the dry film thickness. A vehicle is added to the paint solids so that the solids can be applied to a surface and then it evaporates leaving only the solids on the surface. The percent of solids in a gallon of paint gives an estimate of the wet to dry film thickness ratio.

Recommended paint maintenance is determined with assistance from the Wisconsin Structures Asset Management System (WiSAMS), which utilizes information provided by the routine bridge inspections.

Recommended paint colors and *AMS Standard Color Numbers* for concrete in Wisconsin shall be in accordance with AMS Standard 595A and are:

Pearl Gray	#26622
Medium Tan	#33446
Gray Green	#30372
Dark Brown	#30140
Dawn Mist (Grayish Brown)	#36424
Lt. Coffee (Creamy Brown)	#33722

Table 9.8-2
Standard Colors for Concrete

Most paints require concrete to be a minimum of 30 days old before application. This should be considered when specifying completion times for contracts.

9.10 Granular Materials

Several types of granular materials are used for backfilling excavations, providing foundation improvements, and reinforcing soils. Table 9.10-5 provides recommended uses and notes for commonly used granular materials for structures. Refer to the specifications for material gradations, testing, compaction, and other requirements specific for the application. Refer to 6.4.2 for plan preparations.

Granular pay limits should be shown on all structure plans. See Standards for typical backfill limits and plan notes.

<table>
<tr><th>Granular Material Type</th><th>Uses</th><th>Notes</th></tr>
<tr><td>Backfill Structure – Type A</td><td><u>Backfill</u>
• Abutments
• Retaining walls</td><td rowspan="2">Type A facilitates better drainage than Type B.

Type A may be substituted for Type B material per specifications.</td></tr>
<tr><td>Backfill Structure – Type B</td><td><u>Backfill</u>
• Box culverts
• Structural plate pipes
• Pipe arches
<u>Retained Backfill (if needed)</u>
▪ Various structures
<u>Foundation</u>
• Abutments
• Retaining walls</td></tr>
<tr><td>Backfill Granular – Grade 1</td><td rowspan="2">Refer to Facilities Development Manual (FDM) for usages</td><td rowspan="2">Grade 1 may be substituted for Grade 2 material per specifications.</td></tr>
<tr><td>Backfill Granular – Grade 2</td></tr>
<tr><td>Base Aggregate Dense 1 1/4-inch</td><td>• Structural approach (base)
• GRS Walls (reinforced soil foundation and approach)</td><td></td></tr>
<tr><td>Reinforced Soils</td><td>• MSE Walls</td><td>Backfill included in MSE Wall bid items.</td></tr>
<tr><td>Base Aggregate Open Graded</td><td>• GRS Walls (reinforced soil)
• MSE Walls (for elevations below HW100)</td><td></td></tr>
<tr><td>Breaker Run</td><td>• Box culverts (foundation)</td><td>See Standard Detail 9.01 for alternatives and notes</td></tr>
<tr><td>Flowable Backfill</td><td>• Soldier pile walls</td><td></td></tr>
</table>

Table 9.10-5
Recommendations for Granular Material Usage

9.11 References

1. Ghorbanpoor, A., Kriha, B., Reshadi, R. *Aesthetic Coating for Steel Bridge Components – Amended Study*. S.1.: Wisconsin Department of Transportation, Final Report No. 0092-11-07, 2015.

9.12 Appendix - Draft Bar Tables

The following Draft Bar Tables are provided for information only. We expect the tables to be moved into the main text of Chapter 9 in January of 2020, and at that time to begin their use. We are delaying their use to allow time for modification of details and software that are affected.

The 2015 Interim Revisions to the AASHTO LRFD Bridge Design Specifications (7th Edition), modified the tension development lengths and tension lap lengths for straight deformed bars as follows - (**LRFD [**article number**]** references below match the AASHTO LRFD Bridge Design Specifications – 8th Edition):

The tension development length, ℓ_d ,shall not be less than the product of the basic tension development length, ℓ_{db} , and the appropriate modification factors, λ_i . **LRFD [5.10.8.2.1a]**

$$\ell_d = \ell_{db} \cdot (\lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{er}) / \lambda$$

in which: $\ell_{db} = 2.4 \cdot d_b \cdot [f_y / (f'_c)^{1/2}]$

where:

ℓ_{db} = basic development length (in.)

λ_{rl} = reinforcement location factor

λ_{cf} = coating factor

λ = conc. density modification factor ; for normal weight conc. = 1.0 , **LRFD [5.4.2.8]**

λ_{rc} = reinforcement confinement factor

λ_{er} = excess reinforcement factor

f_y = specified minimum yield strength of reinforcement (ksi)

d_b = nominal diameter of reinforcing bar (in.)

f'_c = compressive strength of concrete for use in design (ksi)

Top bars will continue to refer to horizontal bars placed with more than 12” of fresh concrete cast below it. Bars not meeting this criteria will be referred to as Others.

Per **LRFD [5.10.8.4.3a]**, there are two lap splice classes, Class A and Class B.

- Class A lap splice ……………………1.0 ℓ_d
- Class B lap splice …………………... 1.3 ℓ_d

The criteria for where to apply each Class is covered in the above reference.

Draft Table

Epoxy Coated (f'_c = 4,000 psi; f_y = 60,000 psi)

Basic Lap - Others					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	1'-6"	1'-6"	1'-11"	1'-11"
	2.0"	1'-6"	1'-6"	1'-11"	1'-11"
	≥ 2.5"	1'-6"	1'-6"	1'-11"	1'-11"
5	1.5"	2'-3"	2'-3"	3'-0"	3'-0"
	2.0"	2'-3"	1'-10"	3'-0"	2'-4"
	≥ 2.5"	2'-3"	1'-10"	3'-0"	2'-4"
6	1.5"	2'-11"	2'-9"	3'-10"	3'-7"
	2.0"	2'-11"	2'-9"	3'-10"	3'-7"
	≥ 2.5"	2'-11"	2'-2"	3'-10"	2'-10"
7	1.5"	3'-7"	3'-7"	4'-8"	4'-8"
	2.0"	3'-6"	3'-2"	4'-6"	4'-2"
	≥ 2.5"	3'-6"	3'-2"	4'-6"	4'-2"
8	1.5"	4'-6"	4'-6"	5'-11"	5'-11"
	2.0"	4'-6"	3'-8"	5'-11"	4'-9"
	≥ 2.5"	4'-6"	3'-8"	5'-11"	4'-9"
9	1.5"	5'-9"	5'-7"	7'-5"	7'-4"
	2.0"	5'-9"	4'-6"	7'-5"	5'-10"
	≥ 2.5"	5'-9"	4'-1"	7'-5"	5'-4"
10	1.5"	7'-4"	6'-9"	9'-7"	8'-10"
	2.0"	7'-4"	5'-6"	9'-7"	7'-2"
	≥ 2.5"	7'-4"	4'-10"	9'-7"	6'-3"
11	1.5"	9'-1"	8'-2"	11'-9"	10'-7"
	2.0"	9'-1"	6'-8"	11'-9"	8'-7"
	≥ 2.5"	9'-1"	6'-0"	11'-9"	7'-9"

Epoxy Coated (f'_c = 4,000 psi; f_y = 60,000 psi)

Horizontal Lap w/ >12" Concrete Cast Below - Top					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	1'-11"	1'-11"	2'-6"	2'-6"
	2.0"	1'-11"	1'-11"	2'-6"	2'-6"
	≥ 2.5"	1'-11"	1'-11"	2'-6"	2'-6"
5	1.5"	2'-7"	2'-7"	3'-4"	3'-4"
	2.0"	2'-7"	2'-4"	3'-4"	3'-1"
	≥ 2.5"	2'-7"	2'-4"	3'-4"	3'-1"
6	1.5"	3'-4"	3'-1"	4'-4"	4'-0"
	2.0"	3'-4"	3'-1"	4'-4"	4'-0"
	≥ 2.5"	3'-4"	2'-10"	4'-4"	3'-8"
7	1.5"	4'-1"	4'-1"	5'-3"	5'-3"
	2.0"	4'-0"	3'-7"	5'-2"	4'-8"
	≥ 2.5"	4'-0"	3'-7"	5'-2"	4'-8
8	1.5"	5'-2"	5'-2"	6'-8"	6'-8"
	2.0"	5'-2"	4'-1"	6'-8"	5'-4"
	≥ 2.5"	5'-2"	4'-1"	6'-8"	5'-4"
9	1.5"	6'-6"	6'-4"	8'-5"	8'-3"
	2.0"	6'-6"	5'-1"	8'-5"	6'-7"
	≥ 2.5"	6'-6"	4'-8"	8'-5"	6'-0"
10	1.5"	8'-4"	7'-8"	10'-10"	10'-0"
	2.0"	8'-4"	6'-3"	10'-10"	8'-1"
	≥ 2.5"	8'-4"	5'-6"	10'-10"	7'-1"
11	1.5"	10'-3"	9'-3"	13'-4"	12'-0"
	2.0"	10'-3"	7'-6"	13'-4"	9'-9"
	≥ 2.5"	10'-3"	6'-10"	13'-4"	8'-10"

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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.

11.3.1.3 Battered Piles

Battered piles are used to resist large lateral loads or when there is insufficient lateral soil resistance within the initial 4 to 5 pile diameters of embedment. Battered piles are frequently used in combination with vertical piles. The lateral resistance of battered piling is a function of the vertical load applied to the pile group. Since the sum of the forces at the pile head must equal zero, increasing the number of battered piles does not necessarily increase the lateral load capacity of the pile group. Both the lateral passive resistance of the soil above the footing as well as the sliding resistance developed at the base of footing are generally neglected in design. See the standard details for further guidance when battered piles are used.

Piles are typically battered at 1 horizontal to 4 vertical. Hammer efficiencies must be reduced when piles are battered. Where negative skin friction loads are anticipated, battered piles should not be considered.

11.3.1.4 Corrosion Loss

Piling should be designed with sufficient corrosion resistance to assure a minimum design life of 75 years. Corrosive sites may included those with combinations of organic soils, high water table, man-made coal combustion products or waste materials, and those materials that allow air infiltration such as wood chips. Experience indicates that corrosion is not a practical problem for steel piles driven in natural soil, due primarily to the absence of oxygen in the soil. However, in fill material at or above the water table, moderate corrosion may occur and protection may be required. Concrete piles are prone to deterioration from exposure to excess concentrations of sulfate and/or chloride. Special consideration (including thicker pile shells, heavier pile sections, painting and concrete encasement) should be given to permanent steel piling that is used in areas of northern Wisconsin which are inhabited by corrosion causing bacteria (see Facilities Development Manual 13-1-15). Typically, WisDOT does not increase pile sections or heavier pile sections to provide corrosion protection outside of these areas.

At potentially corrosive sites, encasement by cast-in-place concrete can provide the required protection for piles extending above the ground surface. All exposed piling should be painted. Additional guidance on corrosion is provided in **LRFD [10.7.5]**.

11.3.1.5 Pile Points

A study was conducted on the value of pile tips (pile points) on steel piles when driving into rock. The results indicated that there was very little penetration difference between the piles driven with pile points and those without. The primary advantages for specifying pile points are for penetrating or displacing boulders, driving through dense granular materials and hardpan layers, and to reduce the potential pile damage in hard driving conditions. Piling can generally be driven faster and in straighter alignment when points are used.

Conical pile points have also been used for round, steel piling (friction and point-bearing) in certain situations. These points can also be flush-welded if deemed necessary.

Standard details for pile points are available from the approved suppliers that are listed in WisDOT's current Product Acceptability List (PAL).

Pile points and preboring are sometimes confused. They are not interchangeable. Pile points can be used to help drive piles through soil that has gravel and/or cobbles or presents other difficult driving conditions. They can also be used to get a good ‘bite’ when ending piles on sloping bedrock surfaces. Points cannot be used to ensure that piles penetrate into competent bedrock. They may assist in driving through weathered zones of rock or soft rock, but will generally not be effective when penetration into hard rock is desired.

11.3.1.6 Preboring

If embedment into rock is required or minimum pile penetration is doubtful, preboring should be considered. Preboring is required for displacement piles driven into new embankment that are over 10 feet in height. The WisDOT has developed special provisions to provide preboring requirements.

Except for point resistance piles, preboring should be terminated at least 5 feet above the scheduled pile tip elevation. When the pile is planned to be point resistance on rock, preboring may be advanced to plan pile tip elevation. Restrike is not performed when point-bearing piles are founded in rock within prebored holes. Preboring should only be used when appropriate, since many bridge contractors do not own the required construction equipment necessary for this work.

The annular space between the wall of the prebored hole and the pile is required to be backfilled. The annulus in bedrock should be filled with concrete or cement grout after the pile has been installed. Clean sand may be used to backfill the annulus over the depth of soil overburden. Backfill material should be deposited with either a tremie pipe or concrete pump to reduce potential arching (bridging) and assure that the complete depth of hole is filled.

11.3.1.7 Seating

Care must be taken when seating end bearing piles, especially when seating on bedrock with little to no weathered zone. When a pile is firmly seated on rock in prebored holes, pile driving to refusal is not required or recommended, to avoid driving overstress and pile damage. After reaching the predetermined prebore elevation, piles founded in soil are driven with a pile hammer to achieve the specified average penetration or set per blow for the final ten blows of driving.

11.3.1.8 Pile Embedment in Footings

The length of pile embedment in footings is determined based on the type and function of substructure unit and the magnitude of any uplift load.

WisDOT policy item:

Use a minimum 6-inch pile embedment in footings. This embedment depth is considered to result in a free (pinned) head connection for analysis. When the pile embedment depth into the footing is 2.0 feet or greater, the designer can assume a fixed head connection for analysis.

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12.4 Abutment Wing Walls

This section provides general equations used to compute wing wall lengths, as well as a brief description of wing wall loads and parapets.

12.4.1 Wing Wall Length

Wing walls must be long enough to retain the roadway embankment based on the allowable slopes at the abutment. A slope of 2:1 is usually used, and a slope greater than 2:1 is usually not permitted. Current practice is to round up to the next available wing length based on 2 feet increments and to consider an additional 2 feet to match other wing lengths. When setting wing wall lengths, be sure that the theoretical slope of the earth does not fall above the bridge seat elevation at the corner. Roadway embankment slopes are typically limited to a slope of 2.5:1 and may require a traffic barrier. Refer to the Facilities Development Manual (FDM) for roadway embankment slopes and traffic barrier requirements.

12.4.1.1 Wings Parallel to Roadway

The calculation of wing wall lengths for wings that are parallel to the roadway is illustrated in Figure 12.4-1 and Figure 12.4-2. Wing lengths should be lengthened an additional 2 feet to allow for the finished grade to intersect the top of wall 2 feet from the end of wings for erosion control protection. The additional 2 feet of wing wall length is only intended for wings parallel to the roadway.

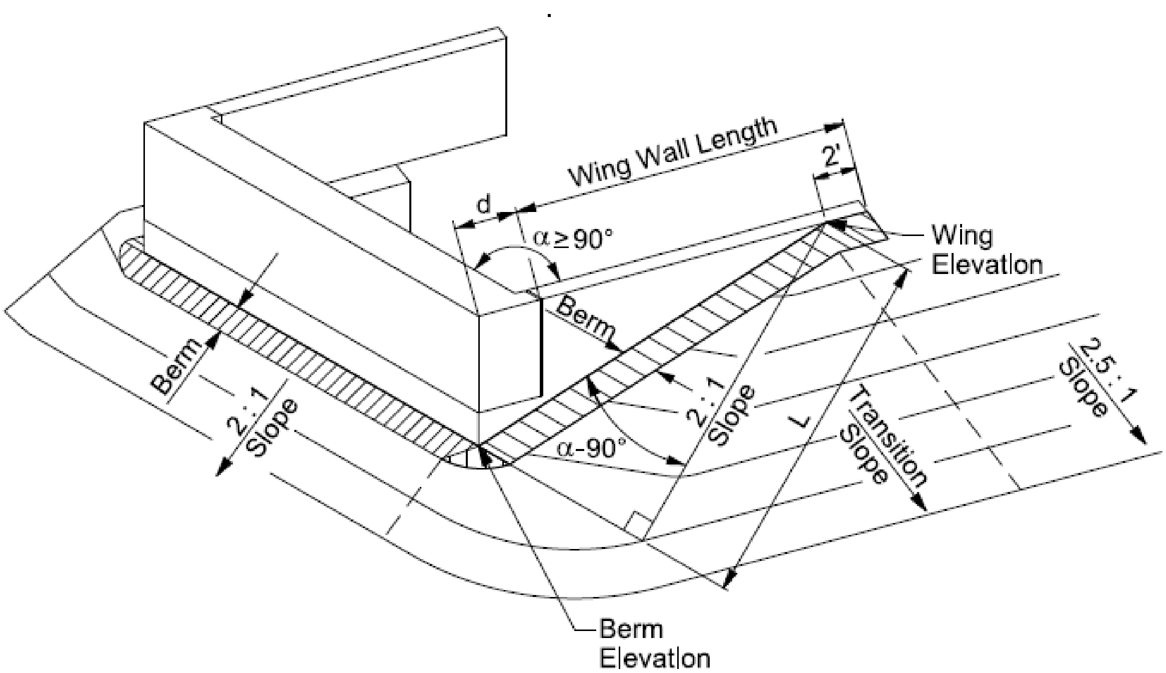

Figure 12.4-1
Wings Parallel to Roadway and Wing Wall Angle ≥ 90°

For wing wall angle, $\alpha \geq 90°$:

$$L = (\text{Wing Elevation} - \text{Berm Elevation})\ (2)$$

$$\text{Wing Wall Length} = \frac{L}{\cos(\alpha - 90^\circ)} - d + 2.0 \text{ feet}$$

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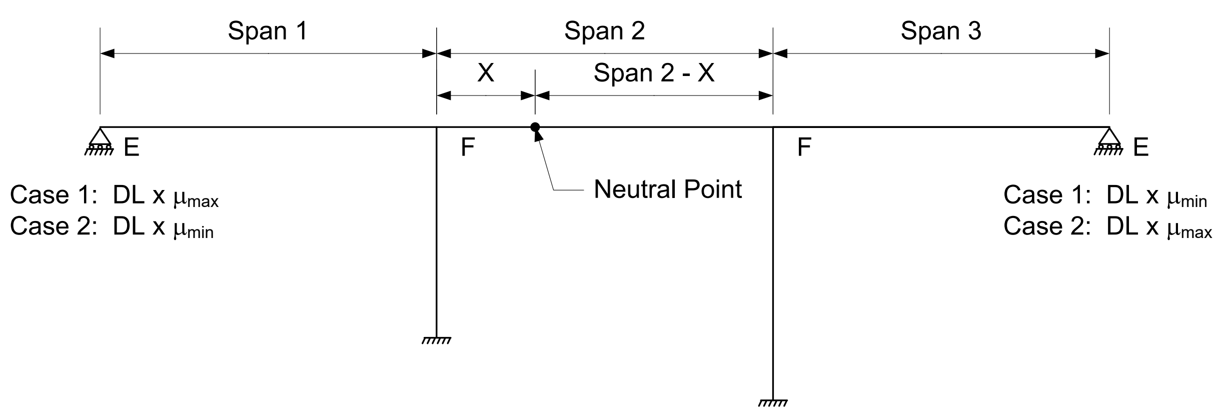
In determining the temperature forces, TU, applied to each substructure unit, the entire bridge superstructure length between expansion joints is considered. In all cases, there is a neutral point on the superstructure which does not move due to temperature changes. All temperature movements will then emanate outwards or inwards from this neutral point. This point is determined by assuming a neutral point. The sum of the expansion forces and fixed pier forces on one side of the assumed neutral point is then equated to the sum of the expansion forces and fixed pier forces on the other side of the assumed neutral point. Maximum friction coefficients are assumed for expansion bearings on one side of the assumed neutral point and minimum coefficients are assumed on the other side to produce the greatest unbalanced force for the fixed pier(s) on one side of the assumed neutral point. The maximum and minimum coefficients are then reversed to produce the greatest unbalanced force for the pier(s) on the other side of the assumed neutral point. For semi-expansion abutments, the assumed minimum friction coefficient is 0.06 and the maximum is 0.10. For laminated elastomeric bearings, the force transmitted to the pier is the shear force generated in the bearing due to temperature movement. Example E27-1.8 illustrates the calculation of this force. Other expansion bearing values can be found in Chapter 27 – Bearings. When writing the equation to balance forces, one can set the distance from the fixed pier immediately to one side of the assumed neutral point as ‘X’ and the fixed pier immediately to the other side as (Span Length – ‘X’). This is illustrated in Figure 13.4-1.



$$\text{Case1: } \sum DL \times \mu_{max} + \sum \frac{3EI\alpha TX}{144h^3} = \sum \frac{3EI\alpha T(\text{Span}\,2 - X)}{144h^3} + \sum DL \times \mu_{min}$$

(Lt. Abutment) (Pier 1) (Pier 2) (Rt. Abutment)

Figure 13.4-1
Neutral Point Location with Multiple Fixed Piers

As used in Figure 13.4-1:

E = Column or shaft modulus of elasticity (ksi)

I	=	Column or shaft gross moment of inertia about longitudinal axis of the pier (in^4)
α	=	Superstructure coefficient of thermal expansion (ft/ft/°F)
T	=	Temperature change of superstructure (°F)
μ	=	Coefficient of friction of the expansion bearing (dimensionless)
h	=	Column height; top of footing to top of cap (ft)
DL	=	Total girder dead load reaction at the bearing (kips)
X	=	Distance between the fixed pier and the neutral point (ft)

The temperature force on a single fixed pier in a bridge is the resultant of the unbalanced forces acting on the substructure units. Maximum friction coefficients are assumed for expansion bearings on one side of the pier and minimum coefficients are assumed on the other side to produce the greatest unbalanced force on the fixed pier. For bridges with only one pier (fixed), do not include temperature force, TU, in the design of the pier when the abutments are either fixed or semi-expansion.

The temperature changes in superstructure length are assumed to be along the longitudinal axis of the superstructure regardless of the substructure skew angle. This assumption is more valid for steel structures than for concrete structures.

The force on a column with a fixed bearing due to a temperature change in length of the superstructure is:

$$F = \frac{3EI\alpha TL}{144h^3}$$

Where:

L	=	Superstructure expansion length between neutral point and location being considered (ft)
F	=	Force per column applied at the bearing elevation (kips)

This force shall be resolved into components along both the longitudinal and transverse axes of the pier.

The values for computing temperature forces in Table 13.4-1 shall be used on Wisconsin bridges. Do not confuse this temperature change with the temperature range used for expansion joint design.

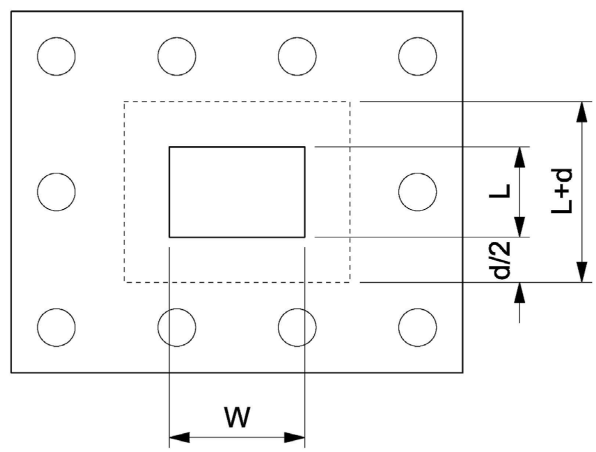


Figure 13.11-3
Critical Perimeter Location for Pile Footings

If the center of a pile falls on a line, then one-half of the pile force is assumed to act on each side of the line.

b. One-way action

The summation of the pile forces located within the area enclosed by the footing edges and a line at distance "d" from the face of the column determines the shear force, as illustrated in Figure 13.11-2. The shear resistance is influenced by the concrete strength, the footing depth and the entire footing width or length. If the center of a pile falls on a line, then one-half of the pile force is assumed to act on each side of the line.

10. The weight of the footing and soil above the areas is used to reduce the shear force.

11. The bottom layer of reinforcing steel is placed directly on top of the piles.

13.11.4 Continuous Footings

Continuous footings are used in pier frames of two or more columns when the use of isolated footings would result in a distance of less than 4'-6" between edges of adjacent footings. They are designed for the moments and shears produced by the frame action of the pier and the soil pressure under the footing.

The soil pressure or pile load under the footing is assumed to be uniform. The soil pressures or pile loads are generally computed only from the vertical column loads along with the soil and footing dead load. The moments at the base of the column are ignored for soil or pile loads.

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 Facilities Development Manual (FDM) Chapter 10 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.

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and do not require an R number. Refer to Facilities Development Manual (FDM) 11-55-5.2 for more information.

- Modular block gravity walls having a maximum height of less than 4.0 ft. measured from the bottom of wall or top of leveling pad to top of wall are deemed to be “minor retaining walls” and do not require an R number. Refer to FDM 11-55-5.2 for more information.

- Non-proprietary walls (e.g., sheet pile walls):

  - Walls having an exposed height of less than 5.5 ft. measured from the plan ground line to top of wall may require an R number based on specific project features. Designer to contact the Bureau of Structures region liaison for more information.

Cast-in-place walls being utilized strictly as bridge abutment or box culvert wings do not require R numbers as they are considered part of the structure.

14.2 Wall Types

Retaining walls can be divided into many categories as discussed below.

Conventional Walls

Retaining walls can be divided into gravity, semi-gravity, and non-gravity cantilever or anchored walls. A brief description of these walls is presented in 14.2.1 and 14.2.2 respectively.

Miscellaneous types of walls including multi-tiered walls, and hybrid or composite walls are also used by combining the wall types mentioned in the previous paragraph. These walls are used only under special project requirements. These walls are briefly discussed in 14.2.3, but the design requirements of these walls will not be presented in this chapter. In addition, some walls are also used for temporary shoring and discussed briefly in 14.2.4.

Permanent or Temporary Walls

All walls can be divided into permanent or temporary walls, depending on project application. Permanent walls have a typical designed life of 75 years. The temporary walls are designed for a service life of 3 years, or the intended project duration, whichever is greater. Temporary wall systems have less restrictive requirements for construction, material and aesthetics.

Fill Walls or Cut Walls

A retaining wall can also be classified as a fill wall, or a cut wall. This description is based on the nature of the earthwork required to construct the wall. If the roadway cross-sections (which include the wall) indicate that existing earth/soil must be removed (excavated) to install the wall, it is considered a 'cut' wall. If the roadway cross-sections indicate that earth fill will be placed behind the wall, with little excavation, the wall is considered a 'fill' wall. Sometimes wall construction requires nearly equal combinations of earth excavation and earth fill, leading to the nomenclature of a 'cut/fill' wall.

Bottom-up or Top-down Constructed Walls

This wall classification method refers to the method in which a wall is constructed. If a wall is constructed from the bottom of the wall, upward to the top, it is considered a bottom-up type of wall. Examples of this include CIP cantilever, MSE and modular block walls. Bottom-up walls are generally the most cost effective type. If a wall is constructed downward, from the top of the wall to the bottom, it is considered a top-down type of wall. This generally requires the insertion of some type of wall support member below the existing ground, and then excavation in front of the wall to the bottom of the exposed face. Examples of this include soil nail, soldier pile, cantilever sheet pile and anchored sheet pile walls. These walls are generally used when excavation room is limited.

ranges. Surface conditions must also be considered. For instance, if soft compressible soils are present, walls that can tolerate larger settlements and movements must be considered. MSE walls are generally more economical for fill locations than CIP cantilever walls.

Cut/fill Walls

CIP cantilever and prefabricated modular walls are most suitable in cut/fill situations as the walls are built from bottom up, have narrower base widths and these walls do not rely on soil reinforcement techniques to provide stability. These types of walls are suitable for both cut or fill situations.

14.3.1.3 Site Characteristics

Site characterization should be performed, as appropriate, to provide the necessary information for the design and construction of retaining wall systems. The objective of this characterization is to determine composition and subsurface soil/rock conditions, define engineering properties of foundation material and retained soils, establish groundwater conditions, determine the corrosion potential of the water, and identify any discontinuities or geotechnical issues such as poor bearing capacity, large settlement potential, and/or any other design and construction problems.

Site characterization mainly includes subsurface investigations and analyses. WisDOT's Geotechnical Engineering Unit generally completes the investigation and analyses for all in-house wall design work.

14.3.1.4 Miscellaneous Design Considerations

Other key factors that may influence wall selection include height limitations for specific systems, limit of wall radius on horizontal alignment, and whether the wall is a component of an abutment.

Foundation conditions that may govern the wall selection are bearing capacity, allowable lateral and vertical movements, tolerable settlement and differential movement of retaining wall systems being designed, susceptibility to scour or undermining due to seepage, and long-term maintenance.

14.3.1.5 Right of Way Considerations

Availability of ROW at a site may influence the selection of wall type. When a very narrow ROW is available, a sheet pile wall may be suitable to support an excavation. In other cases, when walls with tiebacks or soil reinforcement are considered, a relatively large ROW may be required to meet wall requirements. Availability of vertical operating space may influence wall selection where piling installation is required and there is not enough room to operate driving equipment.

FDM 11-55-5.4 describes the ROW requirement for retaining walls. It requires that all segments of a retaining wall should be under the control of WisDOT. No improvements or utility construction should be allowed in the ROW area of the retaining wall systems.

14.3.1.6 Utilities and Other Conflicts

Feasibility of some wall systems may be influenced by the presence of utilities and buried structures. MSE, soil nailing and anchored walls commonly have conflict with the presence of utilities or buried underground structures. MSE walls should not be used where utilities must stay in the reinforcement zone.

14.3.1.7 Aesthetics

In addition to being functional and economical, the walls should be aesthetically pleasing. Wall aesthetics may influence selection of a particular wall system. However, the aesthetic treatment should complement the retaining wall and not disrupt the functionality or selection of wall type. All permanent walls should be designed with due considerations to the wall aesthetics. Each wall site must be investigated individually for aesthetic needs. Temporary walls should generally be designed with little consideration to aesthetics. Chapter 4 - Aesthetics presents structures aesthetic requirements.

14.3.1.8 Constructability Considerations

Availability of construction materials, site accessibility, equipment availability, form work and temporary shoring, dewatering requirements, labor considerations, complicated alignment changes, scheduling consideration, speed of construction, construction staging/phasing and maintaining traffic during construction are some of the important key factors when evaluating the constructability of each wall system for a specific project site.

In addition, it should also be ensured that the temporary excavation slopes used for wall construction are stable as per site conditions and meet all safety requirements laid by Occupation and Safety Health Administration (OSHA).

14.3.1.9 Environmental Considerations

Selection of a retaining wall system is influenced by its potential environmental impact during and after construction. Some of the environmental concerns during construction may include excavation and disposal of contaminated material at the project site, large quantity of water, corrosive nature of soil/water, vibration impacts, noise abatement and pile driving constraints.

14.3.1.10 Cost

Cost of a retaining wall system is influenced by many factors that must be considered while estimating preliminary costs. The components that influence cost include excavation, structure, procurement of additional easement or ROW, drainage, disposal of unsuitable material, traffic maintenance etc. Maintenance cost also affects overall cost of a retaining wall system. The retaining walls that have least structural cost may not be the most economical walls. Wall selection should be based on overall cost. When feasible, MSE Walls and modular block gravity walls generally cost less than other wall types.

assuming a factor of 1.0 for nominal loads, a resistance factor of 1.0 for nominal strengths and elastic analyses.

Extreme Event II limit state is evaluated to design walls for vehicular collision forces. In particular, MSE walls having a traffic barrier at the top are vulnerable to damage due to vehicle collision forces and this case for MSE Walls is discussed further in 14.6.3.10.

14.4.5.3 Design Loads

Retaining walls shall be designed to withstand all applicable loads generally categorized as permanent and transient loads.

Permanent loads include dead load DC due to weight of the structural components and non structural components of the wall, dead load DW loads due to wearing surfaces and utilities, vertical earth pressure EV due to dead load of earth, horizontal earth pressure EH and earth surcharge loads ES. Applied earth pressure and earth pressure surcharge loads are further discussed in 14.4.5.4.

The transient loads include, but are not limited to, water pressure WA, live load surcharge LS, and forces caused by the deformations due to shrinkage SH, creep CR and settlement caused by the foundation SE.

These loads should be computed in accordance with **LRFD [3.4]** and **LRFD [11].** Only loads applicable for each specific wall type should be considered in the engineering analyses.

14.4.5.4 Earth Pressure

Determination of earth pressure will depend upon types of wall structure (gravity, semi gravity, reinforced earth wall, cantilever or anchored walls, etc.), wall movement, wall geometry, wall friction, configuration, retained soil type, ground water conditions, earth surcharge, and traffic and construction related live load surcharge. In general, earth pressure on retaining walls shall be calculated in accordance with **LRFD [3.11.5]**. Earth pressure that will develop on walls includes active, passive or at-rest earth pressure.

Active Earth Pressure

The active earth pressure condition exists when a retaining wall is free to rotate away from the retained backfill. There are two earth pressure theories available for determining the active earth pressure coefficient (K_a); Rankine and Coulomb earth pressure theories. A detailed discussion of Rankine and Coulomb theories can be found in *Foundation Design- Principles and Practices*; by Donald P. Cudoto or *Foundation Analysis and Design*, 5th Edition by Joseph E. Bowles as well as other standard text books on this subject.

Rankine earth pressure makes assumptions that the retained soil has a horizontal surface, the failure surface is a plane and that the wall is smooth (i.e. no friction). Rankine earth pressure theory is the preferred method for developing the active earth pressure coefficient; however, where wall friction is an important consideration or where sloping surcharge loads are considered, Coulomb earth pressure theory may be used. The use of Rankine theory will cause a slight over estimation of $K_{a,}$ therefore, increasing the pressure on the wall resulting in a more conservative design.

Walls that are cast-in-place (CIP) semi gravity concrete cantilever referred, hereafter, as CIP cantilever, Mechanically Stabilized Earth (MSE), modular block gravity, soil nailing, soldier-pile and sheet-pile walls are typically considered flexible enough to justify using an active earth pressure coefficient.

For walls using Coulomb earth pressure theory:

$$K_a = \frac{\sin^2(\theta+\phi_f')}{\Gamma[\sin^2\sin(\theta-\delta)]}$$ **LRFD [Eq'n 3.11.5.3-1]**

Where:

Γ = $\left[1+\sqrt{\frac{\sin(\phi_f'+\delta)\sin(\phi_f'-B)}{\sin(\theta-\delta)\sin(\theta+B)}}\right]^2$

δ = Friction angle between fill and wall (degrees)

B = Angle of fill to the horizontal (degrees)

θ = Angle of back face of wall to the horizontal (degrees)

ϕ_f' = Effective angle of internal friction (degrees)

Note: refer to Figure 14.4-1 for details.

For walls using Rankine earth pressure theory:

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

At-Rest Earth Pressure

In the at-rest earth pressure (K_o) condition, the top of the wall is not allowed to deflect or rotate; therefore, requiring the wall to support the full pressure of the soil behind the wall.

The at-rest earth pressure coefficient shall be used to calculate the lateral earth pressure for non-yielding retaining walls restrained from rotation and/or lateral translation in accordance with **LRFD [3.11.5.2]**. Non-yielding walls include integral abutment walls, or retaining walls resting on bedrock or pile foundation.

For walls (normally consolidated soils, vertical wall, and level ground) using at-rest earth pressure:

$$K_o = 1-\sin\phi_f'$$ **LRFD [Eq'n 3.11.5.2-1]**

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and/or bicycle traffic, a live load of 90 psf is used. Consideration should also be given to maintenance vehicle loads as specified in Chapter 37 – Pedestrian Bridges.

17.2.5 Load Factors

The load factor, γ_i, is used to adjust force effects on a structural element. This factor accounts for variability of loads, lack of accuracy in analysis and the probability of simultaneous occurrence of different loads.

For the design limit states, the values of γ_i for different types of loads are found in **LRFD [Tables 3.4.1-1 and 3.4.1-2]**. Load factors most commonly used for superstructure design are also presented in Table 17.2-5.

Load Combination	Load Factor, γi				
	DC		DW		LL+IM
	Maximum	Minimum	Maximum	Minimum	
Strength I	1.25	0.90	1.50	0.65	1.75
Strength III	1.25	0.90	1.50	0.65	0.00
Strength V	1.25	0.90	1.50	0.65	1.35
Service I	1.00	1.00	1.00	1.00	1.00
Service II	1.00	1.00	1.00	1.00	1.30
Service III	1.00	1.00	1.00	1.00	0.80
Fatigue I	0.00	0.00	0.00	0.00	1.50
Extreme Event II	1.25	0.90	1.50	0.65	0.50

Table 17.2-5
Load Factors

The maximum and minimum values should be used to maximize the intended effect of the load. An example of the use of minimum load factors is the load factor for dead load when uplift is being checked.

17.2.6 Resistance Factors

The resistance factor, ϕ, is used to reduce the computed nominal resistance of a structural element. This factor accounts for variability of material properties, structural dimensions and workmanship, and uncertainty in prediction of resistance.

Resistance factors are presented in **LRFD [1.3.2.1]**, **LRFD [5.5.4.2]**, **LRFD [6.5.4.2]**, **LRFD [6.5.5]** and **LRFD [6.10.1.7]**. The most commonly used resistance factors for superstructure design are also presented in Table 17.2-6.

Limit State	Material	Application	Resistance Factor, ϕ
Strength	Concrete	Flexure (reinforced concrete)	0.90
		Flexure (prestressed concrete)	1.00
		Shear (normal weight)	0.90
		Shear (lightweight)	0.90
	Steel	Flexure	1.00
		Shear	1.00
		Axial compression, steel only	0.90
		Axial compression, composite	0.90
		Tension, fracture in net section	0.80
		Tension, yielding in gross section	0.95
		Bolts bearing on material	0.80
		Shear connectors	0.85
		A325 and A490 bolts in tension	0.80
		A325 and A490 bolts in shear	0.80
		A307 bolts in tension	0.80
		A307 bolts in shear	0.65
		Block shear	0.80
		Web crippling	0.80
		Welds	See **LRFD [6.5.4.2]**
Service	All	All	1.0
Fatigue	All	All	1.0
Extreme Event	All	All	1.0

Table 17.2-6
Resistance Factors

17.2.7 Distribution of Loads for Slab Structures

For slab structures, the distribution of loads is based on strip widths, as illustrated in Figure 17.2-6 through Figure 17.2-11. Figure 17.2-6 and Figure 17.2-7 illustrate the distribution of loads for slab structures with no sidewalks. Figure 17.2-8 and Figure 17.2-9 illustrate the distribution of loads for slab structures with sidewalks. Figure 17.2-10 and Figure 17.2-11 illustrate the distribution of loads for slab structures with raised sidewalks. It should be noted that, although medians are not shown in these figures, medians are treated similar to other superimposed dead loads.

17.9 Bridge Approaches

The structure approach slab, or approach pavement, is part of the roadway design plans. Structure approach standards are provided in the Facilities Development Manual (FDM).

Guidance for the selection of pavement types for bridge approaches is as shown in FDM 14-10-15.

Considerations for site materials, drainage and backfill are provided in Chapter 12 – Abutments. Most approach pavement failures are related to settlement of embankment or foundation materials. Past experience shows that significant settlement is most likely to occur where marginal materials are used. Designers are encouraged to provide perforated underdrains wrapped in geotextile fabric placed in a trench filled with crushed stone. Also, abutment backfill material should be granular in nature and consolidated under optimum moisture conditions.

17.10 Design of Precast Prestressed Concrete Deck Panels

17.10.1 General

An advantage of stay-in-place forms is that they can be placed in less time than it takes to place the forms for a conventional deck. There is also a labor savings because the extra step of removing deck forms is not required. Stay-in-place forms are often the preferred system for shallow box girders because of the difficulty of removing forms in a confined space.

When determined ideal for a project, precast concrete deck panels should be detailed in the contract documents. The contractor is responsible for the shop plans of the panels and any other changes that may be required to the reinforcing steel in the cast-in-place portion of the deck. Contract documents should also include an option for the contractor to use a conventional deck. Contact the Bureau of Structures Design Section for current precast concrete deck panel Special Provisions and for other considerations.

When a conventional deck is detailed in the contract documents and the contractor is interested in utilizing precast deck panels, the department may consider their usage on a project-specific basis. The contractor would be responsible for the shop plans of the panels and any other changes that may be required to the reinforcing steel in the cast-in-place portion of the deck. Payment to a contractor who chooses to use stay-in-place forms is based on the contract prices bid for the conventional cast-in-place deck.

Deck panels are only used between the inside faces of the exterior girders. The overhangs outside the exterior girders are formed and the concrete placed in the same way as in a conventional cast-in-place deck. On skewed decks, the contractor may form and cast the skewed portion of the deck full depth or they may use skewed end deck panels which may be individually precast or saw-cut from square end planks.

One potential issue with decks formed using concrete deck panels is that cracks often form in the cast-in-place concrete over the transverse joints between panels and along the edges of the panels parallel to the girders. Reflection cracking is less of a problem when these panels are used on prestressed concrete girders than on steel girders. Simple-span prestressed concrete girder bridges have less reflective cracking than continuous-span prestressed concrete girder bridges.

17.10.2 Deck Panel Design

The design of precast prestressed concrete deck panels shown in Table 17.10-1 is based on *AASHTO LRFD* design criteria. These panels were designed for flexure due to the HL-93 design truck live load, dead load of the plastic concrete supported by the panels, a construction load of 50 psf, dead load of the panels and a future wearing surface of 20 psf. The live load moments were obtained from **LRFD [Table A4-1]**.

At the request of precast deck panel fabricators, only two strand sizes are used – 3/8 inch and 1/2 inch. Strand spacing is given in multiples of 2 inches.

WisDOT exception to AASHTO:

A 3-inch minimum panel thickness is used, even though **LRFD [9.7.4.3.1]** specifies a minimum thickness of 3.5 inches.

The decision to use a 3-inch minimum panel was based on the successful use of 3-inch panels by other agencies over many years. In addition, a minimum of 5 inches of cast-in-place concrete is preferred for crack control and reinforcing steel placement. A 3.5-inch panel thickness would require an 8.5-inch deck, which would not allow direct substitution of panels for a traditionally designed 8-inch deck.

A study performed at Iowa State University determined that a 3-inch thick panel with coated 3/8-inch strands at midthickness spaced at 6 inches, along with epoxy-coated 6 x 6 – W2.9 x W2.9 welded wire fabric, was adequate to prevent concrete splitting during strand detensioning. The use of #3 bars placed perpendicular to the strands at 9" spacing also prevents concrete splitting.

Panel thicknesses were increased by ½ -inch whenever a strand spacing of less than 6 inches was required. Strands with a ½-inch diameter were used in panels 3½ inches thick or greater when 3/8-inch strands spaced at 6 inches were not sufficient.

The allowable tensile stress in the panels, as presented in **LRFD [Table 5.9.2.3.2b-1]**, is as follows:

$0.0948\lambda\sqrt{f'_c} \leq 0.3$ ksi ; where λ = conc. density modification factor **LRFD [5.4.2.8]**, and has a value of 1.0 for normal weight conc.

This allowable tensile stress limit is based on f'_c in units of ksi and is for components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions.

The transfer length of the strands is assumed to be 60 strand diameters at a stress of 202.5 ksi. The development length, L_d, of the strands, as presented in **LRFD [5.9.4.3.2]**, is assumed to be as follows:

$$L_d = k\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b$$

Where:

k	=	1.0 for pretensioned members with a depth less than 24 inches
d_b	=	Nominal strand diameter (inches)
f_{ps}	=	Average stress in prestressing steel at the time when the nominal resistance of the member is required (ksi)

f_{pe} = Effective stress in prestressing steel after losses (ksi)

L_d = Development length beyond critical section (inches)

The minimum panel width is the length required for the panel to extend 4” onto the top flange as shown in Table 17.10-1. A linear reduction in f_{pe} is required if the panel width is less than two times the development length. The values shown in Table 17.10-1 consider this linear reduction.

The designs in Table 17.10-1 are based on uncoated prestressing strands. Grit-impregnated, epoxy-coated strands cost four times as much as uncoated strands but require about half the transfer and development length as uncoated strands. A cover of 1 1/4 inches is adequate to provide protection from chlorides for uncoated strands using a 5 ksi concrete mix. However, for bridges with high traffic volume, a 6 ksi mix is recommended.

LRFD [9.7.4.3.2] specifies that the strands need not extend beyond the panels into the cast-in-place concrete above the beams. This simplifies construction of the panels at the plant since they can be saw cut to the required length. Installation in the field is also simplified because extended strands may interfere with girder shear connectors. As a substitute for the strands that don’t extend out of the panels, #4 bars spaced at twice the spacing of the transverse bars are placed on top of the panels over the girders in the cast-in-place concrete. These bars anchor the panels together to prevent or reduce longitudinal cracking over the ends of the panels and also resist any positive continuity moments that may develop. Also by not extending the strands into the cast-in-place concrete, the uncoated strands are not exposed to chlorides that may seep through cracks that may develop in the cast-in-place concrete.

LRFD [5.6.3.3] requires that the moment capacity of a flexural member be greater than the cracking moment based on the modulus of rupture. This requirement may be waived if the moment capacity is greater than 1.33 times the factored design moment. The purpose of this requirement is to provide a minimum amount of reinforcement in a flexural member so that a flexural failure will not be sudden or occur without warning. Tests have shown that for slabs on girders, the failure mode is a punching shear failure and not a flexural failure. ACI 10.5.4 also recognizes the difference between slabs and beams and does not require the same minimum reinforcement for slabs. For these reasons, **LRFD [5.6.3.3]** was not considered in the designs of the panels shown in Table 17.10-1. However, panels with a width of 6 feet or more meet the requirements of **LRFD [5.6.3.3]**.

17.10.3 Transverse Reinforcement for Cast-in-Place Concrete on Deck Panels

The design of the transverse reinforcing steel in the cast-in-place concrete placed on deck panels is based on *AASHTO LRFD*. The live load moments used to determine the size and spacing of the transverse reinforcing bars placed in the top of the cast-in-place concrete are from **LRFD [Table A4-1]**. The reinforcing steel in the cast-in-place concrete is also designed for a future wearing surface of 20 psf. With stay-in-place forms, there are no negative moments from the dead load of the cast-in-place concrete. The required reinforcing steel shown in Table 17.10-2 is based on both the strength requirement and crack control requirement.

Crack control was checked in accordance with **LRFD [5.6.7]** and as shown in 17.5.3.1. A concrete strength of 4 ksi was assumed, and the haunch height over the girders was not considered.

The distance from the centerline of the girder to the design section is from **LRFD [4.6.2.1.6]**. For prestressed concrete girders, use the values in Figure 17.5-1.

The reinforcing steel in Table 17.10-2 does not account for deck overhangs. However, Table 17.6-2, Table 17.6-3, Table 17.6-4 and Table 17.6-5 provide the minimum reinforcing steel required in the overhangs. Also for any portion of a deck not supported by deck panels, use Table 17.5-1 for determining the required reinforcing steel.

17.10.3.1 Longitudinal Reinforcement

For continuous prestressed concrete girders, the longitudinal reinforcing steel over the piers is the same as that required for a conventional deck. For steel girders, see 17.5.3.2 for longitudinal continuity reinforcement.

17.10.4 Details

Precast deck panels should extend a minimum of 1.5 inches beyond the face of concrete diaphragms at the substructure units. The transverse joints between panels in adjacent bays should be staggered, preferably a distance about 1/2 panel length. Staggering the joints helps to minimize transverse reflective cracking.

Panels should never rest directly on a girder flange. According to **LRFD [9.7.4.3.4]**, "The ends of the formwork panels shall be supported on a continuous mortar bed or shall be supported during construction in such a manner that the cast-in-place concrete flows into the space between the panel and the supporting component to form a concrete bedding." The minimum width of bearing on the flange of a girder for grout support is 3 inches. See Figure 17.10-1 and Figure 17.10-2 for additional information.

High-density expanded polystyrene is used to support the panels prior to the placement of the grout under the panel. The polystyrene is cut to the required haunch height so a constant slab thickness is maintained. Fiber board or sheathing panel supports are not allowed because the slight deflection of polystyrene compresses the concrete underneath the panel and results in less reflective longitudinal cracking along the panel edge.

The main function of the polystyrene is to form the haunch height and to form a dam for the grout placement. The grout must be placed before placement of the deck concrete.

Some agencies specify a maximum haunch height. When it is exceeded, they allow the contractor to thicken the slab. Wisconsin does not specify a maximum haunch height and leaves that decision to the designer, who is better informed to make that decision based on the specific situation of their project.

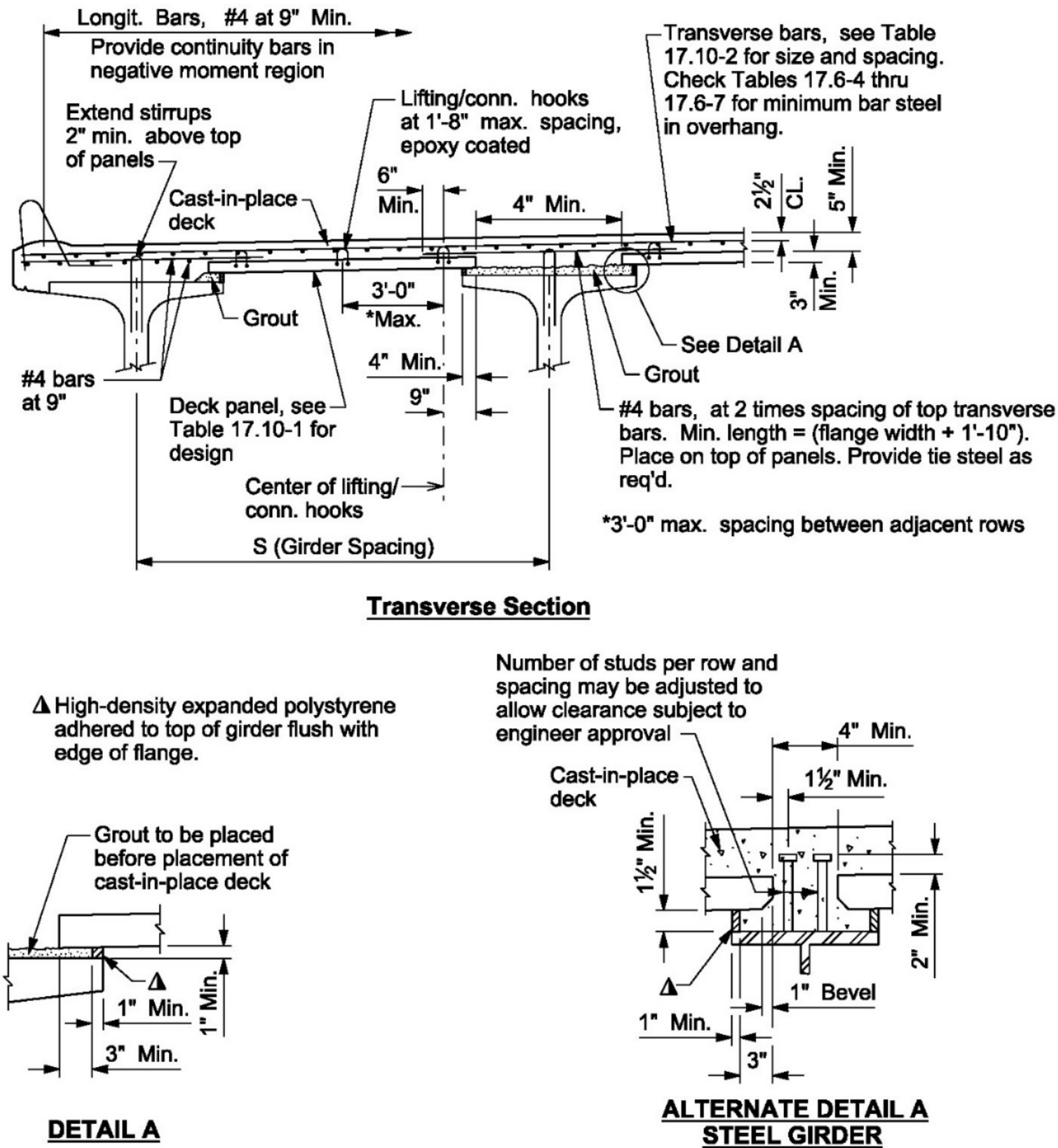
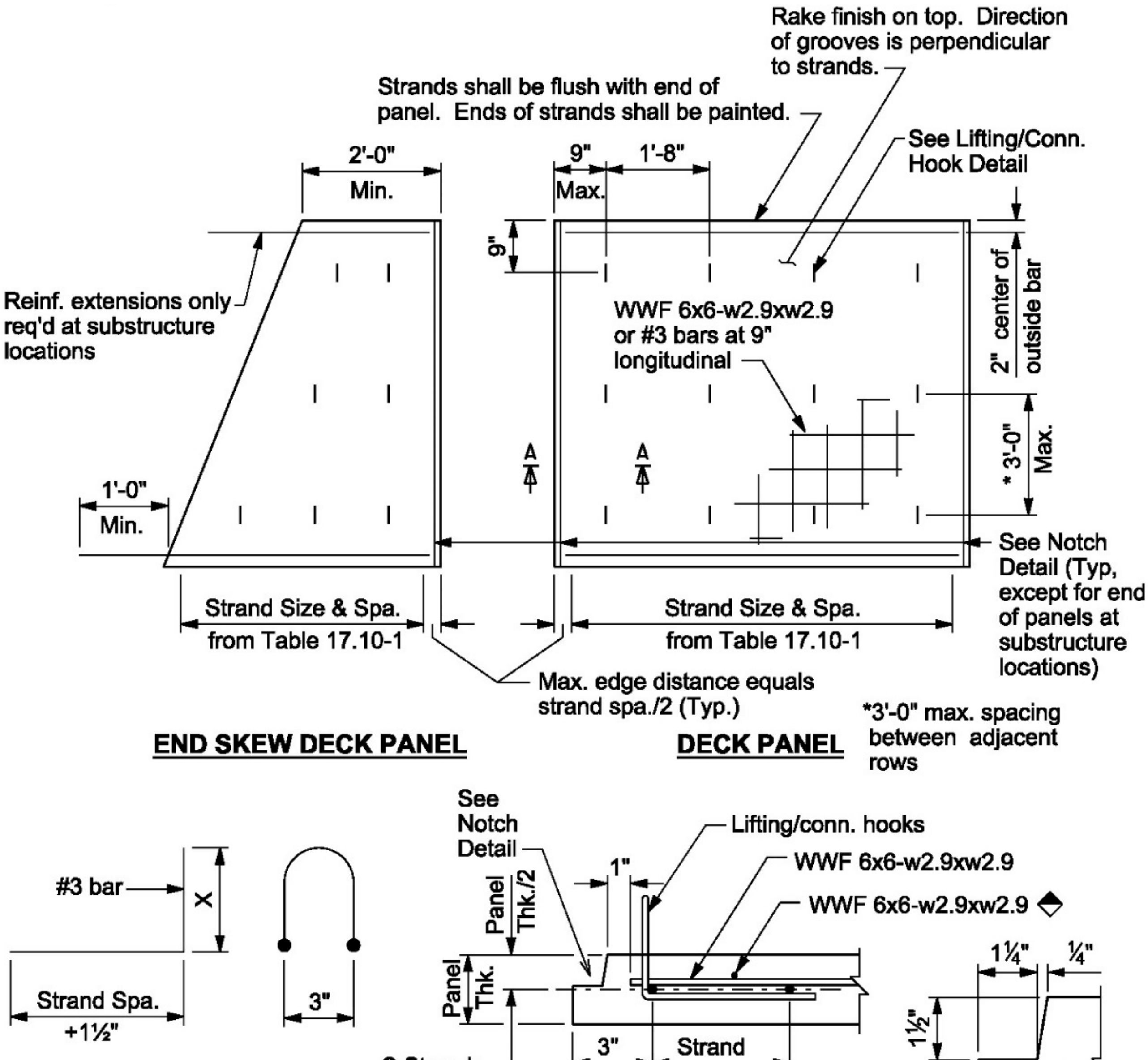


Figure 17.10-1
Transverse Section through Slab on Girders with Deck Panel and Details



Panel Thk. (in.)	X (in.)
3, 3½	4½
4	5
4½ , 5, 5½	5½

Lifting/Conn. Hook Detail

Figure 17.10-2
Deck Panel Details

Girder Spacing "S"	Panel Thick. (Inches)	Total Slab Thick. (Inches)	Top Flange Width (Inches)												
			12		16		18		24		30		48		
			Strand		Strand		Strand		Strand		Strand		Strand		
			Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	
4'-6"	3	8	10	13.17	10	12.33	10	11.92	10	11.08	10	11.08	10	11.08	3/8" Diameter Strands
4'-9"	3	8	10	13.58	10	12.75	10	12.75	10	11.50	10	11.08	10	11.08	
5'-0"	3	8	10	14.42	10	13.58	10	13.17	10	12.33	10	11.08	10	11.08	
5'-3"	3	8	10	14.83	10	14.00	10	13.58	10	12.75	10	11.92	10	11.08	
5'-6"	3	8	10	15.67	10	14.83	10	14.42	10	13.17	10	12.33	10	11.08	
5'-9"	3	8	10	16.50	10	15.67	10	15.25	10	14.00	10	13.17	10	11.50	
6'-0"	3	8	8	14.25	10	16.50	10	16.08	10	14.83	10	13.58	10	11.50	
6'-3"	3	8	8	15.45	8	14.25	10	16.92	10	15.67	10	14.42	10	11.92	
6'-6"	3	8	8	16.12	8	15.45	8	14.78	10	16.50	10	15.25	10	12.33	
6'-9"	3	8	8	17.12	8	16.12	8	15.78	8	14.25	10	16.08	10	13.17	
7'-0"	3	8	6	14.19	8	17.12	8	16.45	8	15.45	8	14.25	10	13.58	
7'-3"	3	8	6	14.94	6	14.19	6	13.62	8	16.12	8	15.12	10	14.42	
7'-6"	3	8	6	15.69	6	14.94	6	14.69	8	17.12	8	15.78	10	15.25	
7'-9"	3	8	6	16.44	6	15.69	6	15.44	6	14.44	8	16.78	10	16.50	
8'-0"	3	8	6	17.19	6	16.44	6	16.19	6	15.19	6	14.19	8	14.25	
8'-3"	3.5	8.5	6	16.76	6	16.01	6	15.76	6	14.76	6	13.47	8	14.14	
8'-6"	3.5	8.5	10	29.48	6	16.76	6	16.51	6	15.51	6	14.51	8	14.97	
8'-9"	3.5	8.5	8	26.44	10	30.06	10	29.06	6	16.26	6	15.26	8	15.97	
9'-0"	3.5	8.5	8	27.44	8	26.44	8	26.10	6	17.01	6	16.01	8	16.64	
9'-3"	3.5	8.5	8	28.77	8	27.77	8	27.10	10	30.06	6	16.76	6	14.01	
9'-6"	4	9	8	27.76	8	26.76	8	25.95	10	29.22	6	16.37	8	17.20	
9'-9"	4	9	8	29.09	8	27.76	8	27.43	10	30.62	6	17.12	6	14.37	
10'-0"	4	9	8	30.09	8	29.09	8	28.43	8	27.09	10	30.20	6	15.12	
10'-3"	4	9	6	25.48	8	30.09	8	29.76	8	28.09	8	26.76	6	15.87	
10'-6"	4	9	6	26.23	6	25.48	8	30.76	8	29.09	8	27.76	6	16.62	
10'-9"	4	9.5	6	26.73	6	25.73	6	25.23	8	29.43	8	27.76	6	16.12	
11'-0"	4	9.5	6	27.48	6	26.73	6	26.23	8	30.43	8	28.76	6	16.87	
11'-3"	4	9.5	6	28.48	6	27.48	6	26.98	6	25.73	8	30.09	10	30.20	1/2" Diameter Strands
11'-6"	4	9.5	6	29.48	6	28.48	6	27.98	6	26.73	6	25.23	8	25.95	
11'-9"	4	10	6	30.23	6	28.98	6	28.48	6	26.98	6	25.48	8	25.95	
12'-0"	4.5	10	6	29.62	6	28.62	6	28.12	6	26.62	6	25.37	8	26.50	
12'-3"	4.5	10	6	30.62	6	29.62	6	29.12	6	27.62	6	26.12	8	27.83	
12'-6"	5	10	6	30.34	6	29.34	6	28.84	6	27.59	6	26.34	8	28.28	
12'-9"	5	10.5	6	30.59	6	29.59	6	29.09	6	27.59	6	26.34	8	27.95	
13'-0"	5.5	10.5	6	30.36	6	29.36	6	29.11	6	27.61	6	26.36	8	28.77	
13'-3"	5.5	10.5	4	23.52	6	30.36	6	29.86	6	28.61	6	27.36	8	29.77	
13'-6"	5.5	10.5	4	24.18	4	23.52	4	23.18	6	29.36	6	28.11	8	30.77	

In regions where stress reversal takes place, continuous concrete slabs will be doubly reinforced. At these locations, the full stress range in the reinforcing bars from tension to compression is considered.

In regions of compressive stress due to unfactored permanent loads, fatigue shall be considered only if this compressive stress is less than 1.75 times the maximum tensile live load stress from the fatigue truck. The section properties for fatigue investigations shall be based on cracked sections where the sum of stresses, due to unfactored permanent loads, and 1.75 times the fatigue load is tensile and exceeds $0.095\ (f'_c)^{1/2}$.

The factored stress range, Q, shall be calculated using factored loads described in 18.3.5.1. The factored resistance, R_r , shall be calculated as in 18.3.5.2.1.

Then check that, Q (factored stress range) $\leq R_r$ is satisfied.

Reference is made to the design example in 18.5 of this chapter for computations relating to reinforcement remaining in tension throughout the fatigue cycle, or going through tensile and compressive stresses during the fatigue cycle.

18.4.6.3 Check for Crack Control

Service Limit State considerations and assumptions are detailed in **LRFD [5.5.2, 5.6.1, 5.6.7]**.

The area of longitudinal slab reinforcement, A_s , should be checked for crack control at locations where maximum tensile stress occurs along the structure, and for haunched slab structures, checked at the haunch/slab intercepts. The area should also be checked for crack control at bar reinforcement cutoff locations using Service I Limit State. Check the reinforcement in an interior and exterior strip (edge beam).

The use of high-strength steels and the acceptance of design methods where the reinforcement is stressed to higher proportions of the yield strength, makes control of flexural cracking by proper reinforcing details more significant than in the past. The width of flexural cracks is proportional to the level of steel tensile stress, thickness of concrete cover over the bars, and spacing of reinforcement. Improved crack control is obtained when the steel reinforcement is well distributed over the zone of maximum concrete tension.

Crack control criteria shall be applied when the tension in the cross-section exceeds 80% of the modulus of rupture, f_r , specified in **LRFD [5.4.2.6]**, for Service I Limit State. The spacing of reinforcement, s, in the layer closest to the tension face shall satisfy:

$$s \leq (700\ \gamma_e / \beta_s f_{ss}) - 2\ (d_c) \quad \text{(in)}$$

LRFD [5.6.7]

in which:

$$\beta_s = 1 + (d_c) / 0.7\ (h - d_c)$$

Where:

γ_e = 1.00 for Class 1 exposure condition (bottom reinforcement)

γ_e = 0.75 for Class 2 exposure condition (top reinforcement)

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto, (in). For top reinforcement, d_c , should not include the ½" wearing surface

f_{ss} = tensile stress in steel reinforcement (ksi) $\leq 0.6f_y$; use factored loads described in 18.3.4.1 at the Service I Limit State, to calculate (f_{ss})

h = overall depth of the section (in)

18.4.6.4 Minimum Reinforcement Check

The area of longitudinal slab reinforcement, A_s , should be checked for minimum reinforcement requirement at locations along the structure **LRFD [5.6.3.3]**.

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r , or moment capacity, at least equal to the lesser of:

M_{cr} (or) 1.33 M_u

$M_{cr} = \gamma_3 (\gamma_1 f_r) S = 1.1 f_r (I_g / c)$; $S = I_g / c$

Where:

f_r = $0.24 \lambda (f'c)^{1/2}$ modulus of rupture (ksi) **LRFD [5.4.2.6]**

γ_1 = 1.6 flexural cracking variability factor

γ_3 = 0.67 ratio of minimum yield strength to ultimate tensile strength; for A615 Grade 60 reinforcement

I_g = gross moment of Inertia (in^4)

c = effective slab thickness/2 (in)

M_u = total factored moment, calculated using factored loads described in 18.3.3.1 for Strength I Limit State

λ = concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

Select lowest value of [M_{cr} (or) 1.33 M_u] = M_L

The factored resistance, M_r , or moment capacity, shall be calculated as in 18.3.3.2.1.

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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}	= 0.80 to 0.85 $f'_c \leq 6.8$ ksi
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.

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 [5.9.2.2]** and **LRFD [5.9.2.3]**.

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 [5.9.3] 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 [5.9.3.3]** 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 [5.9.3.2.3a]**, 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:

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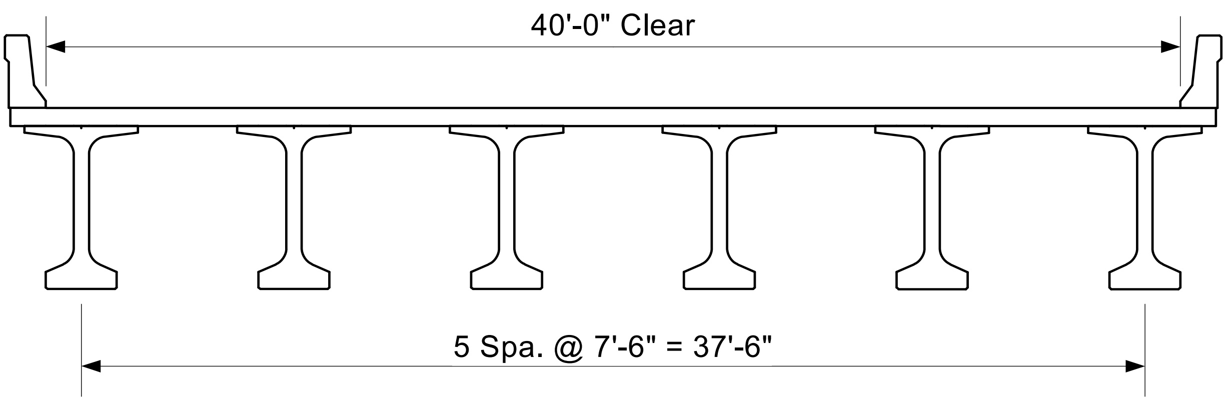
E19-1 Single Span Bridge, 72W" Prestressed Girders - LRFD

This example shows design calculations for a single span prestressed gider bridge. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Eighth Edition - 2017)***

E19-1.1 Design Criteria



$L := 146$	center to center of bearing, ft
$L_g := 147$	total length of the girder (the girder extends 6 inches past the center of bearing at each abutment).
$w_b := 42.5$	out to out width of deck, ft
$w := 40$	clear width of deck, 2 lane road, 3 design lanes, ft
$f'_c := 8$	girder concrete strength, ksi
$f'_{ci} := 6.8$	girder initial concrete strength, ksi New limit for release strength.
$f'_{cd} := 4$	deck concrete strength, ksi
$f_{pu} := 270$	low relaxation strand, ksi
$d_b := 0.6$	strand diameter, inches
$A_{strand} := 0.217$	area of strand, in^2
$w_p := 0.387$	weight of Wisconsin Type LF parapet, klf
$t_s := 8$	slab thickness, in
$t_{se} := 7.5$	effective slab thickness, in
$skew := 20$	skew angle, degrees
$E_s := 28500$	ksi, Modulus of Elasticity of the Prestressing Strands
$w_c := 0.150$	kcf

E19-1.2 Modulus of Elasticity of Beam and Deck Material

Based on past experience, the modulus of elasticity for the precast and deck concrete are given in Chapter 19 as $E_{beam6} := 5500$ ksi and $E_{deck4} := 4125$ ksi for concrete strengths of 6 and 4 ksi respectively. The values of E for different concrete strengths are calculated as follows (ksi):

$$E_{beam8} := 5500 \cdot \frac{\sqrt{f'_c \cdot 1000}}{\sqrt{6000}} \qquad \boxed{E_{beam8} = 6351} \qquad E_B := E_{beam8}$$

$$E_D := E_{deck4}$$

$$n := \frac{E_B}{E_D} \qquad \boxed{n = 1.540}$$

Note that this value of E_B is used for strength, composite section property, and long term deflection (deck and live load) calculations.

The value of the modulus of elasticity at the time of release is calculated in accordance with **LRFD [C5.4.2.4]**. This value of E_{ct} is used for loss and instantaneous deflection (due to prestress and dead load of the girder) calculations.

$$E_{beam6.8} := 33000 \cdot w_c^{1.5} \cdot \sqrt{f'_{ci}} \qquad \boxed{E_{beam6.8} = 4999} \qquad E_{ct} := E_{beam6.8}$$

E19-1.3 Section Properties

72W Girder Properties:

$w_{tf} := 48$ in

$t_t := 5.5$ in

$t_w := 6.5$ in

$t_b := 13$ in

$ht := 72$ in

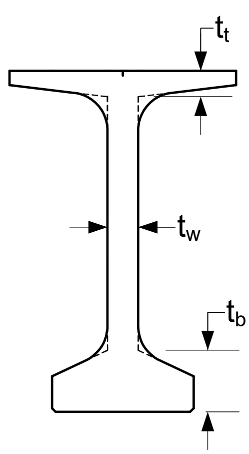
$b_w := 30$ width of bottom flange, in

$A_g := 915$ in^2

$r_{sq} := 717.5$ in^2

$I_g := 656426$ in^4

$y_t := 37.13$ in



$y_b := -34.87$ in

$S_t := 17680$ in^3

$S_b := -18825$ in^3

$$e_g := y_t + 2 + \frac{t_{se}}{2}$$ $\boxed{e_g = 42.88}$ in

Web Depth: $d_w := ht - t_t - t_b$ $\boxed{d_w = 53.50}$ in

$K_g := n \cdot \left(I_g + A_g \cdot e_g^{\,2}\right)$ **LRFD [Eq 4.6.2.2.1-1]** $\boxed{K_g = 3600866}$ in^4

E19-1.4 Girder Layout

Chapter 19 suggests that at a 146 foot span, the girder spacing should be 8'-6" with 72W girders.

$S := 8.5$ ft

Assume a minimum overhang of 2.5 feet (2 ft flange + 6" overhang), $s_{oh} := 2.5$

$$n_{spa} := \frac{w_b - 2 \cdot s_{oh}}{S}$$ $\boxed{n_{spa} = 4.412}$

Use the next lowest integer: $n_{spa} := \mathrm{ceil}\left(n_{spa}\right)$ $\boxed{n_{spa} = 5}$

Number of girders: $ng := n_{spa} + 1$ $\boxed{ng = 6}$

Overhang Length: $s_{oh} := \dfrac{w_b - S \cdot n_{spa}}{2}$ $\boxed{s_{oh} = 0.00}$ ft

Recalculate the girder spacing based on a minimum overhang, $s_{oh} := 2.5$

$$S := \frac{w_b - 2 \cdot s_{oh}}{n_{spa}}$$ $\boxed{S = 7.50}$ ft

E19-1.5 Loads

$w_g := 0.953$ weight of 72W girders, klf

$w_d := 0.100$ weight of 8-inch deck slab (interior), ksf

$w_h := 0.125$ weight of 2.5-in haunch, klf

$w_{di} := 0.460$ weight of diaphragms on interior girder (assume 2), kips

$w_{dx} := 0.230$ weight of diaphragms on exterior girder, kips

$w_{ws} := 0.020$ future wearing surface, ksf

$w_p = 0.387$ weight of parapet, klf

E19-1.5.1 Dead Loads

Dead load on non-composite (DC):

exterior:

$$w_{dlxi} := w_g + w_d \cdot \left(\frac{S}{2} + s_{oh}\right) + w_h + 2 \cdot \frac{w_{dx}}{L}$$ $\boxed{w_{dlxi} = 1.706}$ klf

interior:

$$w_{dlii} := w_g + w_d \cdot S + w_h + 2 \cdot \frac{w_{di}}{L}$$ $\boxed{w_{dlii} = 1.834}$ klf

* Dead load on composite (DC):

$$w_p := \frac{2 \cdot w_p}{ng}$$ $\boxed{w_p = 0.129}$ klf

* Wearing Surface (DW):

$$w_{ws} := \frac{w \cdot w_{ws}}{ng}$$ $\boxed{w_{ws} = 0.133}$ klf

* **LRFD [4.6.2.2.1]** states that permanent loads on the deck may be distributed uniformly among the beams. This method is used for the parapet and future wearing surface loads.

E19-1.5.2 Live Loads

For Strength 1 and Service 1 and 3:

HL-93 loading =	truck + lane	**LRFD [3.6.1.3.1]**
	tandem + lane	

DLA of 33% applied to truck or tandem, but not to lane per **LRFD [3.6.2.1].**

For Fatigue:

LRFD [5.5.3] states that fatigue of the reinforcement need not be checked for fully prestressed components designed to have extreme fiber tensile stress due to Service III Limit State within the tensile stress limit specified in **LRFD [Table 5.9.2.3.2b-1]**.

For fully prestressed components, the compressive stress due to the Fatigue I load combination and one half the sum of effective prestress and permanent loads shall not exceed 0.40 f'c after losses.

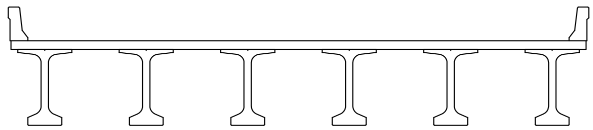
DLA of 15% applied to design truck with a 30 foot axle spacing.

For the Wisconsin Standard Permit Vehicle (Wis-250) Check:

The Wis-250 vehicle is to be checked during the design calculations to make sure it can carry a minimum vehicle weight of 190 kips. See Chapter 45 - Bridge Ratings for calculations.

E19-1.6 Load Distribution to Girders

In accordance with **LRFD [Table 4.6.2.2.1-1]**, this structure is a Type "K" bridge.



Distribution factors are in accordance with **LRFD [Table 4.6.2.2.2b-1]**. For an interior beam, the distribution factors are shown below:

For one Design Lane Loaded:

$$0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}{}^3}\right)^{0.1}$$

For Two or More Design Lanes Loaded:

$$0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}{}^3}\right)^{0.1}$$

Criteria for using distribution factors - Range of Applicability per **LRFD [Table 4.6.2.2.2b-1]**.

$$\text{DeckSpan} := \begin{cases} \text{"OK"} & \text{if } 3.5 \le S \le 16 \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$\text{DeckThickness} := \begin{cases} \text{"OK"} & \text{if } 4.5 \le t_{se} \le 12 \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$\text{BridgeSpan} := \begin{cases} \text{"OK"} & \text{if } 20 \le L \le 240 \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$\text{NoBeams} := \begin{cases} \text{"OK"} & \text{if } ng \ge 4 \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$\text{LongitStiffness} := \begin{cases} \text{"OK"} & \text{if } 10000 \le K_g \le 7000000 \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$x := \begin{pmatrix} S & DeckSpan \\ t_{se} & DeckThickness \\ L & BridgeSpan \\ ng & NoBeams \\ K_g & LongitStiffness \end{pmatrix}$$

$$x = \begin{pmatrix} 7.5 & \text{"OK"} \\ 7.5 & \text{"OK"} \\ 146.0 & \text{"OK"} \\ 6.0 & \text{"OK"} \\ 3600866.5 & \text{"OK"} \end{pmatrix}$$

E19-1.6.1 Distribution Factors for Interior Beams:

One Lane Loaded:

$$g_{i1} := 0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1} \qquad \boxed{g_{i1} = 0.435}$$

Two or More Lanes Loaded:

$$g_{i2} := 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1} \qquad \boxed{g_{i2} = 0.636}$$

$$g_i := \max\left(g_{i1}, g_{i2}\right) \qquad \boxed{g_i = 0.636}$$

Note: The distribution factors above already have a multiple presence factor included that is used for service and strength limit states. For fatigue limit states, the 1.2 multiple presence factor should be divided out.

E19-1.6.2 Distribution Factors for Exterior Beams:

Two or More Lanes Loaded:

Per **LRFD [Table 4.6.2.2.2d-1]** the distribution factor shall be calculated by the following equations:

$$w_{parapet} := \frac{w_b - w}{2}$$ Width of parapet overlapping the deck $\boxed{w_{parapet} = 1.250}$ ft

$$d_e := s_{oh} - w_{parapet}$$ Distance from the exterior web of exterior beam to the interior edge of parapet, ft. $\boxed{d_e = 1.250}$ ft

Note: Conservatively taken as the distance from the center of the exterior girder.

Check range of applicability for de:

$$d_{e\_check} := \begin{cases} \text{"OK"} & \text{if } -1.0 \le d_e \le 5.5 \\ \text{"NG"} & \text{otherwise} \end{cases} \qquad \boxed{d_{e\_check} = \text{"OK"}}$$

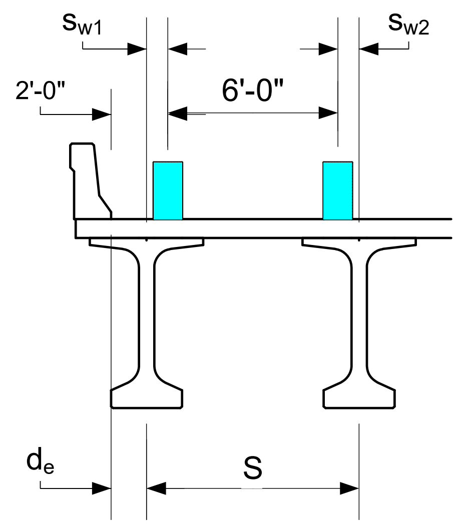
Note: While AASHTO allows the d_e value to be up to 5.5, the deck overhang (from the center of the exterior girder to the edge of the deck) is limited by WisDOT policy as stated in Chapter 17 of the Bridge Manual.

$$e := 0.77 + \frac{d_e}{9.1} \qquad \boxed{e = 0.907}$$

$$g_{x2} := e \cdot g_i \qquad \boxed{g_{x2} = 0.577}$$

One Lane Loaded:

Per **LRFD [Table 4.6.2.2.2d-1]** the distribution factor shall be calculated by the Lever Rule.



$$s_{w1} := d_e - 2$$ Distance from center of exterior girder to outside wheel load, ft. $\boxed{s_{w1} = -0.750}$ ft

$$s_{w2} := S + s_{w1} - 6$$ Distance from wheel load to first interior girder, ft. $\boxed{s_{w2} = 0.750}$ ft

$$R_x := \frac{S + s_{w1} + s_{w2}}{S \cdot 2} \qquad \boxed{R_x = 0.500}$$ % of a lane load

Add the single lane multi-presence factor, $m := 1.2$

$$g_{x1} := R_x \cdot 1.2 \qquad \boxed{g_{x1} = 0.600}$$

The exterior girder distribution factor is the maximum value of the One Lane Loaded case and the Two or More Lanes Loaded case:

$$g_x := \max\left(g_{x1}, g_{x2}\right) \qquad \boxed{g_x = 0.600}$$

Note: The interior girder has a larger live load distribution factor and a larger dead load than the exterior girder. Therefore, for this example, the interior girder is likely to control.

E19-1.6.3 Distribution Factors for Fatigue:

The distribution factor for fatigue is the single lane distribution factor with the multi-presence factor, $m = 1.200$, removed:

$$g_{if} := \frac{g_{i1}}{1.2} \qquad \boxed{g_{if} = 0.362}$$

E19-1.7 Limit States and Combinations

The limit states, load factors and load combinations shall be applied as required and detailed in chapter 17 of this manual and as indicated below.

E19-1.7.1 Load Factors

From **LRFD [Table 3.4.1-1 & Table 3.4.1-4]**:

	DC	DW	LL
Strength 1	$\gamma st_{DC} := 1.25$	$\gamma st_{DW} := 1.50$	$\gamma st_{LL} := 1.75$
Service 1	$\gamma s1_{DC} := 1.0$	$\gamma s1_{DW} := 1.0$	$\gamma s1_{LL} := 1.0$
Service 3	$\gamma s3_{DC} := 1.0$	$\gamma s3_{DW} := 1.0$	$\gamma s3_{LL} := 0.8$ Check Tension Stress
Fatigue I			$\gamma f_{LL} := 1.75$

Dynamic Load Allowance (IM) is applied to the truck and tandem.

E19-1.7.2 Dead Load Moments

The unfactored dead load moments are listed below (values are in kip-ft):

Unfactored Dead Load Interior Girder Moments (kip-ft)				
Tenth Point (Along Span)	DC girder at release	DC non-composite	DC composite	DW composite
0	35	0	0	0
0.1	949	1759	124	128
0.2	1660	3128	220	227
0.3	2168	4105	289	298
0.4	2473	4692	330	341
0.5	2574	4887	344	355

The DC_{nc} values are the component non-composite dead loads and include the weight of the girder, haunch, diaphragms and the deck.

The DC_c values are the component composite dead loads and include the weight of the parapets.

The DW_c values are the composite dead loads from the future wearing surface.

Note that the girder dead load moments at release are calculated based on the girder length. The moments for other loading conditions are calculated based on the span length (center to center of bearing).

E19-1.7.3 Live Load Moments

The unfactored live load load moments (per lane including impact) are listed below (values are in kip-ft). Note that the impact factor is applied only to the truck portion of the HL-93 loads. A separate analysis run will be required if results without impact are desired.

Unfactored Live Load + Impact Moments per Lane (kip-ft)			
Tenth Point	Truck	Tandem	Fatigue
0	0	0	0
0.1	1783	1474	937
0.2	2710	2618	1633
0.3	4100	3431	2118
0.4	4665	3914	2383
0.5	4828	4066	2406

The Wisconsin Standard Permit Vehicle should also be checked. See Chapter 45 - Bridge Rating for further information.

The unfactored live load moments per lane are calculated by applying the appropriate distribution factor to the controlling moment. For the interior girder:

$$g_i = 0.636$$

$$M_{LL} = g_i \cdot 4828 \qquad \boxed{M_{LL} = 3072.8} \text{ kip-ft}$$

$$\boxed{g_{if} = 0.362}$$

$$M_{LLfat} := g_{if} \cdot 2406 \qquad \boxed{M_{LLfat} = 871.4} \text{ kip-ft}$$

E19-1.7.4 Factored Moments

WisDOT's policy is to set all of the load modifiers, η, equal to 1.0. The factored moments for each limit state are calculated by applying the appropriate load factor to the girder moments. For the interior girder:

Strength 1

$$M_{str} := \eta \cdot \left[\gamma st_{DC} \cdot \left(M_{DLnc} + M_{DLc}\right) + \gamma st_{DW} \cdot M_{DWc} + \gamma st_{LL} \cdot M_{LL}\right]$$

$$= 1.0 \cdot \left[1.25 \cdot \left(M_{DLnc} + M_{DLc}\right) + 1.50 \cdot M_{DWc} + 1.75 \cdot M_{LL}\right] \qquad \boxed{M_{str} = 12449.3} \text{ kip-ft}$$

Service 1 (for compression checks)

$$M_{s1} := \eta \cdot \left[\gamma s1_{DC} \cdot \left(M_{DLnc} + M_{DLc}\right) + \gamma s1_{DW} \cdot M_{DWc} + \gamma s1_{LL} \cdot M_{LL}\right]$$

$$= 1.0 \cdot \left[1.0 \cdot \left(M_{DLnc} + M_{DLc}\right) + 1.0 \cdot M_{DWc} + 1.0 \cdot M_{LL}\right] \qquad \boxed{M_{s1} = 8659.3} \text{ kip-ft}$$

Service 3 (for tension checks)

$$M_{s3} := \eta \cdot \left[\gamma s3_{DC} \cdot \left(M_{DLnc} + M_{DLc}\right) + \gamma s3_{DW} \cdot M_{DWc} + \gamma s3_{LL} \cdot M_{LL}\right]$$

$$= 1.0 \cdot \left[1.0 \cdot \left(M_{DLnc} + M_{DLc}\right) + 1.0 \cdot M_{DWc} + 0.8 \cdot M_{LL}\right] \qquad \boxed{M_{s3} = 8044.7} \text{ kip-ft}$$

Service 1 and 3 non-composite DL alone

$$M_{nc} := \eta \cdot \gamma s1_{DC} \cdot M_{DLnc} \qquad \boxed{M_{nc} = 4887.5} \text{ kip-ft}$$

Fatigue 1

$$M_{fat} := \eta \cdot \gamma f_{LL} \cdot M_{LLfat} \qquad \boxed{M_{fat} = 1524.9} \text{ kip-ft}$$

E19-1.8 Composite Girder Section Properties

Calculate the effective flange width in accordance with **LRFD [4.6.2.6]** and section 17.2.11 of the Wisconsin Bridge Manual:

$w_e := S \cdot 12$ $\boxed{w_e = 90.00}$ in

The effective width, w_e, must be adjusted by the modular ratio, n, to convert to the same concrete material (modulus) as the girder.

$w_{eadj} := \frac{w_e}{n}$ $\boxed{w_{eadj} = 58.46}$ in

Calculate the composite girder section properties:

effective slab thickness; $\boxed{t_{se} = 7.50}$ in

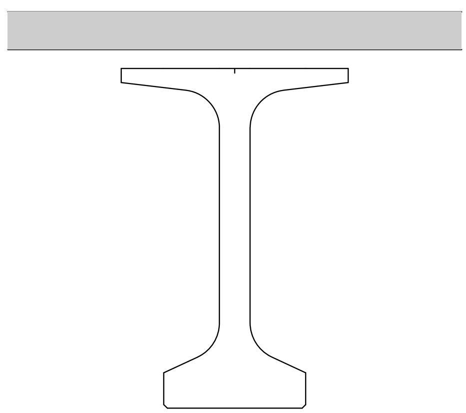
effective slab width; $\boxed{w_{eadj} = 58.46}$ in

haunch thickness; $hau := 2.00$ in

total height; $h_c := ht + hau + t_{se}$

$\boxed{h_c = 81.50}$ in

$\boxed{n = 1.540}$



Note: The area of the concrete haunch is not included in the calculation of the composite section properties.

Component	Ycg	A	AY	AY^2	I	I+AY^2
Deck	77.75	438	34088	2650309	2055	2652364
Girder	34.87	915	31906	1112564	656426	1768990
Haunch	73	0	0	0	0	0
Summation		1353	65994			4421354

$\boxed{\Sigma A = 1353}$ in^2

$\boxed{\Sigma AY = 65994}$ in^3

$\boxed{\Sigma IplusAYsq = 4421354}$ in^4

$$y_{cgb} := \frac{-\Sigma AY}{\Sigma A}$$ $\boxed{y_{cgb} = -48.8}$ in

$$y_{cgt} := ht + y_{cgb}$$ $\boxed{y_{cgt} = 23.2}$ in

$$A_{cg} := \Sigma A$$ $\boxed{A_{cg} = 1353}$ in^2

$$I_{cg} := \Sigma IplusAYsq - A_{cg} \cdot y_{cgb}^{\ 2}$$ $\boxed{I_{cg} = 1203475}$ in^4

$$S_{cgt} := \frac{I_{cg}}{y_{cgt}}$$ $\boxed{S_{cgt} = 51786}$ in^3

$$S_{cgb} := \frac{I_{cg}}{y_{cgb}}$$ $\boxed{S_{cgb} = -24681}$ in^3

Deck:

$$S_{cgdt} := n \cdot \frac{I_{cg}}{y_{cgt} + hau + t_{se}}$$ $\boxed{S_{cgdt} = 56594}$ in^3

$$S_{cgdb} := n \cdot \frac{I_{cg}}{y_{cgt} + hau}$$ $\boxed{S_{cgdb} = 73411}$ in^3

E19-1.9 Preliminary Design Information:

Calculate initial girder loads, service loads, and estimate prestress losses. This information will be utilized in the preliminary design steps.

Note: The initial girder loads will be used to check stresses at transfer (before losses) and the service loads will be used to check stresses while in service (after losses). These calculations and the estimated prestress losses will then be used to select the number of strands for final design calculations.

At transfer (Interior Girder):

$M_{iend} := 0$ kip-ft

$$M_g := w_g \cdot \frac{L_g^{\ 2}}{8}$$ $\boxed{M_g = 2574.2}$ kip-ft

At service (Interior Girder):

Service 1 Moment $\boxed{M_{s1} = 8659.3}$ kip-ft

Service 3 Moment $\boxed{M_{s3} = 8044.7}$ kip-ft

Service 1 Moment Components:

non-composite moment (girder + deck) $\boxed{M_{nc} = 4887.5}$ kip-ft

composite moment (parapet, FWS and LL)

$M_{1c} := M_{s1} - M_{nc}$ $\boxed{M_{1c} = 3771.8}$ kip-ft

Service 3 Moment Components:

non-composite moment (girder + deck) $\boxed{M_{nc} = 4887.5}$ kip-ft

composite moment (parapet, FWS and LL)

$M_{3c} := M_{s3} - M_{nc}$ $\boxed{M_{3c} = 3157.2}$ kip-ft

At service the prestress has decreased (due to CR, SH, RE):

Estimated time dependant losses $\boxed{F_{Delta} := 32.000}$ ksi

Note: The estimated time dependant losses (for low relaxation strands) will be re-calculated using the approximate method in accordance with **LRFD [5.9.3.3]** once the number of strands has been determined.

Assume an initial strand stress; $f_{tr} := 0.75 \cdot f_{pu}$ $\boxed{f_{tr} = 202.500}$ ksi

Based on experience, assume $\Delta f_{pES\_est} := 18$ ksi loss from elastic shortening. As an alternate initial estimate, **LRFD [C.5.9.3.2.3a]** suggests assuming a 10% ES loss.

$ES_{loss} := \frac{\Delta f_{pES\_est}}{f_{tr}} \cdot 100$ $\boxed{ES_{loss} = 8.889}$ %

$f_i := f_{tr} - \Delta f_{pES\_est}$ $\boxed{f_i = 184.500}$ ksi

The total loss is the time dependant losses plus the ES losses:

$loss := F_{Delta} + \Delta f_{pES\_est}$ $\boxed{loss = 50.0}$ ksi

$loss_{\%} := \frac{loss}{f_{tr}} \cdot 100$ $\boxed{loss_{\%} = 24.7}$ % (estimated)

If T_o is the initial prestress, then (1-loss)*T_o is the remaining:

$$T = \left(1 - loss_{\%}\right) \cdot T_o$$

$$ratio := 1 - \frac{loss_{\%}}{100}$$

$$\boxed{ratio = 0.753}$$

$$T = ratio \cdot T_o$$

E19-1.10 Preliminary Design Steps

The following steps are utilized to design the prestressing strands:

1) Design the amount of prestress to prevent tension at the bottom of the beam under the full load at center span after losses.

2) Calculate the prestress losses and check the girder stresses at mid span at the time of transfer.

3) Design the eccentricity of the strands at the girder end to avoid tension or compression over-stress at the time of transfer.

4) If required, design debonding of strands to prevent over-stress at the girder ends.

5) Check resulting stresses at the critical sections of the girder at the time of transfer and after losses.

E19-1.10.1 Determine Amount of Prestress

Design the amount of prestress to prevent tension at the bottom of the beam under the full load (at center span) after losses.

Near center span, after losses, T = the remaining effective prestress, aim for no tension at the bottom. Use Service I for compression and Service III for tension.

For this example, the interior girder has the controlling moments.

Calculate the stress at the bottom of the beam due to combination of non-composite and composite loading (Service 3 condition):

$$f_b := \frac{M_{nc} \cdot 12}{S_b} + \frac{M_{3c} \cdot 12}{S_{cgb}} \qquad \boxed{f_b = -4.651} \text{ ksi}$$

Stress at bottom due to prestressing (after losses):

$$f_{bp} = \frac{T}{A} \cdot \left(1 + e \cdot \frac{y_b}{r^2}\right) \qquad \text{where } T = \left(1 - loss_{\%}\right) \cdot T_o$$

and $f_{bp} := -f_b$ desired final prestress.

We want this to balance out the tensile stress calculated above from the loading, i.e. an initial compression. Since we are making some assumptions on the actual losses, we are ignoring the allowable tensile stress in the concrete for these calculations.

$$f_{bp} = \frac{\left(1 - loss_{\%}\right) \cdot T_o}{A} \cdot \left(1 + e \cdot \frac{y_b}{r^2}\right) \qquad \text{(after losses)}$$

OR:

$$\frac{f_{bp}}{1 - loss_{\%}} = \frac{T_o}{A} \cdot \left(1 + e \cdot \frac{y_b}{r^2}\right)$$

$$f_{bpi\_1} := \frac{f_{bp}}{1 - \frac{loss_{\%}}{100}}$$ $\boxed{f_{bpi\_1} = 6.175}$ ksi

desired bottom initial prestress (before losses)

If we use the actual allowable tensile stress in the concrete, the desired bottom initial prestress is calculated as follows:

The allowable tension, from **LRFD [5.9.2.3.2b]**, is:

$f_{tall} := 0.19 \cdot \lambda \sqrt{f'_c} \le 0.6$ ksi; $\lambda = 1.0$ (norm. wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_{tall} = 0.537}$ ksi

The desired bottom initial prestress (before losses):

$f_{bpi\_2} := f_{bpi\_1} - f_{tall}$ $\boxed{f_{bpi\_2} = 5.638}$ ksi

Determine the stress effects for different strand patterns on the 72W girder:

$A_{strand} = 0.21\,in^2$

$f'_s := 270000$ psi

$f_s := 0.75 \cdot f'_s$ $\boxed{f_s = 202500}$ psi

$P := A_{strand} \cdot \frac{f_s}{1000}$ $\boxed{P = 43.94}$ kips

$$f_{bpi} := \frac{P \cdot N}{A_g} \cdot \left(1 + e \cdot \frac{y_b}{r_{sq}}\right)$$ (bottom initial prestress - before losses)

The values of f_{bpi} for various strand patterns is shown in the following table.

72W Stress Effects		
Pi (per strand) = 43.94 kips		
No. Strands	e (in)	bottom stress (ksi)
36	-31.09	4.3411
38	-30.98	4.5726
40	-30.87	4.8030
42	-30.77	5.0333
44	-30.69	5.2648
46	-30.52	5.4858
48	-30.37	5.7075
50	-30.23	5.9290
52	-30.10	6.1504

Solution:

Try $ns := 44$ strands, 0.6 inch diameter.

Initial prestress at bottom $f_{bpi} := 5.2648$ ksi,

Eccentricity, $e_s := -30.69$ inches; actual tension should be less than allowed.

E19-1.10.2 Prestress Loss Calculations

The loss in prestressing force is comprised of the following components:

1) Elastic Shortening (ES), shortening of the beam as soon as prestress is applied.

2) Shrinkage (SH), shortening of the concrete as it hardens, time function.

3) Creep (CR), slow shortening of concrete due to permanent compression stresses in the beam, time function.

4) Relaxation (RE), the tendon slowly accommodates itself to the stretch and the internal stress drops with time

E19-1.10.2.1 Elastic Shortening Loss

at transfer (before ES loss) **LRFD [5.9.2.2]**

$$T_{oi} := ns \cdot f_{tr} \cdot A_{strand} \qquad \boxed{= 44 \cdot 0.75 \cdot 270 \cdot A_{strand} = 1933} \text{ kips}$$

The ES loss estimated above was: $\Delta f_{pES\_est} = 18.0$ ksi, or $ES_{loss} = 8.889$ %. The resulting force in the strands after ES loss:

$$T_o := \left(1 - \frac{ES_{loss}}{100}\right) \cdot T_{oi} \qquad \boxed{T_o = 1761.6} \text{ kips}$$

If we assume all strands are straight we can calculate the initial elastic shortening loss;

$$f_{cgp} := \frac{T_o}{A_g} + (T_o \cdot e_s) \cdot \frac{e_s}{I_g} + M_g \cdot 12 \cdot \frac{e_s}{I_g} \qquad \boxed{f_{cgp} = 3.009} \text{ ksi}$$

$$\boxed{E_{ct} = 4999} \text{ ksi}$$

$$E_p := E_s \qquad \boxed{E_p = 28500} \text{ ksi}$$

$$\Delta f_{pES} := \frac{E_p}{E_{ct}} \cdot f_{cgp} \qquad \boxed{\Delta f_{pES} = 17.152} \text{ ksi}$$

This value of Δf_{pES} is in agreement with the estimated value above; $\Delta f_{pES\_est} = 18.00$ ksi. If these values did not agree, T_o would have to be recalculated using f_{tr} minus the new value of Δf_{pES}, and a new value of f_{cgp} would be determined. This iteration would continue until the assumed and calculated values of Δf_{pES} are in agreement.

The initial stress in the strand is:

$f_i := f_{tr} - \Delta f_{pES}$ $\boxed{f_i = 185.348}$ ksi

The force in the beam after transfer is:

$T_o := ns \cdot A_{strand} \cdot f_i$ $\boxed{T_o = 1770}$ kips

Check the design to avoid premature failure at the center of the span at the time of transfer. Check the stress at the center span (at the plant) at both the top and bottom of the girder.

$$f_{ttr} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_t} + \frac{M_g \cdot 12}{S_t}$$ $\boxed{f_{ttr} = 0.609}$ ksi

$$f_{btr} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_b} + \frac{M_g \cdot 12}{S_b}$$ $\boxed{f_{btr} = 3.178}$ ksi

temporary allowable stress (compression) **LRFD [5.9.2.3.1a]**:

$f_{ciall} := 0.65 \cdot f'_{ci}$ $\boxed{f_{ciall} = 4.420}$ ksi

Is the stress at the bottom of the girder less than the allowable? $\boxed{check = \text{"OK"}}$

E19-1.10.2.2 Approximate Estimate of Time Dependant Losses

Calculate the components of the time dependant losses; shrinkage, creep and relaxation, using the approximate method in accordance with **LRFD [5.9.3.3]**.

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

From **LRFD [Figure 5.4.2.3.3-1]**, the average annual ambient relative humidity, $H := 72$ %.

$\gamma_h := 1.7 - 0.01 \cdot H$ $\boxed{\gamma_h = 0.980}$

$\gamma_{st} := \dfrac{5}{1 + f'_{ci}}$ $\boxed{\gamma_{st} = 0.641}$

$\Delta f_{pR} := 2.4$ ksi for low relaxation strands

$\Delta f_{pCR} := 10.0 \cdot \dfrac{f_{tr} \cdot A_{strand} \cdot ns}{A_g} \cdot \gamma_h \cdot \gamma_{st}$ $\boxed{\Delta f_{pCR} = 13.274}$ ksi

$\Delta f_{pSR} := 12.0 \cdot \gamma_h \cdot \gamma_{st}$ $\boxed{\Delta f_{pSR} = 7.538}$ ksi

$\Delta f_{pRE} := \Delta f_{pR}$ $\boxed{\Delta f_{pRE} = 2.400}$ ksi

$\Delta f_{pLT} := \Delta f_{pCR} + \Delta f_{pSR} + \Delta f_{pRE}$ $\boxed{\Delta f_{pLT} = 23.213}$ ksi

The total estimated prestress loss (Approximate Method):

$\Delta f_p := \Delta f_{pES} + \Delta f_{pLT}$ $\boxed{\Delta f_p = 40.365}$ ksi

$\boxed{\dfrac{\Delta f_p}{f_{tr}} \cdot 100 = 19.93}$ % total prestress loss

This value is slightly less than but in general agreement with the initial estimated $loss_{\%} = 24.691$.

The remaining stress in the strands and total force in the beam after all losses is:

$f_{pe} := f_{tr} - \Delta f_p$ $\boxed{f_{pe} = 162.13}$ ksi

$T := ns \cdot A_{strand} \cdot f_{pe}$ $\boxed{T = 1548}$ kips

E19-1.10.3 Design of Strand Drape

Design the eccentricity of the strands at the end to avoid tension or compression over stress at the time of transfer. Check the top stress at the end. If the strands are straight, $M_g = 0$.

top:

$f_{tetr} := \dfrac{T_o}{A_g} + \dfrac{T_o \cdot e_s}{S_t}$ $\boxed{f_{tetr} = -1.138}$ ksi

high tension stress

In accordance with **LRFD [Table 5.9.2.3.1b]**, the temporary allowable tension stress is calculated as follows (assume there is no bonded reinforcement):

$f_{tiall} := -\min\left(0.0948 \cdot \lambda \sqrt{f'_{ci}}, 0.2\right)$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_{tiall} = -0.200}$ ksi

bottom:

$$f_{betr} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_b}$$ $\boxed{f_{betr} = 4.819}$ ksi

$\boxed{f_{ciall} = 4.420}$ ksi

high compressive stress

The tension at the top is too high, and the compression at the bottom is also too high!!

Drape some of the strands upward to decrease the top tension and decrease the compression at the bottom.

Find the required position of the steel centroid to avoid tension at the top. Conservatively set the top stress equal to zero and solve for "e":

$$f_{tetr} = \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_t}$$

$$e_{sendt} := \frac{S_t}{T_o} \cdot \left(0 - \frac{T_o}{A_g}\right)$$ $\boxed{e_{sendt} = -19.32}$ inches or higher

Therefore, we need to move the resultant centroid of the strands up:

$move := e_{sendt} - e_s$ $\boxed{move = 11.37}$ inches upward

Find the required position of the steel centroid to avoid high compression at the bottom of the beam. Set the bottom compression equal to the allowable stress and find where the centroid of $ns = 44$ strands needs to be:

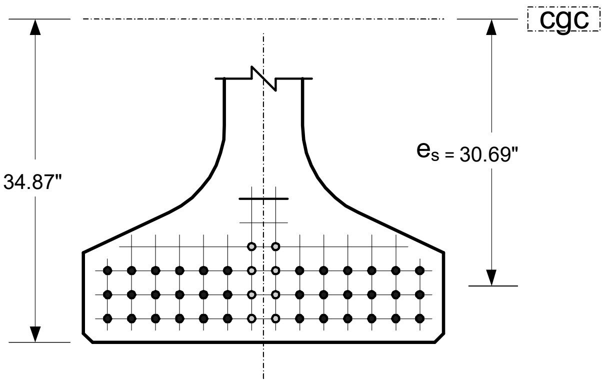
$$f_{betr} = \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_b}$$

Set equal to allowed: $f_{betr} := f_{ciall}$

$$e_{sendb} := \frac{S_b}{T_o} \cdot \left(f_{ciall} - \frac{T_o}{A_g}\right)$$ $\boxed{e_{sendb} = -26.44}$ inches or higher

Top stress condition controls:

$e_{send} := \max\left(e_{sendt}, e_{sendb}\right)$ $\boxed{e_{send} = -19.32}$ inches



36 undraped strands
8 draped strands

LRFD [Table 5.10.1-1] requires 2 inches of cover. However, WisDOT uses 2 inches to the center of the strand, and 2 inch spacing between centers.

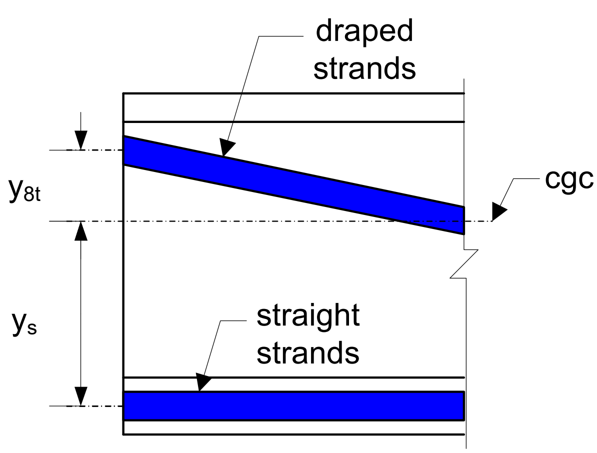
The center $ns_d := 8$ strands will be draped at the end of the girder.

Find the center of gravity of the remaining $ns_s = 36$ straight strands from the bottom of the girder:

$$Y_s := \frac{12 \cdot 2 + 12 \cdot 4 + 12 \cdot 6}{ns_s}$$ $\boxed{Y_s = 4.00}$ inches from the bottom of the girder

OR:

$$y_s := y_b + Y_s$$ $\boxed{y_s = -30.87}$ inches from the center of gravity of the girder (cgc)



y_{8t} is the eccentricity of the draped strands at the end of the beam. We want the eccentricity of all of the strands at the end of the girder to equal, $e_{send} = -19.322$ inches for stress control.

$$e_{send} = \frac{ns_s \cdot y_s + ns_d \cdot y_{8t}}{ns}$$

$$y_{8t} := \frac{ns \cdot e_{send} - ns_s \cdot y_s}{ns_d}$$ $\boxed{y_{8t} = 32.64}$ inches above the cgc

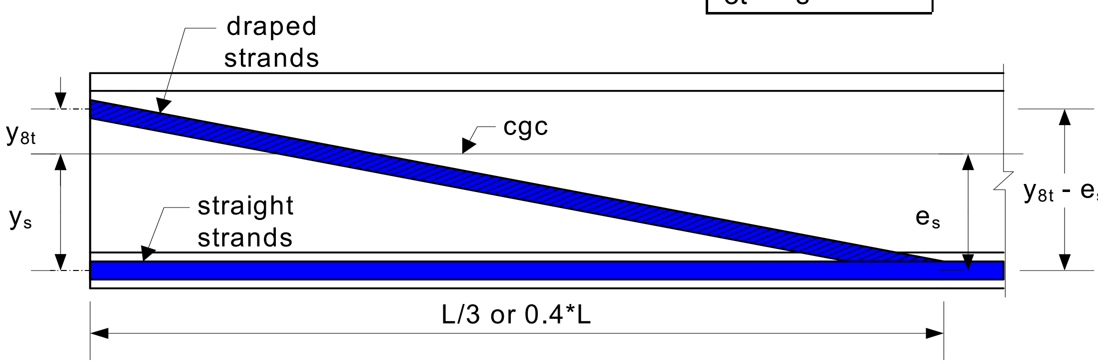
However, $y_t = 37.13$ inches to the top of the beam. If the draped strands are raised $y_{8t} = 32.64$ inches or more above the cgc, the stress will be OK.

Drape the center strands the maximum amount: Maximum drape for $ns_d = 8$ strands:

$y_{8t} := y_t - 5$ $\boxed{y_{8t} = 32.13}$ in

$\boxed{e_s = -30.69}$ in

$\boxed{y_{8t} - e_s = 62.82}$ in



Try a drape length of: $\frac{L_g}{3} = 49.00$ feet $HD := \frac{L_g}{3}$

The eccentricity of the draped strands at the hold down point:

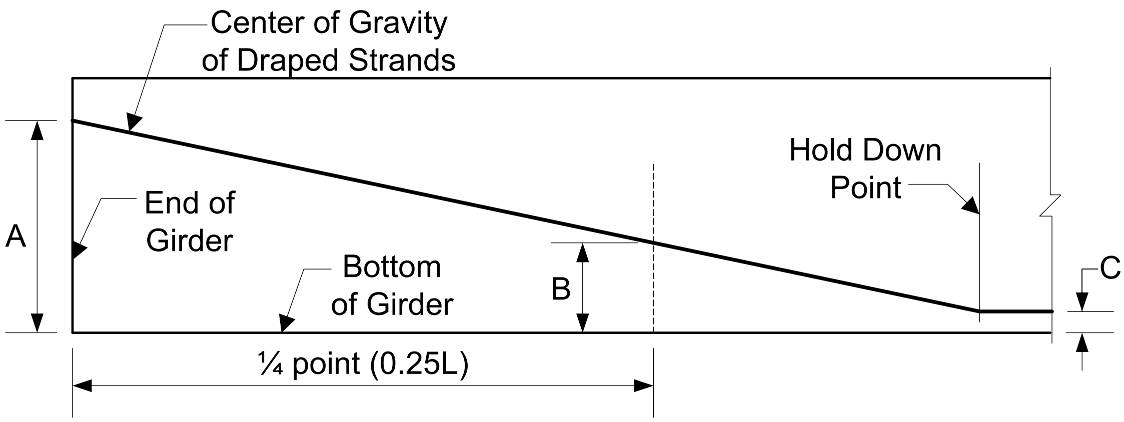
$e_{8hd} := y_b + 5$ $\boxed{e_{8hd} = -29.870}$ in

Strand slope, $slope := \frac{y_{8t} - e_{8hd}}{(HD \cdot 12)} \cdot 100$ $\boxed{slope = 10.54}$ %

Is the slope of the strands less than 12%? $\boxed{check = \text{"OK"}}$

12% is a suggested maximum slope, actual acceptable slope is dependant on the form capacity or on the fabricator.

Calculate the values of A, B_{min}, B_{max} and C to show on the plans:



$A := |y_b| + y_{8t}$ $\boxed{A = 67.00}$ in

$\boxed{C := 5.00}$ in

$B_{min} := \dfrac{A + 3C}{4}$ $\boxed{B_{min} = 20.50}$ in

$B_{max} := B_{min} + 3$ $\boxed{B_{max} = 23.50}$ in

Check hold down location for B_{max} to make sure it is located between $L_g/3$ and $0.4*L_g$:

$slope_{Bmax} := \dfrac{A - B_{max}}{0.25 \cdot L_g \cdot 12}$ $\boxed{slope_{Bmax} = 0.099}$ ft/ft

$x_{Bmax} := \dfrac{A - C}{slope_{Bmax}} \cdot \dfrac{1}{12}$ $\boxed{x_{Bmax} = 52.38}$ ft

$\boxed{L_g \cdot 0.4 = 58.80}$ ft

Is the resulting hold down location less than $0.4*L_g$? $\boxed{check = \text{"OK"}}$

Check the girder stresses at the end of the transfer length of the strands at release:

Minimum moment on section = girder moment at the plant

The transfer length may be taken as:

$l_{tr} := 60 \cdot d_b$ $\boxed{l_{tr} = 36.00}$ in

$x := \dfrac{l_{tr}}{12}$ $\boxed{x = 3.00}$ feet

The eccentricity of the draped strands and the entire strand group at the transfer length is:

$$y_{8tt} := y_{8t} - \frac{slope}{100} \cdot x \cdot 12$$ $\boxed{y_{8tt} = 28.334}$ in

$$e_{st} := \frac{ns_s \cdot y_s + 8 \cdot y_{8tt}}{ns}$$ $\boxed{e_{st} = -20.106}$ in

The moment at the end of the transfer length due to the girder dead load:

$$M_{gt} := \frac{w_g}{2} \cdot \left(L_g \cdot x - x^2\right)$$ $\boxed{M_{gt} = 206}$ kip-ft

The girder stresses at the end of the transfer length:

$$f_{tt} := \frac{T_o}{A_g} + \frac{T_o \cdot e_{st}}{S_t} + \frac{M_{gt} \cdot 12}{S_t}$$ $\boxed{f_{tt} = 0.061}$ ksi

$\boxed{f_{tiall} = -0.200}$ ksi

Is f_{tt} less than f_{tiall}? $\boxed{check = "OK"}$

$$f_{bt} := \frac{T_o}{A_g} + \frac{T_o \cdot e_{st}}{S_b} + \frac{M_{gt} \cdot 12}{S_b}$$ $\boxed{f_{bt} = 3.693}$ ksi

$\boxed{f_{ciall} = 4.420}$ ksi

Is f_{bt} less than f_{ciall}? $\boxed{check = "OK"}$

E19-1.10.4 Stress Checks at Critical Sections

Critical Sections	Critical Conditions		
	At Transfer	Final	Fatigue
Girder Ends	X		
Midspan	X	X	X
Hold Down Points	X	X	X

Data:

$\boxed{T_o = 1770}$ kips $\boxed{T = 1548}$ kips

$\boxed{M_{nc} = 4887}$ kip-ft $\boxed{M_{s3} = 8045}$ kip-ft

$\boxed{M_{s1} = 8659}$ kip-ft $\boxed{M_g = 2574}$ kip-ft

Need moments at hold down points: $\boxed{\frac{L_g}{3} = 49.00}$ feet, from the end of the girder.

girder: $M_{ghd} = 2288$ kip-ft

non-composite: $M_{nchd} = 4337$ kip-ft

Service I composite: $M_{1chd} = 3371$ kip-ft

Service III composite: $M_{3chd} = 2821$ kip-ft

Note: The release girder moments shown above at the hold down location are calculated based on the total girder length.

Check the girder at the end of the beam (at the transfer length):

$\boxed{e_{st} = -20.11}$ inches $\boxed{f_{tiall} = -0.200}$ ksi $\boxed{f_{ciall} = 4.420}$ ksi

At transfer, $\boxed{M_{gt} = 206}$ kip-ft

<u>Top of girder (Service 3):</u>

$$f_{tei} := \frac{T_o}{A_g} + \frac{T_o \cdot e_{st}}{S_t} + \frac{M_{gt} \cdot 12}{S_t}$$ $\boxed{f_{tei} = 0.061}$ ksi

Is f_{tei} greater than f_{tiall}? $\boxed{check = \text{"OK"}}$

<u>Bottom of girder (Service 1):</u>

$$f_{bei} := \frac{T_o}{A_g} + \frac{T_o \cdot e_{st}}{S_b} + \frac{M_{gt} \cdot 12}{S_b}$$ $\boxed{f_{bei} = 3.693}$ ksi

Is f_{bei} less than f_{ciall}? $\boxed{check = \text{"OK"}}$

Check at the girder and deck at midspan:

$e_s = -30.69$ inches

Initial condition at transfer: $f_{tiall} = -0.200$ ksi $f_{ciall} = 4.420$ ksi

Top of girder stress (Service 3):

$$f_{ti} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_t} + \frac{M_g \cdot 12}{S_t}$$

$f_{ti} = 0.609$ ksi

Is f_{ti} greater than f_{tiall}? check = "OK"

Bottom of girder stress (Service 1):

$$f_{bi} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_b} + \frac{M_g \cdot 12}{S_b}$$

$f_{bi} = 3.178$ ksi

Is f_{bi} less than f_{ciall}? check = "OK"

Final condition:

Allowable Stresses,**LRFD [5.9.2.3.2]**:

There are two compressive stress limits: (Service 1) **LRFD [5.9.2.3.2a]**

$f_{call1} := 0.45 \cdot f'_c$ PS + DL $f_{call1} = 3.600$ ksi

$f_{call2} := 0.60 \cdot f'_c$ LL + PS + DL $f_{call2} = 4.800$ ksi

(Service 3) **LRFD [5.9.2.3.2b]** (Moderate Corrosion Condition)

tension: $f_{tall} = -0.19 \cdot \lambda \sqrt{f'_c}$ $\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

$f_{tall} := -0.19 \cdot \sqrt{f'_c}$ $|f_{tall}| \le 0.6$ ksi $f_{tall} = -0.537$ ksi

Allowable Stresses (Fatigue),**LRFD [5.5.3]**:

Fatige compressive stress limit:

$f_{call\_fat} := 0.40 \cdot f'_c$ LLfat + 1/2(PS + DL) $f_{call\_fat} = 3.200$ ksi

Top of girder stress (Service 1):

$$f_{t1} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_t} + \frac{M_{DLnc} \cdot 12}{S_t} + \frac{(M_{DLc} + M_{DWc}) \cdot 12}{S_{cgt}}$$

$\boxed{f_{t1} = 2.484}$ ksi

$$f_{t2} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_t} + \frac{M_{DLnc} \cdot 12}{S_t} + \frac{(M_{DLc} + M_{DWc} + M_{LL}) \cdot 12}{S_{cgt}}$$

$\boxed{f_{t2} = 3.196}$ ksi

Is f_t less than f_{call}?

$\boxed{check1 = \text{"OK"}}$

$\boxed{check2 = \text{"OK"}}$

Top of girder stress (Fatigue 1):

$$f_{tfat} := \frac{1}{2}\left(\frac{T}{A_g} + \frac{T \cdot e_s}{S_t} + \frac{M_{DLnc} \cdot 12}{S_t}\right) + \frac{\left[\frac{1}{2}(M_{DLc} + M_{DWc}) + M_{LLfat}\right] \cdot 12}{S_{cgt}}$$

$\boxed{f_{tfat} = 1.444}$ ksi

Is f_{tfat} less than f_{call_fat}?

$\boxed{check = \text{"OK"}}$

Bottom of girder stress (Service 3):

$$f_b := \frac{T}{A_g} + \frac{T \cdot e_s}{S_b} + \frac{M_{nc} \cdot 12}{S_b} + \frac{(M_{s3} - M_{nc}) \cdot 12}{S_{cgb}}$$

$\boxed{f_b = -0.435}$ ksi

Is f_{tb} greater than f_{tall}?

$\boxed{check = \text{"OK"}}$

Top of deck stress (Service 1):

$$f_{dall} := 0.40 \cdot f'_{cd}$$

$\boxed{f_{dall} = 1.600}$ ksi

$$f_{dt} := \frac{(M_{s1} - M_{nc}) \cdot 12}{S_{cgdt}}$$

$\boxed{f_{dt} = 0.800}$ ksi

Is f_{dt} less than f_{dall}?

$\boxed{check = \text{"OK"}}$

Bottom of deck stress (Service 1):

$$f_{db} := \frac{(M_{s1} - M_{nc}) \cdot 12}{S_{cgdb}}$$ $\boxed{f_{db} = 0.617}$ ksi

Is f_{db} less than f_{dall}? $\boxed{check = "OK"}$

Check at hold-down location:

At transfer: $\boxed{f_{tiall} = -0.200}$ ksi $\boxed{f_{ciall} = 4.420}$ ksi

Top of girder stress (Service 3):

$$f_{t3i} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_t} + \frac{M_{ghd} \cdot 12}{S_t}$$ $\boxed{f_{t3i} = 0.415}$ ksi

Is f_{t3i} greater than f_{tiall}? $\boxed{check = "OK"}$

Bottom of girder stress (Service 1):

$$f_{b3i} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_b} + \frac{M_{ghd} \cdot 12}{S_b}$$ $\boxed{f_{b3i} = 3.361}$ ksi

Is f_{b3i} less than f_{ciall}? $\boxed{check = "OK"}$

Final condition, after losses, full load: $\boxed{f_{tall} = -0.537}$ ksi $\boxed{f_{call2} = 4.800}$ ksi

Top of girder stress (Service 1):

$$f_{t3} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_t} + \frac{M_{nchd} \cdot 12}{S_t} + \frac{M_{1chd} \cdot 12}{S_{cgt}}$$ $\boxed{f_{t3} = 2.729}$ ksi

Is f_{t3} less than f_{call2}? $\boxed{check = "OK"}$

Top of girder stress (Fatigue 1):

$$f_{tfat} := \frac{1}{2}\cdot\left(\frac{T}{A_g} + \frac{T\cdot e_s}{S_t} + \frac{M_{nchd}\cdot 12}{S_t}\right) + \frac{M_{fatchd}\cdot 12}{S_{cgt}}$$ $\boxed{f_{tfat} = 1.373}$ ksi

Is f_{tfat} less than f_{call_fat}? $\boxed{check = \text{"OK"}}$

Bottom of girder stress (Service 3):

$$f_{b3} := \frac{T}{A_g} + \frac{T\cdot e_s}{S_b} + \frac{M_{nchd}\cdot 12}{S_b} + \frac{M_{3chd}\cdot 12}{S_{cgb}}$$ $\boxed{f_{b3} = 0.080}$ ksi

Is f_{b3} greater than f_{tall}? $\boxed{check = \text{"OK"}}$

Top of deck stress (Service 1):

$\boxed{f_{dall} = 1.600}$ ksi

$$f_{dt3} := \frac{(M_{1chd})\cdot 12}{S_{cgdt}}$$ $\boxed{f_{dt3} = 0.715}$ ksi

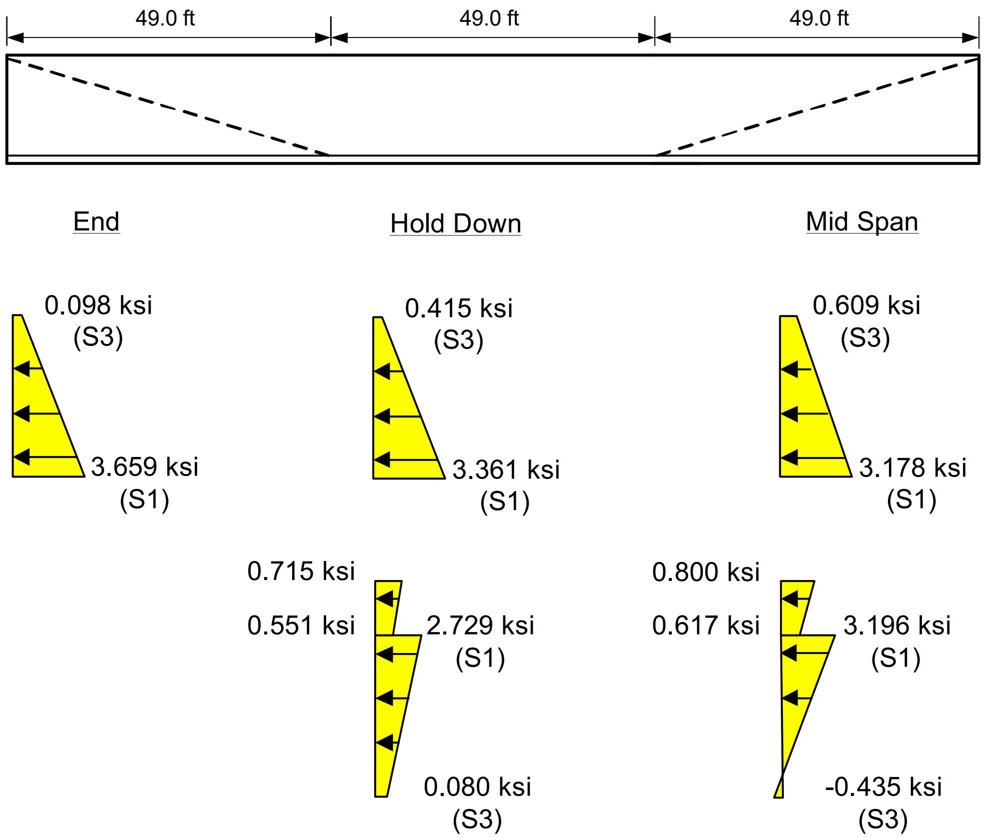
Is f_{dt} less than f_{dall}? $\boxed{check = \text{"OK"}}$

Bottom of deck stress (Service 1):

$$f_{db3} := \frac{(M_{1chd})\cdot 12}{S_{cgdb}}$$ $\boxed{f_{db3} = 0.551}$ ksi

Is f_{db} less than f_{dall}? $\boxed{check = \text{"OK"}}$

Summary of Design Stresses:



Initial Allowable:

$\text{compression} := f_{ciall} = 4.42$ ksi

Final Allowable:

$\text{compression}_1 := f_{call1} = 3.6$ ksi

$\text{compression}_2 := f_{call2} = 4.8$ ksi

$\text{compression\_fatigue} := f_{call\_fat} = 3.2$ ksi

$\text{tension} = f_{tall} = -0.537$ ksi **All stresses are acceptable!**

E19-1.11 Calculate Jacking Stress

The fabricator is responsible for calculation of the jacking force. See **LRFD [5.9.2]** for equations for low relaxation strands.

E19-1.12 Flexural Strength Capacity at Midspan

Check f_{pe} in accordance with **LRFD [5.6.3.1.1]**:

$\boxed{f_{pe} = 162.13}$ ksi $\boxed{0.5 \cdot f_{pu} = 135.00}$ ksi

Is $0.5^{*}f_{pu}$ less than f_{pe}? $\boxed{check = \text{"OK"}}$

Then at failure, we can assume that the tendon stress is:

$$f_{ps} = f_{pu}\left(1 - k \cdot \frac{c}{d_p}\right)$$

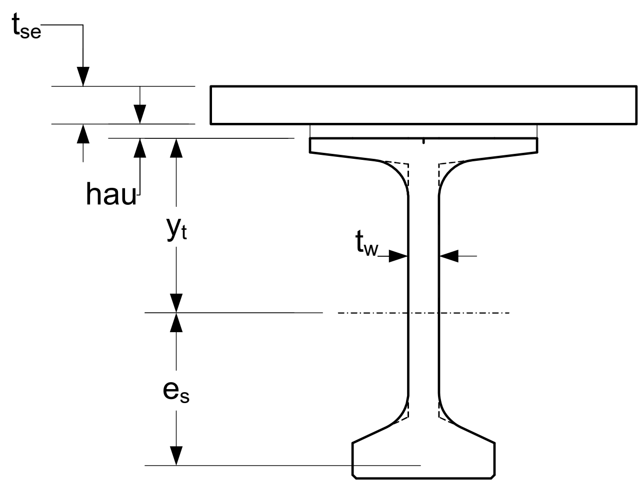
where:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right)$$

From **LRFD [Table C5.6.3.1.1-1]**, for low relaxation strands, $k := 0.28$.

"c" is defined as the distance between the neutral axis and the compression face (inches).

Assumed dimensions:



Assume that the compression block is in the deck. Calculate the capacity as if it is a rectangular section (with the compression block in the flange). The neutral axis location,calculated in accordance with **LRFD [5.6.3.1.1]** for a rectangular section, is:

$$c = \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$

where:

$A_{ps} := ns \cdot A_{strand}$ $\boxed{A_{ps} = 9.55}$ in²

$b := w_e$ $\boxed{b = 90.00}$ in

LRFD [5.6.2.2] $\alpha_1 := 0.85$ (for $f'_{cd} \le 10.0$ ksi)

$\beta_1 := \max\left[0.85 - \left(f'_{cd} - 4\right) \cdot 0.05, 0.65\right]$ $\boxed{\beta_1 = 0.850}$

$d_p := y_t + hau + t_{se} - e_s$ $\boxed{d_p = 77.32}$ in

$$c := \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 9.57}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 8.13}$ in

The calculated value of "a" is greater than the deck thickness. Therefore, the rectangular assumption is incorrect and the compression block extends into the haunch. Calculate the neutral axis location and capacity for a flanged section:

$h_f := t_{se}$ depth of compression flange $\boxed{h_f = 7.500}$ in

$w_{tf} = 48.00$ width of top flange, inches

$$c := \frac{A_{ps} \cdot f_{pu} - \alpha_1 \cdot f'_{cd} \cdot \left(b - w_{tf}\right) \cdot h_f}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot w_{tf} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 10.178}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 8.65}$ in

This is within the depth of the haunch (9.5 inches). Therfore our assumption is OK.

Now calculate the effective tendon stress at ultimate:

$$f_{ps} := f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right)$$ $\boxed{f_{ps} = 260.05}$ ksi

$T_u := f_{ps} \cdot A_{ps}$ $\boxed{T_u = 2483}$ kips

Calculate the nominal moment capacity of the composite section in accordance with **LRFD [5.6.3.2]; [5.6.3.2.2]**

$$M_n := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + \alpha_1 \cdot f'_{cd} \cdot \left(b - w_{tf}\right) \cdot h_f \cdot \left(\frac{a}{2} - \frac{h_f}{2}\right)\right] \cdot \frac{1}{12}$$

$\boxed{M_n = 15155}$ kip-ft

For prestressed concrete, $\phi_f := 1.00$, **LRFD [5.5.4.2]**. Therefore the usable capacity is:

$M_r := \phi_f \cdot M_n$ $\boxed{M_r = 15155}$ kip-ft

The required capacity:

Interior Girder Moment $\boxed{M_{str} = 12449}$ kip-ft

Exterior Girder Moment $\boxed{M_{strx} = 11183}$ kip-ft

Check the section for minimum reinforcement in accordance with **LRFD [5.6.3.3]** for the interior girder:

$\boxed{1.33 \cdot M_{str} = 16558}$ kip-ft

$f_r = 0.24 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := 0.24 \cdot \sqrt{f'_c}$ $\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.679}$ ksi

$f_{cpe} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_b}$ $\boxed{f_{cpe} = 4.216}$ ksi

$M_{dnc} := M_{nc}$ $\boxed{M_{dnc} = 4887}$ kip-ft

$S_c := -S_{cgb}$ $\boxed{S_c = 24681}$ in^3

$S_{nc} := -S_b$ $\boxed{S_{nc} = 18825}$ in^3

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_2 := 1.1$ prestress variability factor

$\gamma_3 := 1.0$ for prestressed concrete structures

$$M_{cr} := \gamma_3 \cdot \left[S_c \cdot (\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe}) \cdot \frac{1}{12} - M_{dnc} \cdot \left(\frac{S_c}{S_{nc}} - 1 \right) \right]$$ $\boxed{M_{cr} = 10251}$ kip-ft

Is M_r greater than the lesser value of M_{cr} and $1.33{*}M_{str}$? $\boxed{check = \text{"OK"}}$

The moment capacity looks good, with some over strength for the interior girder. However, we must check the capacity of the <u>exterior girder</u> since the available flange width is less.

Check the exterior girder capacity:

The effective flange width for exterior girder is calculated in accordance with **LRFD [4.6.2.6]** as one half the effective width of the adjacent interior girder plus the overhang width :

$$w_{ex\_oh} := s_{oh} \cdot 12$$ $\boxed{w_{ex\_oh} = 30.0}$ in

$$w_{ex} := \frac{w_e}{2} + w_{ex\_oh}$$ $\boxed{w_{ex} = 75.00}$ in

$b_x := w_{ex}$ effective deck width of the compression flange.

Calculate the neutral axis location for a flanged section:

LRFD [5.6.2.2] $\boxed{\alpha_1 = 0.850}$ $\boxed{\beta_1 = 0.850}$

$$c_x := \frac{A_{ps} \cdot f_{pu} - \alpha_1 \cdot f'_{cd} \cdot (b_x - w_{tf}) \cdot h_f}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot w_{tf} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c_x = 12.76}$ in

$$a_x := \beta_1 \cdot c_x$$ $\boxed{a_x = 10.85}$ in

Now calculate the effective tendon stress at ultimate:

$$f_{ps\_x} := f_{pu} \cdot \left(1 - k \cdot \frac{c_x}{d_p} \right)$$ $\boxed{f_{ps\_x} = 257.52}$ ksi

The nominal moment capacity of the composite section (exterior girder) ignoring the increased strength of the concrete in the girder flange:

$$M_{n\_x} := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a_x}{2} \right) + \alpha_1 \cdot f'_{cd} \cdot (b_x - w_{tf}) \cdot h_f \cdot \left(\frac{a_x}{2} - \frac{h_f}{2} \right) \right] \cdot \frac{1}{12}$$

$\boxed{M_{n\_x} = 14972}$ kip-ft

$M_{r\_x} := \phi_f \cdot M_{n\_x}$ $\boxed{M_{r\_x} = 14972}$ kip-ft

$\boxed{1.33M_{strx} = 14874}$ kip-ft

Is M_{r_x} greater than $1.33*M_{strx}$? $\boxed{check = \text{"OK"}}$

Since M_{r_x} is greater than $1.33*M_{strx}$, the check for M_{cr} does not need to be completed.

E19-1.13 Shear Analysis

A separate analysis must be conducted to estimate the total shear force in each girder for shear design purposes.

Calculate the shear distribution to the girders, **LRFD [Table 4.6.2.2.3a-1]**:

Interior Beams:

One lane loaded:

$$g_{vi1} := 0.36 + \frac{S}{25} \qquad \boxed{g_{vi1} = 0.660}$$

Two or more lanes loaded:

$$g_{vi2} := 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 \qquad \boxed{g_{vi2} = 0.779}$$

$$g_{vi} := \max(g_{vi1}, g_{vi2}) \qquad \boxed{g_{vi} = 0.779}$$

Note:The distribution factors above include the multiple lane factor. The skew correction factor, is required by a WisDOT policy item for all girders.

Apply the shear magnification factor in accordance with **LRFD [4.6.2.2.3c]**.

$$skew_{correction} := 1.0 + 0.2 \cdot \left(\frac{12L \cdot t_{se}^{\,3}}{K_g}\right)^{0.3} \cdot \tan\left(skew \cdot \frac{\pi}{180}\right)$$

$\boxed{L = 146.00}$

$\boxed{t_s = 8.00}$

$\boxed{K_g = 3600866}$

$\boxed{skew = 20.000}$ $\qquad \boxed{skew_{correction} = 1.045}$

$$g_{vi} := g_{vi} \cdot skew_{correction} \qquad \boxed{g_{vi} = 0.814}$$

Exterior Beams:

Two or more lanes loaded:

The distance from the centerline of the exterior beam to the inside edge of the parapet, $d_e = 1.25$ feet.

$e_v := 0.6 + \frac{d_e}{10}$ $\boxed{e_v = 0.725}$

$g_{vx2} := e_v \cdot g_{vi}$ $\boxed{g_{vx2} = 0.590}$

With a single lane loaded, we use the lever rule (same as before). Note that the multiple presence factor has already been applied to g_{x2}..

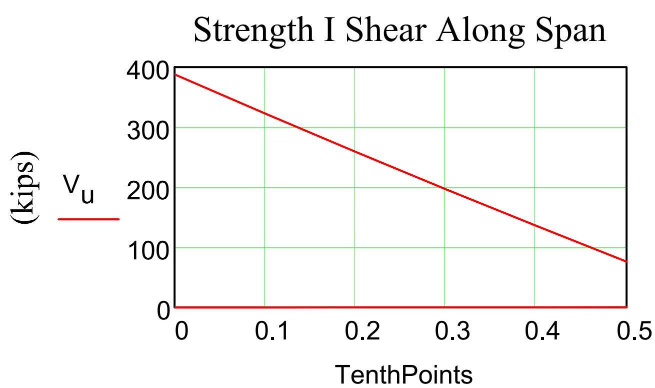
$g_{vx1} := g_{x1} \quad = e \cdot g_i$ $\boxed{g_{vx1} = 0.600}$

$g_{vx} := \max(g_{vx1}, g_{vx2})$ $\boxed{g_{vx} = 0.600}$

$g_{vx} := g_{vx} \cdot skew_{correction}$ $\boxed{g_{vx} = 0.627}$

The interior girder will control. It has a larger distribution factor and a larger dead load.

Conduct a bridge analysis as before with similar load cases for the maximum girder shear forces. We are interested in the Strength 1 condition now for shear design.



$\boxed{Vu_{0.0} = 388.1}$ kips

$\boxed{Vu_{0.5} = 76.2}$ kips

General Procedure for Prestressed Sections, **LRFD [5.7.3.4.2]**

$b_v := t_w$ $\boxed{b_v = 6.50}$ in

The critical section for shear is taken at a distance of d_v from the face of the support, **LRFD [5.7.3.2]**.

d_v = effective shear depth taken as the distance between the resultants of the tensile and compressive forces due to flexure. It need not be taken less than the greater of $0.9*d_e$ or 0.72h (inches). **LRFD [5.7.2.8]**

The first estimate of d_v is calculated as follows:

$$d_v := -e_s + y_t + hau + t_{se} - \frac{a}{2}$$ $\boxed{d_v = 72.99}$ in

However, since there are draped strands for a distance of $HD = 49.00$ feet from the end of the girder, a revised value of e_s should be calculated based on the estimated location of the critical section. Since the draped strands will raise the center of gravity of the strand group near the girder end, try a smaller value of "d_v" and recalculate "e_s" and "a".

Try $d_v := 64.50$ inches.

For the standard bearing pad of width, $w_{brg} := 8$ inches, the distance from the end of the girder to the critical section:

$$L_{crit} := \left(\frac{w_{brg}}{2} + d_v\right)\cdot\frac{1}{12} + 0.5 \qquad \boxed{L_{crit} = 6.21} \text{ ft}$$

Calculate the eccentricity of the strand group at the critical section.

$$y_{8t\_crit} := y_{8t} - \frac{slope}{100}\cdot L_{crit}\cdot 12 \qquad \boxed{y_{8t\_crit} = 24.27} \text{ in}$$

$$e_{s\_crit} := \frac{ns_s\cdot y_s + ns_d\cdot y_{8t\_crit}}{ns_s + ns_d} \qquad \boxed{e_{s\_crit} = -20.84} \text{ in}$$

Calculation of compression stress block based on revised eccentricity:

$$d_{p\_crit} := y_t + hau + t_{se} - e_{s\_crit} \qquad \boxed{d_{p\_crit} = 67.47} \text{ in}$$

$$A_{ps\_crit} := \left(ns_d + ns_s\right)\cdot A_{strand} \qquad \boxed{A_{ps\_crit} = 9.55} \text{ in}^2$$

Also, the value of f_{pu}, should be revised if the critical section is located less than the development length from the end of the beam. The development length for a prestressing strand is calculated in accordance with **LRFD [5.9.4.3.2]**:

$K := 1.6$ for prestressed members with a depth greater than 24 inches

$\boxed{d_b = 0.600}$ in

$$l_d := K\cdot\left(f_{ps} - \frac{2}{3}\cdot f_{pe}\right)\cdot d_b \qquad \boxed{l_d = 145.9} \text{ in}$$

The transfer length may be taken as: $l_{tr} := 60\cdot d_b$ $\boxed{l_{tr} = 36.00}$ in

Since $L_{crit} = 6.208$ feet is between the transfer length and the development length, the design stress in the prestressing strand is calculated as follows:

$$f_{pu\_crit} := f_{pe} + \frac{L_{crit}\cdot 12 - l_{tr}}{l_d - l_{tr}}\cdot\left(f_{ps} - f_{pe}\right) \qquad \boxed{f_{pu\_crit} = 196} \text{ ksi}$$

For rectangular section behavior:

LRFD [5.6.2.2] $\boxed{\alpha_1 = 0.850}$ $\boxed{\beta_1 = 0.850}$

$$c := \frac{A_{ps\_crit} \cdot f_{pu\_crit}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps\_crit} \cdot \frac{f_{pu\_crit}}{d_{p\_crit}}}$$ $\boxed{c = 7.002}$ in

$$a_{crit} := \beta_1 \cdot c$$ $\boxed{a_{crit} = 5.951}$ in

Calculation of shear depth based on refined calculations of e_s and a:

$$d_{v\_crit} := -e_{s\_crit} + y_t + hau + t_{se} - \frac{a_{crit}}{2}$$ $\boxed{d_{v\_crit} = 64.50}$ in

This value matches the assumed value of d_v above. OK!

The nominal shear resistance of the section is calculated as follows, **LRFD [5.8.3.3]**:

$$V_n = \min\left(V_c + V_s + V_p, 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p\right)$$

The nominal shear resistance of the concrete is calculated as follows:

$$V_c = 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$$

where:

$$\beta = \frac{4.8}{1 + 750 \cdot \varepsilon_s}$$

$$\varepsilon_s = \frac{\frac{|M_u|}{d_v} \cdot 0.5 \cdot N_u + |V_u - V_p| - A_{ps} \cdot f_{po}}{E_s \cdot A_s + E_p \cdot A_{ps}}$$

ε_s = Net longitudinal tensile strain in the section at the centroid of the tension reinforcement.

I M_u I = Absolute value of the factored moment at the section, not taken less than I V_u-V_p I d_v (kip-in)

N_u = Factored axial force, taken as positive if tensile and negative if compression (kips).

V_p = Componet of prestressing force in the direction of the shear force positive if resisiting the applied shear(kips)

f_{po} = A parameter taken as modulus of elasticity of prestressing steel multiplied by the locked-in difference in strain between the prestressing steel and the surrounding concrete (ksi).

Values at the critical section, $L_{crit} = 6.21$ feet from the end of the girder at the abutment, are as follows:

$d_v = 64.50$

$N_u := 0$ kips

$V_u \equiv 360.4$ kips

$V_p := ns_d \cdot A_{strand} \cdot f_{pe} \cdot \frac{slope}{100}$ $\quad V_p = 29.68$ kips

$f_{po} := 0.70 \cdot f_{pu}$ $\quad f_{po} = 189.00$ ksi

$M_u = \max(M_{u1}, M_{u2}) \cdot 12$

$M_{u1} := 1880.2$ kip-ft

$M_{u2} := \left|V_u - V_p\right| \cdot \frac{d_v}{12} = 1777.6$ kip-ft

$M_u := \max(M_{u1}, M_{u2}) \cdot 12 = 22562.40$ kip-in

$A_{ps} \equiv 5.78$ area of prestressing steel on the flexural tension side, in^2

$A_s := 0.0$ area of nonprestressing steel on the flexural tension side, in^2

$A_{ct} := 505.8$ area of concrete on the flexural tension side, in^2

Calculation of net longitudinal tensile strain at the centroid of the tension reinforcement per **LRFD [5.7.3.4.2]**:

$$\varepsilon_{s1} := \frac{\frac{\left|M_u\right|}{d_v} + 0.5 \cdot N_u + \left|V_u - V_p\right| - A_{ps} \cdot f_{po}}{E_s \cdot A_s + E_p \cdot A_{ps}} \qquad \varepsilon_{s1} = -0.0025$$

Since the value is negative, recalculate the strain value using the concrete term shown below:

$$\varepsilon_{s2} := \frac{\frac{\left|M_u\right|}{d_v} + 0.5 \cdot N_u + \left|V_u - V_p\right| - A_{ps} \cdot f_{po}}{E_s \cdot A_s + E_p \cdot A_{ps} + E_c \cdot A_{ct}} \qquad \varepsilon_{s2} = -0.000122$$

Strain limits: $-0.0004 < \varepsilon_s < 0.006$

$$\varepsilon_s := \begin{cases} \min(\varepsilon_{s1}, 0.006) & \text{if } \varepsilon_{s1} > 0 \\ \max(\varepsilon_{s2}, -0.00040) & \text{otherwise} \end{cases} \qquad \varepsilon_s = -0.000122$$

$$\beta := \frac{4.8}{1 + 750 \cdot \varepsilon_s} \qquad \beta = 5.283$$

Calculate the nominal shear resistance of the concrete:

$$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_c} \cdot b_v \cdot d_v \qquad V_c = 198.0 \text{ kips}$$

Calculate the required shear resistance:

$\phi_v := 0.9$ **LRFD [5.5.4.2]**

$$V_{u\_crit} = \gamma st_{DC} \cdot (V_{DCnc} + V_{DCc}) + \gamma st_{DW} \cdot V_{DWc} + \gamma st_{LL} \cdot Vu_{LL}$$

where:

$V_{DCnc} = 123.43$ kips

$V_{DCc} = 8.68$ kips

$V_{DWc} = 8.97$ kips

$Vu_{LL} = 105.05$ kips

$V_{u\_crit} = 362.4$ kips $V_n := \frac{V_{u\_crit}}{\phi_v}$ $\boxed{V_n = 402.7}$ kips

Transverse Reinforcing Design at Critical Section:

The required steel capacity:

$V_s := V_n - V_c - V_p$ $\boxed{V_s = 175.1}$ kips

$A_v := 0.40$ in^2 for #4 rebar

$f_y := 60$ ksi

Transverse Reinforcing Design at Critical Section:

$\boxed{d_v = 64.50}$ in

$\theta := 29 + 3500 \cdot \varepsilon_s$ **LRFD [5.7.3.4.2]** $\theta = 28.573$

$\cot\theta = 1.836$

$V_s = A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s}$ **LRFD [Eq 5.7.3.3-4]** reduced when α = 90 degrees.

$s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{V_s}$ $\boxed{s = 16.24}$ in

Check Maximum Spacing, **LRFD [5.7.2.6]**:

$v_u := \frac{V_{u\_crit}}{\phi_v \cdot b_v \cdot d_v}$ $\boxed{v_u = 0.961}$ ksi

Max. stirrup spacing per WisDOT policy item is 18" $\boxed{0.125 \cdot f'_c = 1.000}$ ksi

$$s_{max1} := \begin{cases} \min(0.8 \cdot d_v, 18) & \text{if } v_u < 0.125 \cdot f'_c \\ \min(0.4 \cdot d_v, 12) & \text{if } v_u \geq 0.125 \cdot f'_c \end{cases}$$ $\boxed{s_{max1} = 18.00}$ in

Check Minimum Reinforcing, **LRFD [5.7.2.5]**:

$$s_{max2} := \frac{A_v \cdot f_y}{0.0316 \cdot \lambda \sqrt{f'_c} \cdot b_v}$$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{s_{max2} = 41.31}$ in

$$s_{max} := \min(s_{max1}, s_{max2})$$ $\boxed{s_{max} = 18.00}$ in

The critical section for shear is located within the predetermined stirrup spacing provided on the Standard Detail.

Therefore use the maximum spacing of $s := 16.0$ inches.

$$V_s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s}$$ $\boxed{V_s = 177.7}$ kips

Check V_n requirements:

$$Vn_1 := V_c + V_s + V_p$$ $\boxed{Vn_1 = 405.3}$ kips

$$Vn_2 := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$$ $\boxed{Vn_2 = 868.2}$ kips

$$V_n := \min(Vn_1, Vn_2)$$ $\boxed{V_n = 405.3}$ kips

$$V_r := \phi_v \cdot V_n$$ $\boxed{V_r = 364.8}$ kips

$\boxed{V_{u\_crit} = 362.4}$ kips

Is V_{u_crit} less than V_r? $\boxed{check = \text{"OK"}}$

Web reinforcing is required in accordance with **LRFD [5.7.2.3]** whenever:

$$V_u \geq 0.5 \cdot \phi_v \cdot (V_c + V_p)$$ (all values shown are in kips)

At critical section from end of girder: $\boxed{V_{u\_crit} = 362.4}$ $\boxed{0.5 \cdot \phi_v \cdot (V_c + V_p) = 102.4}$

From calculations similar to those shown above,

At hold down point: $\boxed{V_{u\_hd} = 177.2}$ $\boxed{0.5 \cdot \phi_v \cdot (V_{c\_hd} + V_{p\_hd}) = 62.6}$

At mid-span: $\boxed{V_{u\_mid} = 76.2}$ $\boxed{0.5 \cdot \phi_v \cdot (V_{c\_mid} + V_{p\_mid}) = 36.2}$

Therefore, use web reinforcing over the entire beam.

Resulting Shear Design:

Use #4 U shaped stirrups at 18-inch spacing between the typical end sections. Unless a large savings in rebar can be realized, use a single stirrup spacing between the standard end sections.

E19-1.14 Longitudinal Tension Flange Capacity:

The tensile capacity of the longitudinal reinforcement must meet the requirements of **LRFD [5.7.3.5].**

The tensile force is checked at the critical section for shear:

The values of M_u, d_v, V_u, V_s, V_p and θ are taken at the location of the critical section. $N_u = 0$

$$T_{ps\_crit} = \frac{|M_u|}{d_v \cdot \phi_f} + \frac{0.5 \cdot N_u}{\phi_v} + \left(\left| \frac{V_u}{\phi_f} - V_p \right| - 0.5 \cdot V_s \right) \cdot \cot\theta \qquad \boxed{T_{ps\_crit} = 798.1} \text{ kips}$$

actual capacity of the straight strands:

$$\boxed{ns_s \cdot A_{strand} \cdot f_{pu\_crit} = 1534.6} \text{ kips}$$

Is the capacity of the straight strands greater than T_{ps}? $\boxed{check = \text{"OK"}}$

The tensile force is checked at the edge of the bearing:

The strand is anchored $l_{px} := 10$ inches. The transfer and development lengths for a prestressing strand are calculated in accordance with **LRFD [5.9.4.3.2]**:

$$\boxed{l_{tr} = 36.00} \text{ in}$$

$$\boxed{l_d = 145.9} \text{ in}$$

Since l_{px} is less than the transfer length, the design stress in the prestressing strand is calculated as follows:

The assumed crack plane crosses the centroid of the straight strands at

$$l_{px'} := l_{px} + Y_s \cdot \cot\theta \qquad \boxed{Y_s = 4.00} \text{ in} \qquad \boxed{l_{px'} = 17.34} \text{ in}$$

$$f_{pb} := \frac{f_{pe} \cdot l_{px'}}{60 \cdot d_b} \qquad \boxed{f_{pb} = 78.12} \text{ ksi}$$

Tendon capacity of the straight strands: $\boxed{ns_s \cdot A_{strand} \cdot f_{pb} = 610.2}$ kips

The values of V_u, V_s, V_p and θ may be taken at the location of the critical section.

Over the length d_v, the average spacing of the stirrups is:

$$s_{ave} := \frac{6 \cdot 4.25 + 6 \cdot 5.0}{12} \qquad \boxed{s_{ave} = 4.63} \text{ in}$$

$$V_s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s_{ave}} \qquad \boxed{V_s = 614.6} \text{ kips}$$

The vertical component of the draped strands is: $\boxed{V_p = 29.7}$ kips

The factored shear force at the critical section is: $V_u = 360.4$ kips

Minimum capacity required at the front of the bearing:

$$T_{breqd} := \left(\frac{V_u}{\phi_v} - 0.5 \cdot V_s - V_p\right) \cdot \cot\theta$$ $T_{breqd} = 116.6$ kips

Is the capacity of the straight strands greater than T_{breqd}? check = "OK"

E19-1.15 Composite Action - Design for Interface Shear Transfer

The total shear to be transferred to the flange between the end of the beam and mid-span is equal to the compression force in the compression block of the flange and haunch in strength condition. For slab on girder bridges, the shear interface force is calculated in accordance with **LRFD [5.7.4.5]**.

$b_{vi} := 18$ in width of top flange available to bond to the deck

$d_v = 64.50$ in

$$v_{ui} := \frac{V_{u\_crit}}{b_{vi} \cdot d_v}$$ $v_{ui} = 0.312$ ksi

$V_{ui} := v_{ui} \cdot 12 \cdot b_{vi}$ $V_{ui} = 67.4$ kips/ft

$V_n = c \cdot A_{cv} + \mu \cdot (A_{vf} \cdot f_y + P_c)$ **LRFD [5.7.4.3]**

The nominal shear resistance, V_n, used in design shall not be greater than the lesser of:

$V_{n1} = K_1 \cdot f'_{cd} \cdot A_{cv}$ or $V_{n2} = K_2 \cdot A_{cv}$

$c := 0.28$ ksi

$\mu := 1.0$

$K_1 := 0.3$

$K_2 := 1.8$

$A_{cv} := b_{vi} \cdot 12$ Area of concrete considered to be engaged in interface shear transfer. $A_{cv} = 216$ in^2/ft

$P_c := 0.0$ kips/ft Conservatively set the permanent net compressive force normal to the shear plane to zero.

From earlier calculations, the maximum #4 stirrup spacing used is $s = 18.0$ inches.

$A_{vf} := \frac{A_v}{s} \cdot 12$ $\boxed{A_{vf} = 0.267}$ in^2/ft

$V_n := c \cdot A_{cv} + \mu \cdot \left(A_{vf} \cdot f_y + P_c\right)$ $\boxed{V_n = 76.5}$ kips/ft

$V_{n1} := K_1 \cdot f'_{cd} \cdot A_{cv}$ $\boxed{V_{n1} = 259.2}$ kips/ft

$V_{n2} := K_2 \cdot A_{cv}$ $\boxed{V_{n2} = 388.8}$ kips/ft

$V_n := \min\left(V_n, V_{n1}, V_{n2}\right)$ $\boxed{V_n = 76.5}$ kips/ft

$V_r := \phi_v \cdot V_n$ $\boxed{V_r = 68.8}$ kips/ft

$\boxed{V_{ui} = 67.4}$ kips/ft

Is V_r greater than V_{ui}? $\boxed{check = \text{"OK"}}$

Solution:
#4 stirrups spaced at $s = 18.0$ inches is adequate to develop the required interface shear resistance for the entire length of the girder.

E19-1.16 Deflection Calculations

Check the Live Load deflection with the vehicle loading as specified in **LRFD [3.6.1.3.2]**; design truck alone or 25% of the design truck + the lane load.

The deflection shall be limited to L/800.

The moment of inertia of the entire bridge shall be used.

$$\Delta_{limit} := \frac{L \cdot 12}{800}$$ $\boxed{\Delta_{limit} = 2.190}$ inches

$I_{cg} = 1203475.476$

$ng = 6$ number of girders

$I_{bridge} := I_{cg} \cdot ng$ $\boxed{I_{bridge} = 7220853}$ in^4

From CBA analysis with 3 lanes loaded, the truck deflection controlled:

$\Delta_{truck} := 0.648$ in

Applying the multiple presence factor from **LRFD Table [3.6.1.1.2-1]** for 3 lanes loaded:

$\Delta := 0.85 \cdot \Delta_{truck}$ $\boxed{\Delta = 0.551}$ in

Is the actual deflection less than the allowable limit, $\Delta < \Delta$ limit? $\boxed{check = \text{"OK"}}$

E19-1.17 Camber Calculations

Moment due to straight strands:

Number of straight strands: $\boxed{ns_s = 36}$

Eccentricity of the straight strands: $\boxed{y_s = -30.87}$ in

$P_{i\_s} := ns_s \cdot A_{strand} \cdot (f_{tr} - \Delta f_{pES})$ $\boxed{P_{i\_s} = 1448}$ kips

$M_1 := P_{i\_s} \cdot |y_s|$ $\boxed{M_1 = 44698}$ kip-in

Upward deflection due to straight strands:

Length of the girder: $\boxed{L_g = 147}$ ft

Modulus of Elasticity of the girder at release: $\boxed{E_{ct} = 4999}$ ksi

Moment of inertia of the girder: $\boxed{I_g = 656426}$ in^4

$$\Delta_s := \frac{M_1 \cdot L_g^{\,2}}{8 \cdot E_{ct} \cdot I_g} \cdot 12^2$$ $\boxed{\Delta_s = 5.298}$ in

Moment due to draped strands:

$P_{i\_d} := ns_d \cdot A_{strand} \cdot \left(f_{tr} - \Delta f_{pES}\right)$ $\boxed{P_{i\_d} = 321.8}$ kips

$\boxed{A = 67.00}$ in

$\boxed{C = 5.00}$ in

$M_2 := P_{i\_d} \cdot (A - C)$ $\boxed{M_2 = 19949.4}$ kip-in

$M_3 := P_{i\_d} \cdot \left(A - \left|y_b\right|\right)$ $\boxed{M_3 = 10338.3}$ kip-in

Upward deflection due to draped strands:

$$\Delta_d := \frac{L_g^{\,2}}{8 \cdot E_{ct} \cdot I_g} \cdot \left(\frac{23}{27} \cdot M_2 - M_3\right) \cdot 12^2$$ $\boxed{\Delta_d = 0.789}$ in

Total upward deflection due to prestress:

$\Delta_{PS} := \Delta_s + \Delta_d$ $\boxed{\Delta_{PS} = 6.087}$ in

Downward deflection due to beam self weight at release:

$$\Delta_{gi} := \frac{5 \cdot w_g \cdot L^4}{384 \cdot E_{ct} \cdot I_g} \cdot 12^3$$ $\boxed{\Delta_{gi} = 2.969}$ in

Anticipated prestress camber at release:

$\Delta_i := \Delta_{PS} - \Delta_{gi}$ $\boxed{\Delta_i = 3.118}$ in

The downward deflection due to the dead load of the deck and diaphragms:

Calculate the additional non-composite dead loads for an interior girder:

$w_{nc} := w_{dlii} - w_g$ $\boxed{w_{nc} = 0.881}$ klf

Modulus of Elasticity of the beam at final strength $\boxed{E_B = 6351}$ ksi

$$\Delta_{nc} := \frac{5 \cdot w_{nc} \cdot L^4}{384 \cdot E_B \cdot I_g} \cdot 12^3$$ $\boxed{\Delta_{nc} = 2.161}$ in

The downward deflection due to the dead load of the parapets is calculated as follows. Note that the deflections due to future wearing surface loads are not considered.

Calculate the composite dead loads for an interior girder:

$w_{ws} := 0$ klf

$w_c := w_p + w_{ws}$ $\boxed{w_c = 0.129}$ klf

$$\Delta_c := \frac{5 \cdot w_c \cdot L^4}{384 \cdot E_B \cdot I_{cg}} \cdot 12^3 \qquad \boxed{\Delta_c = 0.173} \quad \text{in}$$

The total downward deflection due to dead loads acting on an interior girder:

$$\Delta_{DL} := \Delta_{nc} + \Delta_c \qquad \boxed{\Delta_{DL} = 2.334} \quad \text{in}$$

The residual camber for an interior girder:

The anticipated prestress camber at release shall be multiplied by a camber multiplier (1.4) for calculating haunch heights.

$$RC := 1.4 \cdot \Delta_i - \Delta_{DL} \qquad \boxed{RC = 2.031} \quad \text{in}$$

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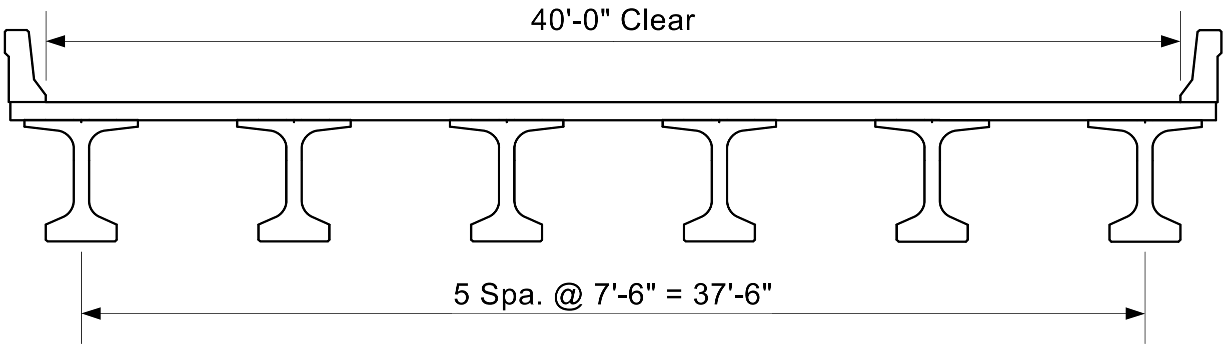
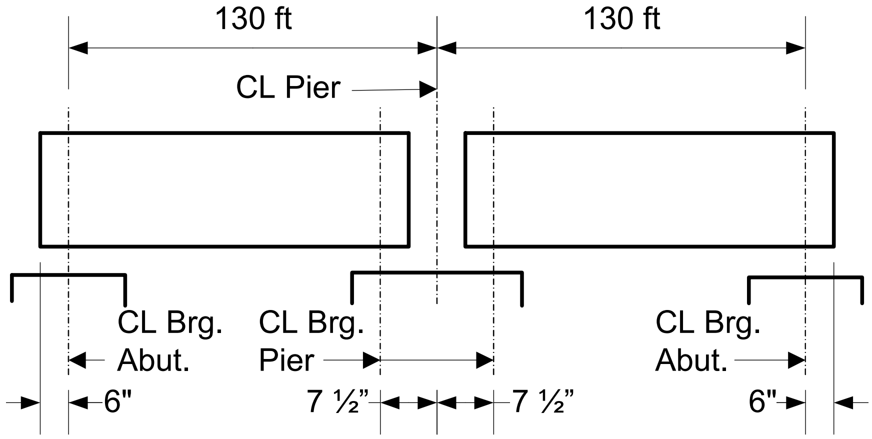
E19-2 Two-Span 54W" Girder, Continuity Reinforcement - LRFD

This example shows design calculations for the continuity reinforcement for a two span prestressed girder bridge. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter.***(Example is current through LRFD Eighth Ed. - 2017)***

E19-2.1 Design Criteria





$L := 130$	center of bearing at abutment to CL pier for each span, ft
$L_g := 130.375$	total length of the girder (the girder extends 6 inches past the center of bearing at the abutment and 1.5" short of the center line of the pier).
$w_b := 42.5$	out to out width of deck, ft
$w := 40$	clear width of deck, 2 lane road, 3 design lanes, ft
$f'_c := 8$	girder concrete strength, ksi
$f'_{cd} := 4$	deck concrete strength, ksi
$f_y := 60$	yield strength of mild reinforcement, ksi

$w_p := 0.387$ — weight of Wisconsin Type LF parapet, klf

$t_s := 8$ — slab thickness, in

$t_{se} := 7.5$ — effective slab thickness, in

$skew := 0$ — skew angle, degrees

$w_c := 0.150$ — kcf

$E_s := 29000$ — ksi, Modulus of Elasticity of the reinforcing steel

E19-2.2 Modulus of Elasticity of Beam and Deck Material

Based on past experience, the modulus of elasticity for the precast and deck concrete are given in Chapter 19 as $E_{beam6} := 5500$ ksi and $E_{deck4} := 4125$ ksi for concrete strengths of 6 and 4 ksi respectively. The values of E for different concrete strengths are calculated as follows (ksi):

$$E_{beam8} := 5500 \cdot \frac{\sqrt{f'_c \cdot 1000}}{\sqrt{6000}}$$

$\boxed{E_{beam8} = 6351}$

$E_B := E_{beam8}$

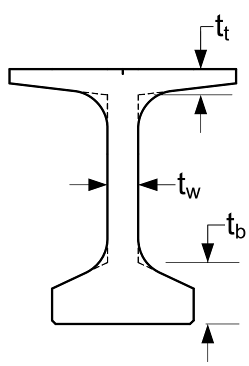
$E_D := E_{deck4}$

$$n := \frac{E_B}{E_D}$$

$\boxed{n = 1.540}$

E19-2.3 Section Properties

54W Girder Properties:



$w_{tf} := 48$ — in

$t_t := 4.625$ — in

$t_w := 6.5$ — in

$t_b := 10.81$ — in

$ht := 54$ — in

$b_w := 30$ — width of bottom flange, in

$A_g := 798$ — in^2

$I_g := 321049$ — in^4

$y_t := 27.70$ — in

$y_b := -26.30$ — in

$$e_g := y_t + 2 + \frac{t_{se}}{2}$$

$\boxed{e_g = 33.45}$ in

$S_t := 11592$ in^3

$S_b := -12205$ in^3

LRFD [Eq 4.6.2.2.1-1]

$K_g := n \cdot \left(I_g + A_g \cdot e_g^{\,2}\right)$ $\boxed{K_g = 1868972}$ in^4

E19-2.4 Girder Layout

Chapter 19 suggests that at a 130 foot span, the girder spacing should be 7'-6" with 54W girders.

$S := 7.5$ ft

Assume a minimum overhang of 2.5 feet (2 ft flange + 6" overhang), $s_{oh} := 2.5$

$$ns := \frac{w_b - s_{oh}}{S} \qquad \boxed{ns = 5.333}$$

Use the lowest integer: $ns := \text{floor}(ns)$ $\boxed{ns = 5}$

Number of girders: $ng := ns + 1$ $\boxed{ng = 6}$

Overhang Length: $s_{oh} := \dfrac{w_b - S \cdot ns}{2}$ $\boxed{s_{oh} = 2.50}$ ft

E19-2.5 Loads

$w_g := 0.831$	weight of 54W girders, klf
$w_d := 0.100$	weight of 8-inch deck slab (interior), ksf
$w_h := 0.100$	weight of 2-in haunch, klf
$w_{di} := 0.410$	weight of diaphragms on interior girder (assume 2), kips
$w_{dx} := 0.205$	weight of diaphragms on exterior girder, kips
$w_{ws} := 0.020$	future wearing surface, ksf
$w_p = 0.387$	weight of parapet, klf

E19-2.5.1 Dead Loads

Dead load on non-composite (DC):

exterior:

$$w_{dlxi} := w_g + w_d \cdot \left(\frac{S}{2} + s_{oh}\right) + w_h + 2 \cdot \frac{w_{dx}}{L} \qquad \boxed{w_{dlxi} = 1.559} \text{ klf}$$

interior:

$$w_{dlii} := w_g + w_d \cdot S + w_h + 2 \cdot \frac{w_{di}}{L}$$ $\boxed{w_{dlii} = 1.687}$ klf

* Dead load on composite (DC):

$$w_p := \frac{2 \cdot w_p}{ng}$$ $\boxed{w_p = 0.129}$ klf

* Wearing Surface (DW):

$$w_{ws} := \frac{w \cdot w_{ws}}{ng}$$ $\boxed{w_{ws} = 0.133}$ klf

* **LRFD [4.6.2.2.1]** states that permanent loads on the deck may be distributed uniformly among the beams. This method is used for the parapet and future wearing surface loads.

E19-2.5.2 Live Loads

For Strength 1 and Service 1:

HL-93 loading = truck + lane **LRFD [3.6.1.3.1]**

truck pair + lane

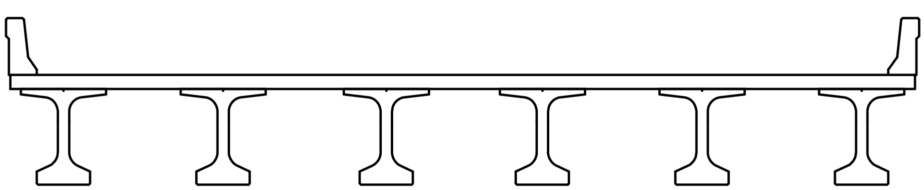
DLA of 33% applied to truck or tandem, but not to lane per **LRFD [3.6.2.1].**

For Fatigue 1:

HL-93 truck (no lane) with 15% DLA and 30 ft rear axle spacing per **LRFD [3.6.1.4.1].**

E19-2.6 Load Distribution to Girders

In accordance with **LRFD [Table 4.6.2.2.1-1]**, this structure is a Type "K" bridge.



Distribution factors are in accordance with **LRFD [Table 4.6.2.2.2b-1]**. For an interior beam, the distribution factors are shown below:

For one Design Lane Loaded:

$$0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1}$$

For Two or More Design Lanes Loaded:

$$0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1}$$

Criteria for using distribution factors - Range of Applicability per **LRFD [Table 4.6.2.2.2b-1]**.

$$DeckSpan := \begin{cases} \text{"OK"} & \text{if } 3.5 \le S \le 16 \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$DeckThickness := \begin{cases} \text{"OK"} & \text{if } 4.5 \le t_{se} \le 12 \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$BridgeSpan := \begin{cases} \text{"OK"} & \text{if } 20 \le L \le 240 \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$NoBeams := \begin{cases} \text{"OK"} & \text{if } ng \ge 4 \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$LongitStiffness := \begin{cases} \text{"OK"} & \text{if } 10000 \le K_g \le 7000000 \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$x := \begin{pmatrix} S & DeckSpan \\ t_{se} & DeckThickness \\ L & BridgeSpan \\ ng & NoBeams \\ K_g & LongitStiffness \end{pmatrix} \qquad \boxed{x = \begin{pmatrix} 7.5 & \text{"OK"} \\ 7.5 & \text{"OK"} \\ 130.0 & \text{"OK"} \\ 6.0 & \text{"OK"} \\ 1868972.4 & \text{"OK"} \end{pmatrix}}$$

E19-2.6.1 Distribution Factors for Interior Beams:

One Lane Loaded:

$$g_{i1} := 0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1} \qquad \boxed{g_{i1} = 0.427}$$

Two or More Lanes Loaded:

$$g_{i2} := 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{3}}\right)^{0.1}$$ $\boxed{g_{i2} = 0.619}$

$$g_i := \max(g_{i1}, g_{i2})$$ $\boxed{g_i = 0.619}$

Note: The distribution factors above already have a multiple lane factor included that is used for service and strength limit states. The distribution factor for One Lane Loaded should be used for the fatigue vehicle and the 1.2 multiple presence factor should be divided out.

E19-2.6.2 Distribution Factors for Exterior Beams:

Two or More Lanes Loaded:

Per **LRFD [Table 4.6.2.2.2d-1]** the distribution factor shall be calculated by the following equations:

$$w_{parapet} := \frac{w_b - w}{2}$$ Width of parapet overlapping the deck $\boxed{w_{parapet} = 1.250}$ ft

$$d_e := s_{oh} - w_{parapet}$$ Distance from the exterior web of exterior beam to the interior edge of parapet, ft. $\boxed{d_e = 1.250}$ ft

Note: Conservatively taken as the distance from the center of the exterior girder.

Check range of applicability for de:

$$d_{e\_check} := \begin{cases} \text{"OK"} & \text{if } -1.0 \le d_e \le 5.5 \\ \text{"NG"} & \text{otherwise} \end{cases}$$ $\boxed{d_{e\_check} = \text{"OK"}}$

Note: While AASHTO allows the d_e value to be up to 5.5, the deck overhang (from the center of the exterior girder to the edge of the deck) is limited by WisDOT policy as stated in Chapter 17 of the Bridge Manual.

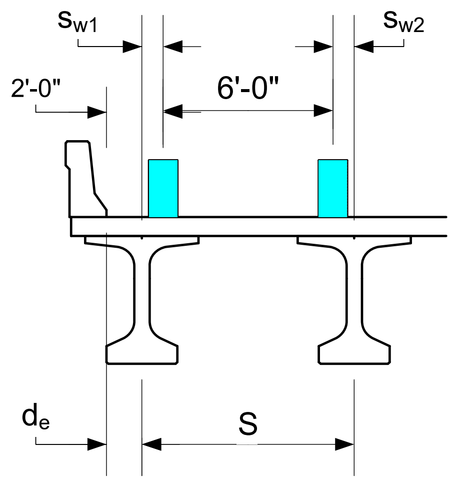
$$e := 0.77 + \frac{d_e}{9.1}$$ $\boxed{e = 0.907}$

$$g_{x1} := e \cdot g_i$$ $\boxed{g_{x1} = 0.562}$

One Lane Loaded:

Per **LRFD [Table 4.6.2.2.2d-1]** the distribution factor shall be calculated by the Lever Rule.

Calculate the distribution factor by the Lever Rule:



$s_{w1} := d_e - 2$	Distance from center of exterior girder to outside wheel load, ft.	$\boxed{s_{w1} = -0.75}$	ft
$s_{w2} := S + s_{w1} - 6$	Distance from wheel load to first interior girder, ft.	$\boxed{s_{w2} = 0.75}$	ft
$R_x := \dfrac{S + s_{w1} + s_{w2}}{S \cdot 2}$		$\boxed{R_x = 0.500}$	% of a lane load

Add the single lane multi-presence factor, $m := 1.2$

$g_{x2} := R_x \cdot 1.2$ $\quad\boxed{g_{x2} = 0.600}$

The exterior girder distribution factor is the maximum value of the One Lane Loaded case and the Two or More Lanes Loaded case:

$g_x := \max(g_{x1}, g_{x2})$ $\quad\boxed{g_x = 0.600}$

Note: The interior girder has a larger live load distribution factor and a larger dead load than the exterior girder. Therefore, for this example, the interior girder is likely to control.

E19-2.7 Load Factors

From **LRFD [Table 3.4.1-1]**:

	DC	DW	LL
Strength 1	$\gamma st_{DC} := 1.25$	$\gamma st_{DW} := 1.50$	$\gamma st_{LL} := 1.75$
Service 1	$\gamma s1_{DC} := 1.0$	$\gamma s1_{DW} := 1.0$	$\gamma s1_{LL} := 1.0$
Fatigue 1	$\gamma f_{DC} := 1.0$	$\gamma f_{DW} := 1.0$	$\gamma f_{LL} := 1.75$

Impact factor (DLA) is applied to the truck and tandem.

E19-2.8 Dead Load Moments

The unfactored dead load moments are listed below (values are in kip-ft):

Unfactored Dead Load Interior Girder Moments (ft-kips)			
Tenth Point	DC non-composite	DC composite	DW composite
0.5	3548	137	141
0.6	3402	99	102
0.7	2970	39	40
0.8	2254	-43	-45
0.9	1253	-147	-151
1.0	0	-272	-281

The DC_{nc} values are the component non-composite dead loads and include the weight of the girder, haunch, diaphragms and the deck.

The DC_c values are the component composite dead loads and include the weight of the parapets.

The DW_c values are the composite dead loads from the future wearing surface.

Note that the girder dead load moments (a portion of DC_{nc}) are calculated based on the CL bearing to CL bearing length. The other DC_{nc} moments are calculated based on the span length (center of bearing at the abutment to centerline of the pier).

E19-2.9 Live Load Moments

The unfactored live load moments (per lane including impact) are listed below (values are in kip-ft). Note that the impact factor is applied only to the truck portion of the HL-93 loads. A separate analysis run will be required if results without impact are desired.

Unfactored Live Load + Impact Moments per Lane (kip-ft)				
Tenth Point	Truck Pair	Truck + Lane	- Fatigue	+ Fatigue
0.5	--	-921	-476	1644
0.6	--	-1106	-572	1497
0.7	--	-1290	-667	1175
0.8	-1524	-1474	-762	718
0.9	-2046	-1845	-857	262
1	-3318	-2517	-953	0

The unfactored live load moments per lane are calculated by applying the appropriate distribution factor to the controlling moment. For the interior girder:

$$g_i = 0.619$$

$$M_{LL} = g_i \cdot -3317.97 \qquad \boxed{M_{LL} = -2055} \quad \text{kip-ft}$$

The single lane distribution factor should be used and the multiple presence factor of 1.2 must be removed from the fatigue moments.

$$M_{LLfatigue} = g_{i1} \cdot -952.64 \cdot \frac{1}{1.2} \qquad \boxed{M_{LLfatigue} = -339} \quad \text{kip-ft}$$

E19-2.10 Factored Moments

The factored moments for each limit state are calculated by applying the appropriate load factor to the girder moments. For the interior girder:

Strength 1

$$M_u := \eta \cdot \left(\gamma st_{DC} \cdot M_{DCc} + \gamma st_{DW} \cdot M_{DWc} + \gamma st_{LL} \cdot M_{LL}\right)$$

$$= 1.0 \cdot \left(1.25 \cdot M_{DCc} + 1.50 \cdot M_{DWc} + 1.75 \cdot M_{LL}\right) \qquad \boxed{M_u = -4358} \quad \text{kip-ft}$$

Service 1 (for compression checks in prestress and crack control in deck)

$$M_{s1} := \eta \cdot \left(\gamma s1_{DC} \cdot M_{DCc} + \gamma s1_{DW} \cdot M_{DWc} + \gamma s1_{LL} \cdot M_{LL}\right)$$

$$= 1.0 \cdot \left(1.0 \cdot M_{DCc} + 1.0 \cdot M_{DWc} + 1.0 \cdot M_{LL}\right) \qquad \boxed{M_{s1} = -2608} \quad \text{kip-ft}$$

Fatigue 1

$$M_f := \eta \cdot \left(\gamma f_{DC} \cdot M_{DCc} + \gamma f_{DW} \cdot M_{DWc} + \gamma f_{LL} \cdot M_{LLfatigue} \right)$$

$$= 1.0 \cdot \left(1.0 \cdot M_{DCc} + 1.0 \cdot M_{DWc} + 1.75 \cdot M_{LLfatigue} \right) \qquad \boxed{M_f = -1147} \text{ kip-ft}$$

$$Mf_{DL} := \eta \cdot \left(\gamma f_{DC} \cdot M_{DCc} + \gamma f_{DW} \cdot M_{DWc} \right) \qquad \boxed{Mf_{DL} = -553} \text{ kip-ft}$$

$$Mf_{LL} := \eta \cdot \gamma f_{LL} \cdot M_{LLfatigue} \qquad \boxed{Mf_{LL} = -594} \text{ kip-ft}$$

E19-2.11 Composite Girder Section Properties

Calculate the effective flange width in accordance with Chapter 17.2.11.

$$w_e := S \cdot 12 \qquad \boxed{w_e = 90.00} \text{ in}$$

The effective width, w_e, must be adjusted by the modular ratio, $n = 1.54$, to convert to the same concrete material (modulus) as the girder.

$$w_{eadj} := \frac{w_e}{n} \qquad \boxed{w_{eadj} = 58.46} \text{ in}$$

Calculate the composite girder section properties:

effective slab thickness; $\boxed{t_{se} = 7.50}$ in

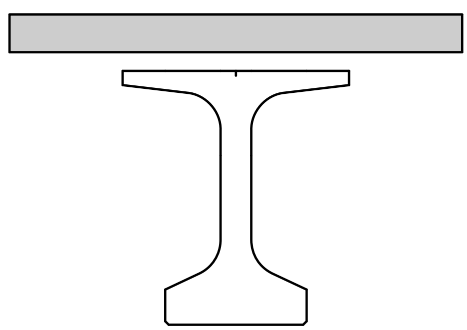
effective slab width; $\boxed{w_{eadj} = 58.46}$ in

haunch thickness; $hau := 2.00$ in

total height; $h_c := ht + hau + t_{se}$

$\boxed{h_c = 63.50}$ in

$\boxed{n = 1.540}$



Note: The area of the concrete haunch is not included in the calculation of the composite section properties.

Component	Ycg	A	AY	AY^2	I	$I+AY^2$
Deck	59.75	438	26197	1565294	2055	1567349
Girder	26.3	798	20987	551969	321049	873018
Haunch	55	0	0	0	0	0
Summation		1236	47185			2440367

$\Sigma A := 1236$ in^2

$\Sigma AY := 47185$ in^4

$\Sigma IplusAYsq := 2440367$ in^4

$$y_{cgb} := \frac{-\Sigma AY}{\Sigma A} \qquad \boxed{y_{cgb} = -38.2} \text{ in}$$

$$y_{cgt} := ht + y_{cgb} \qquad \boxed{y_{cgt} = 15.8} \text{ in}$$

$A_{cg} := \Sigma A$ in^2

$$I_{cg} := \Sigma IplusAYsq - A_{cg} \cdot y_{cgb}^{\;2} \qquad \boxed{I_{cg} = 639053} \text{ in}^4$$

Deck:

$$S_c := n \cdot \frac{I_{cg}}{y_{cgt} + hau + t_{se}} \qquad \boxed{S_c = 38851} \text{ in}^4$$

E19-2.12 Flexural Strength Capacity at Pier

All of the continuity reinforcement shall be placed in the top mat. Therefore the effective depth of the section at the pier is:

$cover := 2.5$ in

$bar_{trans} := 5$ (transverse bar size)

$Bar_D(bar_{trans}) = 0.625$ in (transverse bar diameter)

$Bar_{No} = 9$

$Bar_D(Bar_{No}) = 1.13$ in (Assumed bar size)

$$d_e := ht + hau + t_s - cover - Bar_D(bar_{trans}) - \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_e = 60.31}$ in

For flexure in non-prestressed concrete, $\phi_f := 0.9$.

The width of the bottom flange of the girder, $b_w = 30.00$ inches.

$$R_u := \frac{M_u \cdot 12}{\phi_f \cdot b_w \cdot d_e^2}$$ $\boxed{R_u = 0.532}$ ksi

$$\rho := 0.85 \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R_u}{0.85 \cdot f'_c}}\right)$$ $\boxed{\rho = 0.00925}$

$$A_s := \rho \cdot b_w \cdot d_e$$ $\boxed{A_s = 16.74}$ in^2

This reinforcement is distributed over the effective flange width calculated earlier, $w_e = 90.00$ inches. The required continuity reinforcement in in^2/ft is equal to:

$$As_{req} := \frac{A_s}{\frac{w_e}{12}}$$ $\boxed{As_{req} = 2.232}$ in^2/ft

From Chapter 17, Table 17.5-3, for a girder spacing of $S = 7.5$ feet and a deck thickness of $t_s = 8.0$ inches, use a longitudinal bar spacing of #4 bars at $s_{longit} := 8.5$ inches. The continuity reinforcement shall be placed at 1/2 of this bar spacing, .

#9 bars at 4.25 inch spacing provides an $\boxed{As_{prov} = 2.82}$ in^2/ft, or the total area of steel provided:

$$As := As_{prov} \cdot \frac{w_e}{12}$$ $\boxed{As = 21.18}$ in^2

Calculate the capacity of the section in flexure at the pier:

Check the depth of the compression block:

Assume $f_s = f_y$ **LRFD [5.6.2.2]** $\alpha_1 := 0.85$ (for $f'_c \leq 10.0$ ksi)

$$a := \frac{As \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c}$$ $\boxed{a = 6.228}$ in

This is within the thickness of the bottom flange height of 7.5 inches.

If $\frac{c}{d_s} \leq 0.6$ for ($f_y = 60$ ksi) **LRFD [5.6.2.1]**, the reinforcement has yielded and the assumption is correct.

LRFD [5.7.2.2] $\beta_1 := 0.65$; $c := \frac{a}{\beta_1}$ $\boxed{c = 9.582}$ in

$\frac{c}{d_s} = 0.16 < 0.6$ therefore, the reinforcement will yield

$$M_n := As \cdot f_y \cdot \left(d_e - \frac{a}{2}\right) \cdot \frac{1}{12}$$ $\boxed{M_n = 6056}$ kip-ft

$$M_r := \phi_f \cdot M_n$$ $\boxed{M_r = 5451}$ kip-ft

$\boxed{M_u = 4358}$ kip-ft

Is M_u less than M_r? $\boxed{check = "OK"}$

Check the minimum reinforcement limits in accordance with **LRFD [5.6.3.3]**:

$f_r = 0.24 \cdot \lambda \sqrt{f'_{cd}}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := 0.24 \cdot \sqrt{f'_{cd}}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.480}$ ksi

$$M_{cr} = \gamma_3(\gamma_1 \cdot f_r)S_c$$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement <u>for A615, Grade 60 reinforcement</u>

$$M_{cr} := 1.1 f_r \cdot S_c \cdot \frac{1}{12}$$ $\boxed{M_{cr} = 1709}$ kip-ft

$\boxed{1.33 \cdot M_u = 5796}$ kip-ft

Is M_r greater than the lesser value of M_{cr} and 1.33*M_u? $\boxed{check = "OK"}$

Check the Service I crack control requirements in accordance with **LRFD [5.6.7]**:

$$\rho := \frac{As}{b_w \cdot d_e}$$ $\boxed{\rho = 0.01170}$

$$n := \frac{E_s}{E_B}$$ $\boxed{n = 4.566}$

$$k := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n} - \rho \cdot n$$ $\boxed{k = 0.278}$

$j := 1 - \frac{k}{3}$ $\boxed{j = 0.907}$

Note that the value of d_c should not include the 1/2-inch wearing surface.

$d_c := cover - 0.5 + Bar_D(bar_{trans}) + \frac{Bar_D(Bar_{No})}{2}$ $\boxed{d_c = 3.19}$ in

$\boxed{M_{s1} = 2608}$ kip-ft

$f_s := \frac{M_{s1}}{As \cdot j \cdot d_e} \cdot 12 \le 0.6 f_y$ $\boxed{f_s = 27.006}$ ksi $\le 0.6 f_y$ O.K.

The height of the composite section, h, is:

$h := ht + hau + t_{se}$ $\boxed{h = 63.500}$ in

$\beta := 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$ $\boxed{\beta = 1.076}$

$\gamma_e := 0.75$ for Class 2 exposure

$S_{max} := \frac{700 \gamma_e}{\beta \cdot f_s} - 2 \cdot d_c$ $\boxed{S_{max} = 11.70}$ in

$\boxed{spa = 4.25}$ in

Is the bar spacing less than S_{max}? $\boxed{check = "OK"}$

Check the Fatigue 1 reinforcement limits in accordance with **LRFD [5.5.3]**:

$\gamma f_{LL} \cdot \Delta f \le \Delta F_{TH}$ where $\Delta F_{TH} := 26 - 22 \frac{f_{min}}{f_y}$

$\Delta F_{TH} := 26 - 0.367 \ f_{min}$ (for $f_y = 60$ ksi)

f_{min} is equal to the stress in the reinforcement due to the moments from the permanent loads combined with the Fatigue I load combination. Δf is the stress range resulting from the fatigue vehicle.

Check stress in section for determination of use of cracked or uncracked section properties:

$f_{top} := \frac{M_f}{S_c} \cdot 12$ $\boxed{f_{top} = 0.354}$ ksi

$f_{limit} := 0.095 \cdot \sqrt{f'_c}$ $\boxed{f_{limit} = 0.269}$ ksi

Therefore: $\boxed{\text{SectionProp} = \text{"Cracked"}}$

If we assume the neutral axis is in the bottom flange, the distance from cracked section neutral axis to bottom of compression flange, y_{cr}, is calculated as follows:

$$\frac{b_w \cdot y_{cr}^2}{2} = n \cdot As \cdot (d_e - y_{cr})$$

$$y_{cr} := \frac{n \cdot As}{b_w} \cdot \left(\sqrt{1 + \frac{2 \cdot b_w \cdot d_e}{n \cdot As}} - 1 \right)$$ $\boxed{y_{cr} = 16.756}$ in No Good

Assume the neutral axis is in the web:

$t_{bf\_min} := 7.5$

$t_{bf\_max} := 15$ $t_{taper} := t_{bf\_max} - t_{bf\_min}$ $\boxed{t_{taper} = 7.500}$

$t_{web} := 7$ $w_{taper} := b_w - t_w$ $\boxed{w_{taper} = 23.500}$

$$(w_{taper}) \cdot t_{bf\_min} \cdot \left(x - \frac{t_{bf\_min}}{2} \right) + t_w \cdot \frac{x^2}{2} \; ... \qquad = 0$$
$$+ \left(\frac{w_{taper} \cdot t_{taper}}{2} \right) \cdot \left(x - t_{bf\_min} - \frac{t_{taper}}{3} \right) - n \cdot As \cdot (d_e - x)$$

CG of cracked section, $\boxed{x = 17.626}$ in

Cracked section moment of inertia:

$$I_{cr} := \frac{w_{taper} \cdot t_{bf\_min}^3}{12} + w_{taper} \cdot t_{bf\_min} \cdot \left(x - \frac{t_{bf\_min}}{2} \right)^2 + \frac{t_{web} \cdot x^3}{3} \; ...$$
$$+ \frac{w_{taper} \cdot t_{taper}^3}{36} + \frac{w_{taper} \cdot t_{taper}}{2} \cdot \left(x - t_{bf\_min} - \frac{t_{taper}}{2} \right)^2 + n \cdot As \cdot (d_e - x)^2$$

$\boxed{I_{cr} = 227583}$ in^4

Distance from centroid of tension reinforcement to the cracked section neutral axis:

$y_{rb} := d_e - x$ $\boxed{y_{rb} = 42.685}$ in

$f_{min} := n \cdot \frac{M_f \cdot y_{rb}}{I_{cr}} \cdot 12$ $\boxed{f_{min} = 11.785}$ ksi

$\Delta F_{TH} := 26 - 0.367 \cdot f_{min}$ (for $f_y = 60$ ksi) $\boxed{\Delta F_{TH} = 21.675}$ ksi

$\Delta f := n \cdot \frac{|M_{LLfatigue}| \cdot y_{rb}}{I_{cr}} \cdot 12$ $\boxed{\Delta f = 3.488}$ ksi

$\boxed{\gamma f_{LL} \cdot \Delta f = 6.104}$ ksi

Is $\gamma f_{LL} \cdot \Delta f$ less than ΔF_{TH}? $\boxed{check = \text{"OK"}}$

E19-2.13 Bar Cut Offs

The first cut off is located where half of the continuity reinforcement satisfies the moment diagram. Non-composite moments from the girder and the deck are considered along with the composite moments when determining the Strength I moment envelope. (It should be noted that since the non-composite moments are opposite in sign from the composite moments in the negative moment region, the minimum load factor shall be applied to the non-composite moments.) Only the composite moments are considered when checking the Service and Fatigue requirements.

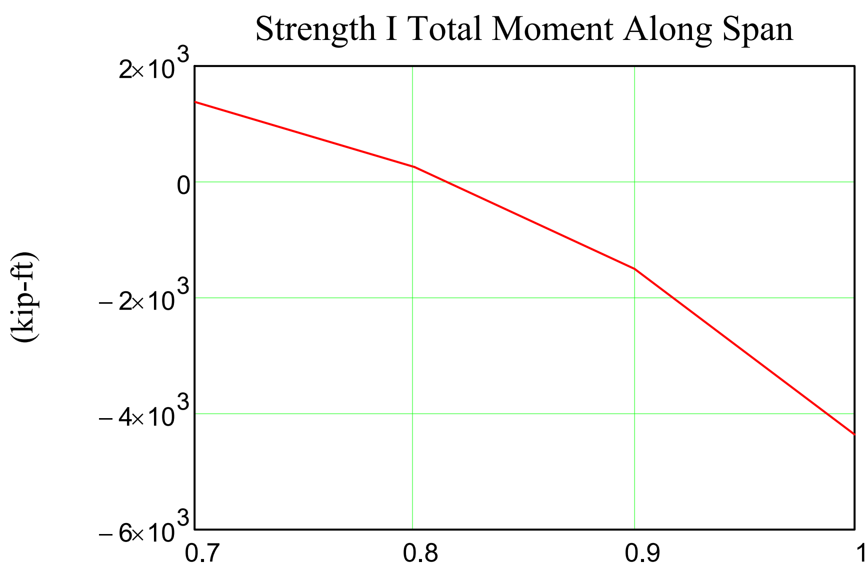
$spa' := spa \cdot 2$ $\boxed{spa' = 8.50}$ in

$As' := \frac{As}{2}$ $\boxed{As' = 10.588}$ in^2

$a' := \frac{As' \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c}$ $\boxed{a' = 3.11}$ in

$M_{n'} := As' \cdot f_y \cdot \left(d_e - \frac{a'}{2}\right) \cdot \frac{1}{12}$ $\boxed{M_{n'} = 3111}$ kip-ft

$M_{r'} := \phi_f \cdot M_{n'}$ $\boxed{M_{r'} = 2799}$ kip-ft



Based on the moment diagram, try locating the first cut off at $cut_1 := 0.90$ span. Note that the Service I crack control requirements control the location of the cut off.

$\boxed{M_{r'} = 2799}$ kip-ft

$\boxed{Mu_{cut1} = 1501}$ kip-ft

$\boxed{Ms_{cut1} = 1565}$ kip-ft

Is Mu_{cut1} less than $M_{r'}$? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.6.3.3]**:

$\boxed{M_{cr} = 1709}$ kip-ft

$\boxed{1.33 \cdot Mu_{cut1} = 1996}$ kip-ft

Is $M_{r'}$ greater than the lesser value of M_{cr} and 1.33*Mu_{cut1}? $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.6.7]**:

$\rho' := \frac{As'}{b_w \cdot d_e}$ $\boxed{\rho' = 0.00585}$

$k' := \sqrt{(\rho' \cdot n)^2 + 2 \cdot \rho' \cdot n} - \rho' \cdot n$ $\boxed{k' = 0.206}$

$j' := 1 - \frac{k'}{3}$ $\boxed{j' = 0.931}$

$\boxed{Ms_{cut1} = 1565}$ kip-ft

$f_{s'} := \frac{Ms_{cut1}}{As' \cdot j' \cdot d_e} \cdot 12 \quad \leq 0.6 f_y$ $\boxed{f_{s'} = 31.582}$ ksi $\leq 0.6 f_y$ O.K.

$\boxed{\beta = 1.076}$

$\boxed{\gamma_e = 0.750}$

$S_{max'} := \frac{700 \gamma_e}{\beta \cdot f_{s'}} - 2 \cdot d_c$ $\boxed{S_{max'} = 9.08}$ in

$\boxed{spa' = 8.50}$ in

Is the bar spacing less than S_{max}? $\boxed{check = \text{"OK"}}$

Check the Fatigue 1 reinforcement limits in accordance with **LRFD [5.5.3]**:

The factored moments at the cut off are: $\boxed{Mf_{DLcut1} = 298}$ kip-ft

$\boxed{Mf_{LLcut1} = 534}$ kip-ft

$\boxed{Mfpos_{LLcut1} = 163}$ kip-ft

$Mf_{cut1} := Mf_{DLcut1} + Mf_{LLcut1}$ $\boxed{Mf_{cut1} = 833}$ kip-ft

Check stress in section for determination of use of cracked or uncracked section properties:

$f_{top\_cut1} := \frac{Mf_{cut1}}{S_c} \cdot 12$ $\boxed{f_{top\_cut1} = 0.257}$ ksi

$f_{limit} := 0.095 \cdot \sqrt{f'_c}$ $\boxed{f_{limit} = 0.269}$ ksi

Therefore: $\boxed{SectionProp = \text{"Un-Cracked"}}$

$f_{min\_cut1} := n \cdot \frac{Mf_{cut1}}{S_c} \cdot 12$ $\boxed{f_{min\_cut1} = 1.174}$ ksi

$\Delta F_{TH\_cut1} := 26 - 0.367 \cdot f_{min\_cut1}$ (for f_y = 60 ksi) $\boxed{\Delta F_{TH\_cut1} = 25.569}$ ksi

The live load range is the sum of the positive and negative fatigue moments:

$Mf_{LLrange} := Mf_{LLcut1} + Mfpos_{LLcut1}$ $\boxed{Mf_{LLrange} = 698}$ kip-ft

$\gamma fLL\Delta f\_cut1 := n \cdot \frac{Mf_{LLrange}}{S_c} \cdot 12$ $\boxed{\gamma fLL\Delta f\_cut1 = 0.984}$ ksi

Is $\gamma f_{LL} \cdot \Delta f$ less than ΔF_{TH}? $\boxed{check = \text{"OK"}}$

Therefore this cut off location, $\boxed{cut_1 = 0.90}$, is OK. The bar shall be extended past the cut off point a distance not less than the maximum of the following, **LRFD [5.10.8.1.2c]**:

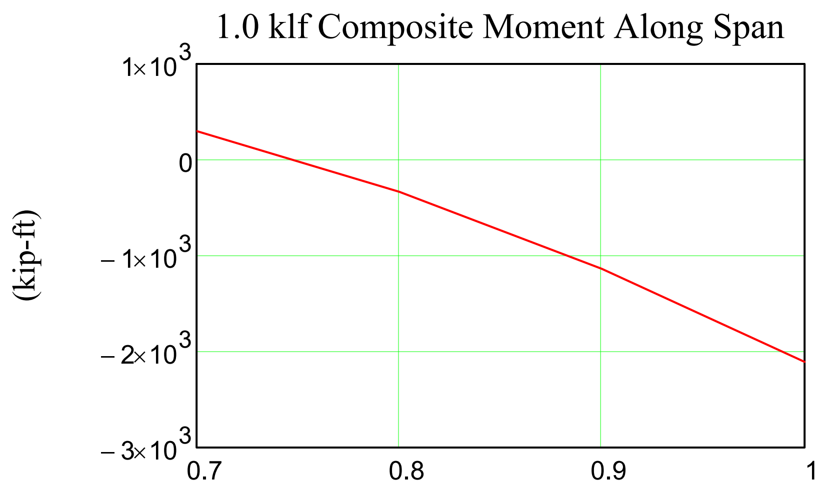
$$extend := \begin{pmatrix} d_e \\ 12 \cdot Bar_D(Bar_{No}) \\ 0.0625 \cdot L \cdot 12 \end{pmatrix} \qquad \boxed{extend = \begin{pmatrix} 60.311 \\ 13.536 \\ 97.500 \end{pmatrix}}$$

$\boxed{\frac{max(extend)}{12} = 8.13}$ ft

$X_1 := L \cdot (1 - cut_1) + \frac{max(extend)}{12}$ $\boxed{X_1 = 21.12}$ feet

USE $\boxed{X_1 = 22}$ feet from the CL of the pier.

The second bar cut off is located at the point of inflection under a uniform 1.0 klf composite dead load. At $cut_2 = 0.750$, $M_{cut2} = (\ 79\ )$ kip-ft. Extend the bar the max(extend) distance calculated above past this point, or 4 feet past the first cut off, whichever is greater.



$X_{2a} := L \cdot (1 - cut_2) + \frac{max(extend)}{12}$ $\boxed{X_{2a} = 40.63}$ feet from the center of the pier

$X_{2b} := X_1 + 4$ $\boxed{X_{2b} = 26.00}$ feet from the center of the pier

$X_2 := max(X_{2a}, X_{2b})$ $\boxed{X_2 = 40.63}$ feet

USE $\boxed{X_2 = 41}$ feet from the CL of the pier.

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24.6 Design Approach - Steps in Design

24.6.1 Obtain Design Criteria

The first design step for a steel girder is to choose the correct design criteria. The design criteria include the following:

- Number of spans
- Span lengths
- Skew angles
- Number of girders
- Girder spacing
- Deck overhang
- Cross-frame spacing
- Flange and web yield strengths
- Deck concrete strength
- Deck reinforcement strength
- Deck thickness
- Dead loads
- Roadway geometry
- Haunch depth

For steel girder design, the following load combinations are generally considered:

- Strength I
- Service II
- Fatigue I

The extreme event limit state (including earthquake load) is generally not considered for a steel girder design.

The following steps are taken in determining the girder or beam spacing and the slab thickness:

1. The girder spacing (and the resulting number of girders) for a structure is determined by considering the desirable girder depth and the span lengths. Refer to 24.4.224.4.2 for design aids. Where depth or deflection limitations do not control the design, it is usually more economical to use fewer girders with a wider spacing and a thicker slab. Four girders are generally considered to be the minimum, and five girders are desirable to facilitate future redecking.

2. The slab overhang on exterior girders is limited to 3'-7" measured from the girder centerline to the edge of slab. The overhang is limited to prevent rotation and bending of the web during construction caused by the forming brackets. The overhang width is generally determined such that the moments and shears in the exterior girder are similar to those in the interior girder. In addition, the overhang is set such that the positive and negative moments in the deck slab are balanced. A common rule of thumb is to make the overhang approximately 0.28 to 0.5 times the girder spacing. For girders less than, or equal to 36-inches in depth, limit the overhang to the girder depth, and preferably no wider than 0.80 the girder depth. The limits for raised sidewalk overhangs on the Standard for Median and Raised Sidewalk Details are likely excessive for such shallow girders.

3. Check if a thinner slab and the same number of members can be used by slightly reducing the spacing.

24.6.2 Select Trial Girder Section

Before the dead load effects can be computed, a trial girder section must be selected. This trial girder section is selected based on previous experience and based on preliminary design. 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.

The following tips are presented to help bridge designers in developing an economical steel girder for most steel girder designs. Other design tips are available in various publications from the American Institute of Steel Construction (AISC) and from steel fabricators.

- Girder depth – The minimum girder depth is specified in **LRFD [2.5.2.6.3]**. An estimate of the optimum girder depth can be obtained from trial runs using design software. The web depth may be varied by several inches more or less than the optimum without significant cost penalty. Refer to 24.4.2 for recommended girder depths for a given girder spacing and span length.

- Web thickness – A "nominally stiffened" web (approximately 1/16 inch thinner than "unstiffened") will generally provide the least cost alternative or very close to it. However, for web depths of approximately 50” or less, unstiffened webs may be more economical.

- Plate transitions – For rolled sections, a change in section should occur only at field splice locations. For plate girders, include the change in section at butt splices and

24.8 Field Splices

24.8.1 Location of Field Splices

Field splices shall be placed at the following locations whenever it is practical:

- At or near a point of dead load contraflexure for continuous spans
- Such that the maximum rolling length of the flange plates is not exceeded, thus eliminating one or more butt splices
- At a point where the fatigue in the net section of the base metal is minimized
- Such that section lengths between splices are limited to 120', unless special conditions govern

24.8.2 Splice Material

For homogeneous girders, the splice material is the same as the members being spliced. Generally, 3/4" diameter high-strength A325 bolted friction-type connectors, conforming to ASTM F3125, are used unless the proportions of the structure warrant larger diameter bolts.

24.8.3 Design

The following is a general description of the basic steps required for field splice design. These procedures and the accompanying equations are described in greater detail in **LRFD [6.13.6]**.

24.8.3.1 Obtain Design Criteria

The first design step is to identify the appropriate design criteria. This includes defining material properties, identifying relevant superstructure information and determining the splice location based on the criteria presented in 24.8.1.

Resistance factors used for field splice design are as presented in 17.2.6.

When calculating the nominal slip resistance of a bolt in a slip-critical connection, the value of the surface condition factor, K_s, shall be taken as follows for the surfaces in contact (faying):

- For steel with fully painted surfaces, use Ks = 0.30.
- For unpainted, blast-cleaned steel or steel with organic zinc paint, use Ks = 0.50.

Where a section changes at a splice, the smaller of the two connected sections should be used in the design, as specified in **LRFD [6.13.6.1.1]**.

24.8.3.1.1 Section Properties Used to Compute Stresses

The section properties used to compute stresses are described in **LRFD [6.10.1.1.1]**.

For calculating flexural stresses in sections subjected to positive flexure, the composite sections for short-term (transient) and long-term (permanent) moments shall be based on n and 3n, respectively.

For calculating flexural stresses in sections subjected to negative flexure, the composite section for both short-term and long-term moments shall consist of the steel section and the longitudinal reinforcement within the effective width of the concrete deck, except as specified otherwise in **LRFD [6.6.1.2.1]**, **LRFD [6.10.1.1.1d]** or **LRFD [6.10.4.2.1]**.

WisDOT policy item:

When computing composite section properties based on the steel section and the longitudinal reinforcement within the effective width of the concrete deck, only the top layer of reinforcement shall be considered.

Where moments due to short-term and long-term loads are of opposite sign at the strength limit state, the associated composite section may be used with each of these moments if the resulting net stress in the concrete deck due to the sum of the factored moments is compressive. Otherwise, the provisions of **LRFD [6.10.1.1.1c]** shall be used to determine the stresses in the steel section. Stresses in the concrete deck shall be determined as specified in **LRFD [6.10.1.1.1d]**.

However, for members 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, as described in **LRFD [6.10.4.2.1]**.

- Live load stresses and stress ranges for fatigue design may be computed using the short-term composite section assuming the concrete deck to be effective for both positive and negative flexure, as described in **LRFD [6.6.1.2.1]**.

WisDOT policy item:

When stresses at the top and bottom of the web are required for web splice design, the flange stresses at the mid-thickness of the flanges can be conservatively used. This allows use of the same stress values for both the flange and web splices, which simplifies the calculations.

24.8.3.1.2 Constructability

As described in **LRFD [6.13.6.1.4a]**, splice connections shall be proportioned to prevent slip during the erection of the steel and during the casting of the concrete deck.

24.13 Painting

The final coat of paint on all steel bridges shall be an approved color. Exceptions to this policy may be considered on an individual basis for situations such as scenic river crossings, unique or unusual settings or local community preference. The Region is to submit requests for an exception along with the Structure Survey Report. The *AMS Standard Color Numbers* available for use on steel structures are shown in Chapter 9 - Materials.

Unpainted weathering steel is used on bridges over streams and railroads. All highway grade separation structures require the use of painted steel, since unpainted steel is subject to excessive weathering from salt spray distributed by traffic. On weathering steel bridges, the end 6' of any steel adjacent to either side of an expansion joint and/or hinge is required to have two shop coats of paint. The second coat is to be brown color similar to rusted steel. Do not paint the exterior face of the exterior girders for aesthetic reasons, but paint the hanger bar on the side next to the web. Additional information on painting is presented in Chapter 9 - Materials.

For painted steel plate I-girders utilize a three-coat system defined by the Standard Specification bid item “Painting Epoxy System (Structure)”. For painted tub girders use a two-coat system defined by the STSP “Painting Polysiloxane System (Structure)”, which includes painting of the inside of the tubs.

Paint on bridges affects the slip resistance of bolted connections. Since faying surfaces that are not galvanized are typically blast-cleaned as a minimum, a Class A surface condition should only be used to compute the slip resistance when Class A coatings are applied or when unpainted mill scale is left on the faying surface. Most commercially available primers will qualify as Class B coatings.

24.14 Floor Systems

In the past, floor systems utilizing two main girders were used on long span structures. Current policy is to use multiple plate girder systems for bridges having span lengths up to 400'. Multiple girder systems are preferred since they are redundant; that is, failure of any single member will not cause failure of the structure.

In a two-girder system, the main girders are designed equally to take the dead load and live load unless the roadway cross section is unsymmetrical. The dead load and live load carried by the intermediate stringers is transferred to the floor beams, which transmit the load to the main girders. In designing the main girders, it is an acceptable practice to assume the same load distribution along the stringers as along the girder and ignore the concentrated loads at the floor beam connections.

The design criteria used for such girders is the same as the criteria used for plate girders and rolled sections. Particular attention should be paid to the sufficiency of the girder connection details and to the lateral bracing requirements and connections.

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E24-1 2-Span Continous Steel Plate Girder Bridge - LRFD

This example shows design calculations conforming to the *AASHTO LRFD Eight Edition -2017* 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, Eigth 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.)

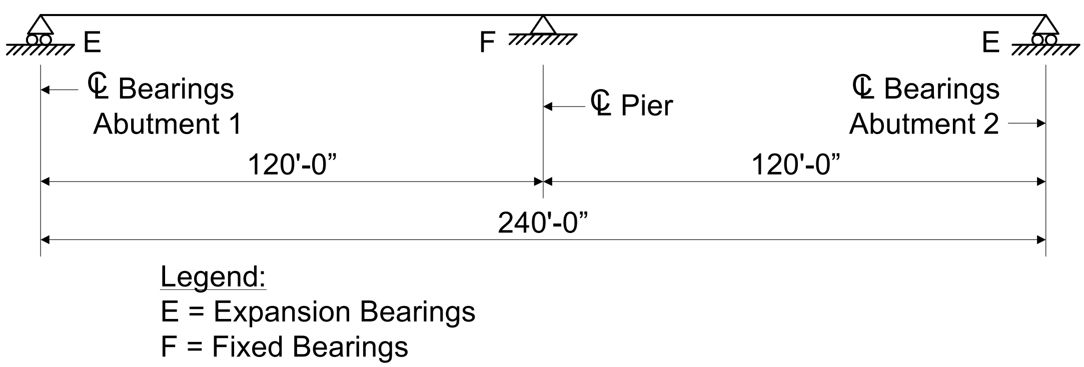


Figure E24-1.1-1
Span Configuration

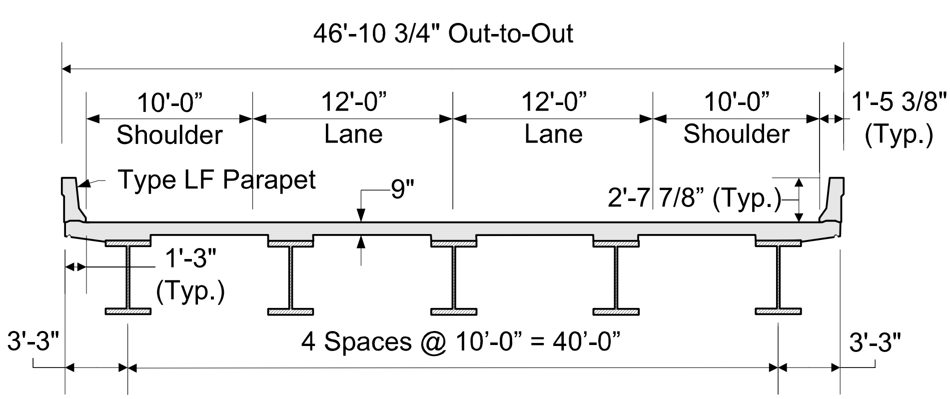


Figure E24-1.1-2
Superstructure Cross Section

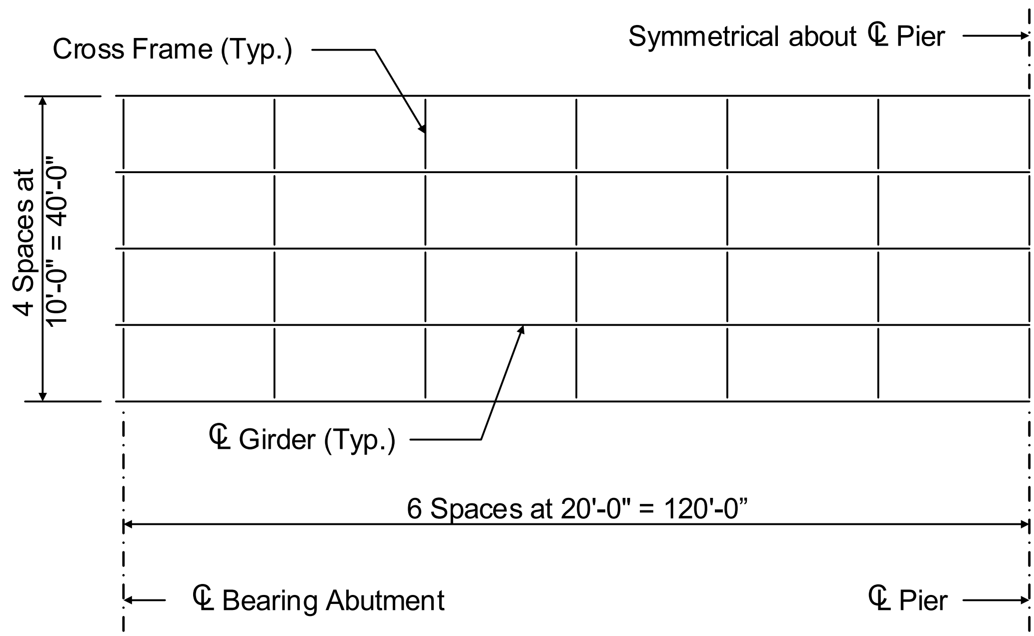


Figure E24-1.1-3
Framing Plan

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	-	-	1.75	1.75	-	-	-

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.

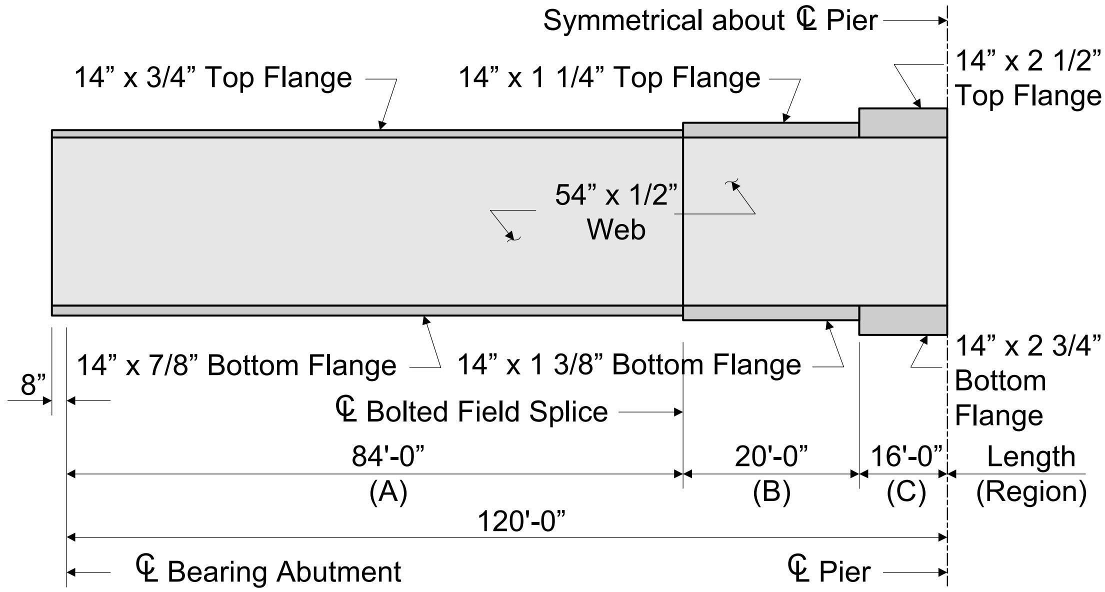


Figure E24-1.2-1
Plate Girder Elevation

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 (reba	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:

$$DL_A := W_s \cdot \frac{A_A}{12^2} \qquad \boxed{DL_A = 0.169} \text{ klf}$$

$$DL_B := W_s \cdot \frac{A_B}{12^2} \qquad \boxed{DL_B = 0.217} \text{ klf}$$

$DL_C := W_s \cdot \frac{A_C}{12^2}$ $\boxed{DL_C = 0.342}$ 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 - Interior Beams (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 Grider 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 E24-1.4-2
Dead Load Moments

Dead Load Shears - Interior Beams (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 Grider 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 E24-1.4-3
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 4.6.2.2.2.3a-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:

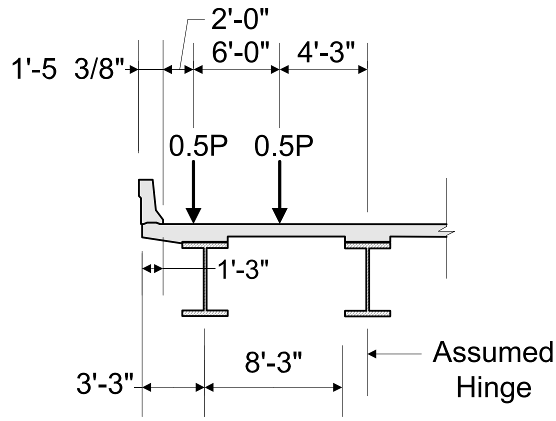


Figure E24-1.5-1
Lever Rule

$x_1 := S - 6 + (d_e - 2)$ $\qquad$ $x_2 := S + (d_e - 2)$

$$g_{ext\_moment\_1} := \frac{(0.5)\cdot(x_1) + (0.5)\cdot(x_2)}{S}$$ $\boxed{g_{ext\_moment\_1} = 0.700}$ 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:

$$e := 0.77 + \frac{d_e}{9.1}$$ $\boxed{e = 0.990}$

$g_{ext\_moment\_2} := e\cdot g_{int\_moment\_2}$ $\qquad$ $\boxed{g_{ext\_moment\_2} = 0.693}$ 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:

$$g_{ext\_shear\_1} := \frac{(0.5)\cdot(x_1) + (0.5)\cdot(x_2)}{S}$$ $\boxed{g_{ext\_shear\_1} = 0.700}$ 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]**:

$$e := 0.6 + \frac{d_e}{10} \qquad \boxed{e = 0.800}$$

$$g_{ext\_shear\_2} := e \cdot g_{int\_shear\_2} \qquad \boxed{g_{ext\_shear\_2} = 0.761} \quad \text{lanes}$$

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 and fatigue 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 Moments - Interior Beams (Kip-Feet)											
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	0.0	863.8	1470.0	1871.8	2037.7	2001.3	1785.7	1384.6	826.7	258.3	0.0
Maximum Negative	0.0	-114.8	-229.6	-344.4	-459.9	-574.7	-689.5	-804.3	-919.1	-1274.7	-2065.7
Fatigue Range	0.0	401.6	668.1	836.1	888.5	862.6	787.9	618.3	406.9	457.7	508.6

Table E24-1.5-3
Live Load Moments

Live Load Shears - Interior Beams (Kips)											
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	114.4	97.1	80.1	64.4	50.1	37.2	25.9	16.2	8.2	2.5	0.0
Maximum Negative	-13.0	-13.5	-21.4	-34.4	-48.3	-62.6	-77.1	-91.6	-105.9	-119.7	-132.9
Fatigue Range	59.2	51.3	43.6	40.8	43.6	44.9	46.6	48.7	52.2	55.4	58.8

Table E24-1.5-4
Live Load Shears

The design live load values for HL-93 and fatigue loading, as presented in the previous tables, 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

$LF_{LL} := 1.75$

$M_{LL} := 2037.7$ kip-ft

$M_{total} := LF_{DC} \cdot M_{DC} + LF_{DW} \cdot M_{DW} + LF_{LL} \cdot M_{LL}$ $\boxed{M_{total} = 5382.9}$ 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

$$f_{noncompDL} := \frac{-M_{noncompDL} \cdot (12)}{S_{topgdr}}$$ $\boxed{f_{noncompDL} = -16.26}$ ksi

Parapet dead load (composite):

$M_{parapet} := 159.1$ kip-ft

$S_{topgdr} := 4404.7$ in^3

$$f_{parapet} := \frac{-M_{parapet} \cdot (12)}{S_{topgdr}}$$ $\boxed{f_{parapet} = -0.43}$ ksi

Future wearing surface dead load (composite):

$M_{fws} := 150.6$ kip-ft

$S_{topgdr} := 4404.7$ in^3

$f_{fws} := \frac{-M_{fws} \cdot (12)}{S_{topgdr}}$ $\boxed{f_{fws} = -0.41}$ ksi

Live load (HL-93) and dynamic load allowance:

$M_{LL} = 2037.70$ kip-ft

$S_{topgdr} := 24820.6$ in^3

$f_{LL} := \frac{-M_{LL} \cdot (12)}{S_{topgdr}}$ $\boxed{f_{LL} = -0.99}$ 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]**:

$$f_{Str} := (LF_{DC} \cdot f_{noncompDL}) + (LF_{DC} \cdot f_{parapet}) + (LF_{DW} \cdot f_{fws}) + (LF_{LL} \cdot f_{LL})$$

$\boxed{f_{Str} = -23.21}$ 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	1113.7	15.23	-16.26	0.00
Parapet DL	159.1	1.55	-0.43	-0.05
FWS DL	150.6	1.47	-0.41	-0.05
LL - HL-93	2037.7	18.25	-0.99	-0.62
LL - Fatigue Range	888.5	7.96	-0.43	-0.27
Summary of Factored Values:				
Limit State	**Moment (K-ft)**	**f_{botgdr} (ksi)**	**f_{topgdr} (ksi)**	**$f_{topslab}$ (ksi)**
Strength I	5382.9	55.12	-23.21	-1.21
Service II	4072.4	41.98	-18.39	-0.90
Fatigue I	1554.9	13.93	-0.75	-0.47

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	-3143.1	-16.56	17.60	0.00
Parapet DL	-405.7	-2.05	1.85	2.22
FWS DL	-383.9	-1.94	1.75	2.10
LL - HL-93	-2065.7	-10.41	9.44	11.28
Summary of Unfactored Values (Assuming Concrete Effective):				
Loading	**Moment (K-ft)**	**f_{botgdr} (ksi)**	**f_{topgdr} (ksi)**	**f_{deck} (ksi)**
Noncomposite DL	-3143.1	-16.56	17.60	0.00
Parapet DL	-405.7	-1.84	0.92	0.09
FWS DL	-383.9	-1.74	0.87	0.08
LL - HL-93	-2065.7	-8.70	1.85	0.45
LL - Fatigue Range	-506.3	-2.13	0.45	0.11
Summary of Factored Values:				
Limit State	**Moment (K-ft)**	**f_{botgdr} (ksi)**	**f_{topgdr} (ksi)**	**f_{deck} (ksi)**
Strength I *	-8626.8	-44.38	43.47	25.66
Service II **	-6618.1	-31.45	21.80	0.75
Fatigue I **	-886.0	-3.73	0.79	0.19

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	111.5
Parapet DL	14.5
FWS DL	13.8
LL - HL-93	132.9
LL - Fatigue Range	58.8
Summary of Factored Values:	
Limit State	**Shear (kips)**
Strength I	410.8
Service II	312.6
Fatigue I	102.9

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.

Factored Moments - Interior Beams (Kip-feet)											
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
Strength I (Max)	0.0	2440.5	4113.6	5112.8	5382.9	4982.4	3952.3	2280.9	14.5	-2596.7	-5011.9
Strength I (Min)	0.0	727.9	1139.3	1234.4	1012.1	474.4	-379.3	-1549.7	-3040.7	-5279.5	-8626.8
Service II (Max)	0.0	1850.8	3118.7	3872.8	4072.4	3760.8	2968.5	1686.8	-50.2	-2056.9	-3932.7
Service II (Min)	0.0	578.7	909.2	991.8	825.5	412.0	-249.3	-1158.8	-2319.7	-4049.8	-6618.1
Fatigue I	0.0	821.8	1404.7	1762.0	1915.1	1953.9	1912.0	1704.8	1366.9	1042.3	886.1

Table E24-1.6-4
Factored Moments

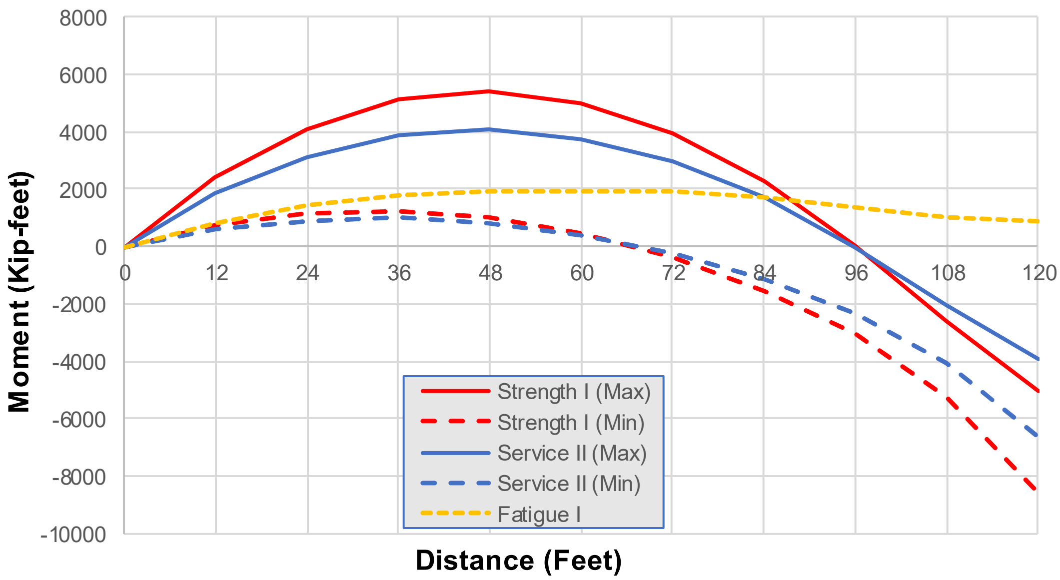


Figure E24-1.6-1
Envelope of Moments

Factored Shears - Interior Beams (Kips)											
Live Load/Fatigue	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
Strength I	67.9	40.6	0.1	-48.7	-99.4	-150.9	-202.6	-254.2	-306.3	-358.4	-410.7
Service II	54.2	32.8	1.6	-35.7	-74.4	-113.8	-153.3	-192.7	-232.5	-272.4	-312.5
Fatigue I	103.6	89.7	76.3	71.4	76.3	78.6	81.5	85.3	91.3	97.0	102.8

Table E24-1.6-5
Factored Shears

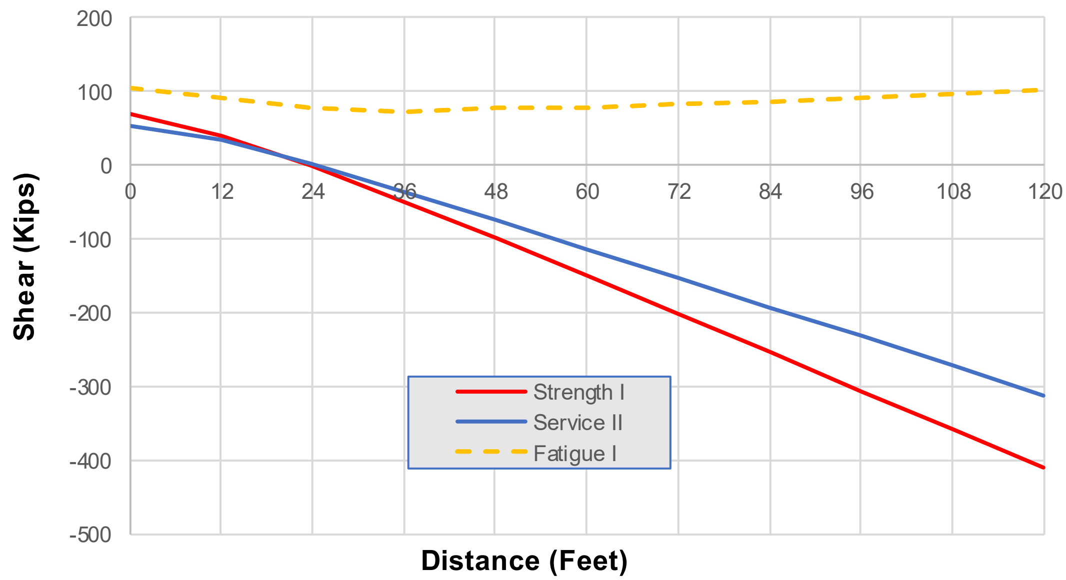


Figure E24-1.6-2
Envelope of Shears

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.

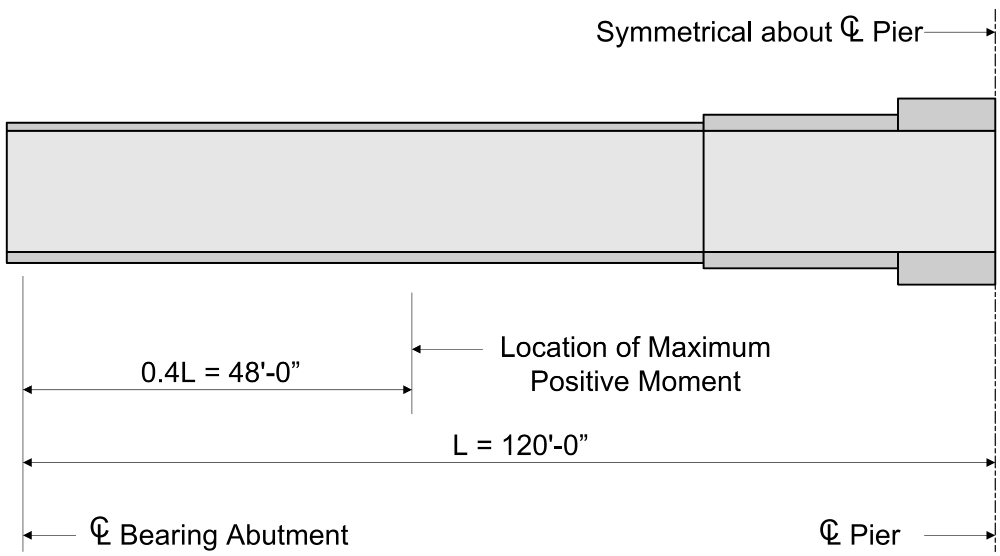


Figure E24-1.6-3
Location of Maximum Positive Moment

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:

$$\frac{D}{t_w} \leq 150$$

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

$t_f \ge 1.1 \cdot t_w$ $\boxed{1.1t_w = 0.55}$ 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]**.

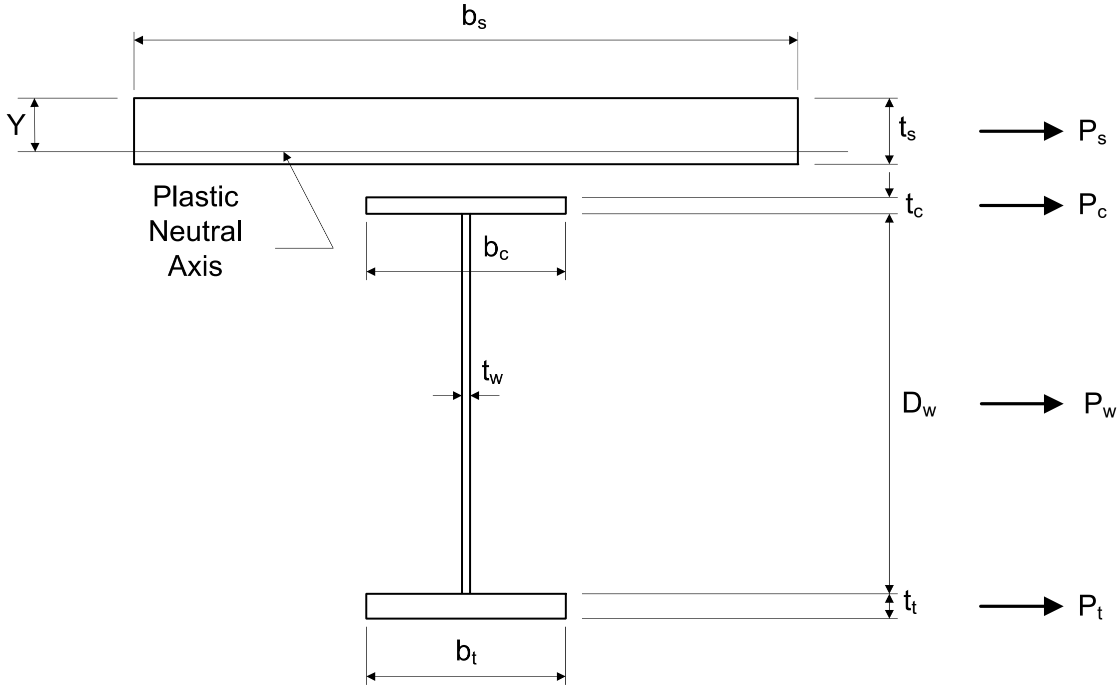


Figure E24-1.8-1
Computation of Plastic Moment Capacity for Positive Bending Sections

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:

$P_t + P_w = 1963$	kips	$P_c + P_s = 3993$	kips
$P_t + P_w + P_c = 2488$	kips	$P_s = 3468$	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.

$$Y := (t_s) \cdot \left(\frac{P_c + P_w + P_t}{P_s} \right) \qquad Y = 6.10 \quad \text{in}$$

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.

$$Compression := 0.85 \cdot f'_c \cdot b_s \cdot Y \qquad Compression = 2488 \quad \text{kips}$$

$$Tension := P_t + P_w + P_c \qquad Tension = 2488 \quad \text{kips} \quad \text{OK}$$

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]**:

$$d_c := \frac{-t_c}{2} + 3.75 + t_s - Y \qquad d_c = 5.78 \quad \text{in}$$

$$d_w := \frac{D}{2} + 3.75 + t_s - Y \qquad d_w = 33.15 \quad \text{in}$$

$$d_t := \frac{t_t}{2} + D + 3.75 + t_s - Y \qquad d_t = 60.59 \quad \text{in}$$

$$M_p := \frac{\frac{Y^2 \cdot P_s}{2 \cdot t_s} + (P_c \cdot d_c + P_w \cdot d_w + P_t \cdot d_t)}{12} \qquad M_p = 7707 \quad \text{kip-ft}$$

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]**:

$$\frac{2 \cdot D_{cp}}{t_w} \le 3.76 \cdot \sqrt{\frac{E}{F_{yc}}}$$

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

$$M_{AD} := \left[\frac{S_{ST}}{12^3} \cdot \left(F_y \cdot 144 - \frac{M_{D1}}{\frac{S_{NC}}{12^3}} - \frac{M_{D2}}{\frac{S_{LT}}{12^3}}\right)\right] \qquad \boxed{M_{AD} = 2994} \text{ kip-ft}$$

$M_{ybot} := M_{D1} + M_{D2} + M_{AD}$ $\boxed{M_{ybot} = 4811}$ kip-ft

For the top flange:

$S_{NC} := 821.7$ in^3

$S_{LT} := 4404.7$ in^3

$S_{ST} := 24820.6$ in^3

$$M_{AD} := \frac{S_{ST}}{12^3} \cdot \left(F_y \cdot 144 - \frac{M_{D1}}{\frac{S_{NC}}{12^3}} - \frac{M_{D2}}{\frac{S_{LT}}{12^3}}\right) \qquad \boxed{M_{AD} = 58974} \text{ kip-ft}$$

$M_{ytop} := M_{D1} + M_{D2} + M_{AD}$ $\boxed{M_{ytop} = 60791}$ 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]**:

$D_p \le 0.1 D_t$

$D_p := Y$ $\boxed{D_p = 6.10}$ 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]**:

$D_p \le 0.42 D_t$

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:

$$\Sigma\eta_i \cdot \gamma_i \cdot Q_i \le R_r$$

or in this case:

$$\Sigma\eta \cdot \gamma \cdot M_u \le M_r$$

$$\eta := 1.00$$

As computed in E24-1.6,

$\Sigma \cdot \gamma \cdot M_u = 5383$ kip-ft

Therefore

$\Sigma\eta \cdot \gamma \cdot M_u = 5383$ 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:

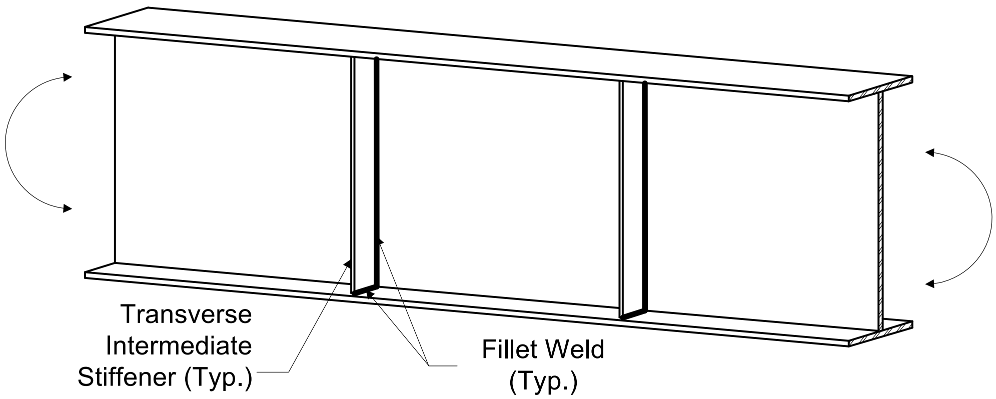


Figure E24-1.13-1
Load-Induced Fatigue Detail

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

$\Delta F_n = 12.00$ 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:

$f_{botgdr} := 13.93$ ksi

$f_{botgdr} \leq \Delta F_n$ NG

NOTE: A new trial girder section is required to statisfy the above fatigue requirement.

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:

$$f_f \le 0.95R_h \cdot F_{yf}$$

Bottom flange

$$f_f + \frac{f_l}{2} \le 0.95R_h \cdot F_{yf}$$

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:

$f_{botgdr} := 41.98$ ksi

$f_{topgdr} := -18.39$ 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 $\le$ 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.

$f_{bu} := 1.5\cdot 16.26$ 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.

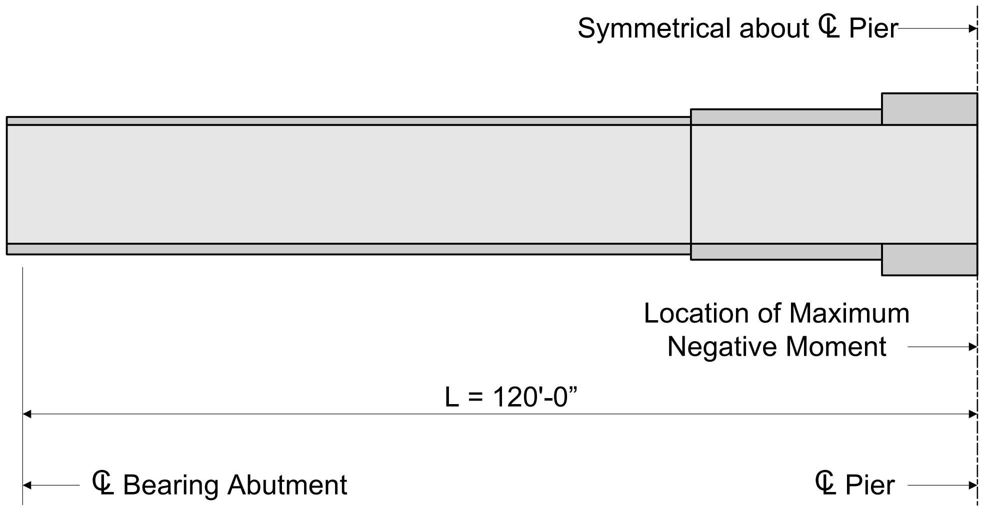


Figure E24-1.17-1
Location of Maximum Negative Moment

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:

$$\frac{D}{t_w} \leq 150 \qquad \boxed{\frac{D}{t_w} = 108.00} \qquad \text{OK}$$

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:

$$\frac{b_f}{2 \cdot t_f} \leq 12.0$$

$b_f := 14$

$t_f := 2.50$

$\boxed{\frac{b_f}{2 \cdot t_f} = 2.80}$ OK

$$b_f \geq \frac{D}{6}$$ $\boxed{\frac{D}{6} = 9.00}$ in OK

$$t_f \geq 1.1 \cdot t_w$$ $\boxed{1.1t_w = 0.55}$ in OK

$$0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10$$

$$I_{yc} := \frac{2.75 \cdot 14^3}{12}$$ $\boxed{I_{yc} = 628.83}$ 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.

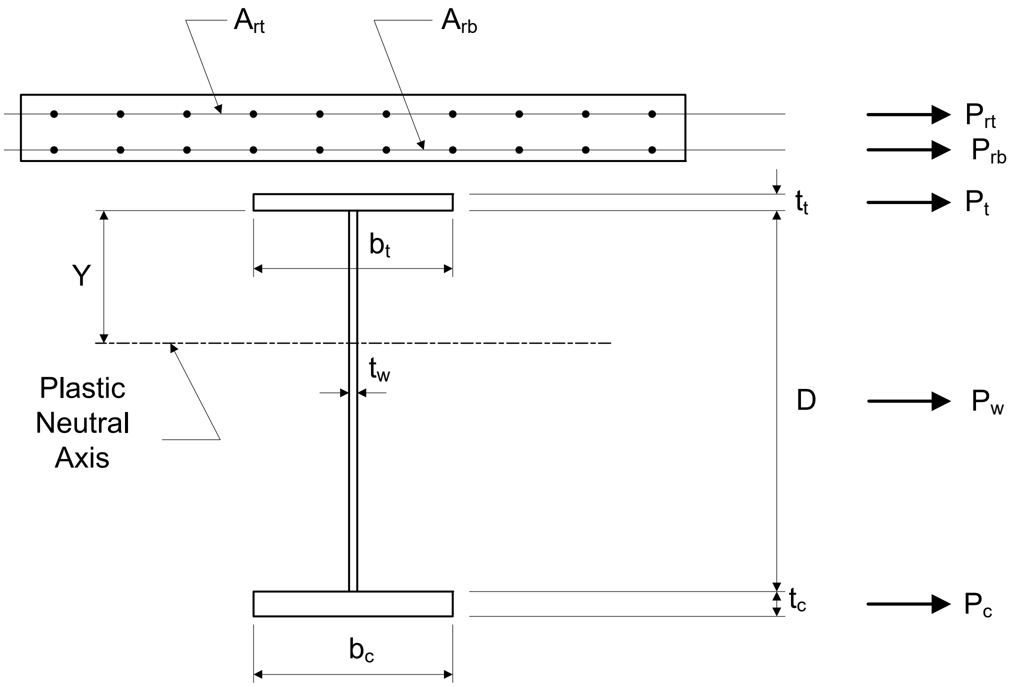


Figure E24-1.18-1
Computation of Plastic Moment Capacity for Negative Bending Sections

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]**:

$$\frac{2 \cdot D_c}{t_w} \leq 5.7 \sqrt{\frac{E_s}{F_{yc}}} \qquad \lambda_{rw} := 5.7 \sqrt{\frac{E_s}{F_{yc}}}$$

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

$M_{LL0.8L} := 919.1$ kip-ft

$S_{rebar0.8L} := 2380.2$ in^3

$$f_1 := 1.25 \cdot \frac{M_{NCDC0.8L} \cdot 12}{S_{NCDC0.8L}} + 1.25 \cdot \frac{M_{par0.8L} \cdot 12}{S_{rebar0.8L}} + 1.50 \cdot \frac{M_{fws0.8L} \cdot 12}{S_{rebar0.8L}} + 1.75 \cdot \frac{M_{LL0.8L} \cdot 12}{S_{rebar0.8L}}$$

$\boxed{f_1 = 15.59}$ ksi

$f_2 := 44.38$ ksi (Table E24-1.6-2)

$\boxed{\frac{f_1}{f_2} = 0.35}$

$$C_b := 1.75 - 1.05 \cdot \left(\frac{f_1}{f_2}\right) + 0.3 \cdot \left(\frac{f_1}{f_2}\right)^2 \quad < 2.3$$

$\boxed{C_b = 1.42}$

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

$$F_{nc} := C_b \cdot \left[1 - \left(1 - \frac{F_{yr}}{R_h \cdot F_{yc}}\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \cdot R_b \cdot R_h \cdot F_{yc}$$

$\boxed{F_{nc} = 58.46}$ ksi

$F_{nc} \leq R_b \cdot R_h \cdot F_{yc}$ $\boxed{R_b \cdot R_h \cdot F_{yc} = 50.00}$ ksi

Use:

$F_{nc} := 50.00$ ksi

$\boxed{\phi_f \cdot F_{nc} = 50.00}$ ksi

$f_{bu} := 44.38$ 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:

$\boxed{f_{bu} + \frac{1}{3} \cdot (0) = 44.38}$ 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) $f_{bu} := 43.47$ 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:

$$V_p := 0.58 \cdot F_{yw} \cdot D \cdot t_w \qquad \boxed{V_p = 783.0} \text{ kips}$$

$V_n := C \cdot V_p$ $\boxed{V_n = 305.6}$ 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:

$\Sigma\eta_i \cdot \gamma_i \cdot Q_i \le R_r$

Or in this case:

$\Sigma\eta_i \cdot \gamma_i \cdot V_i \le V_r$

$\eta_i := 1.00$

As computed in E24-1.6, the factored Strength I Limit State shear at the pier is as follows:

$\Sigma\eta_i \cdot \gamma_i \cdot V_i = 410.8$ 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]**.

$$\frac{2 \cdot D \cdot t_w}{b_{fc} \cdot t_{fc} + b_{ft} \cdot t_{ft}} \le 2.5$$

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:

$$V_n = V_p \cdot \left[C + \frac{0.87 \cdot (1 - C)}{\sqrt{1 + \left(\frac{d_o}{D}\right)^2}} \right]$$

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:

$\Sigma\eta_i \cdot \gamma_i \cdot V_i = 410.8$ kips

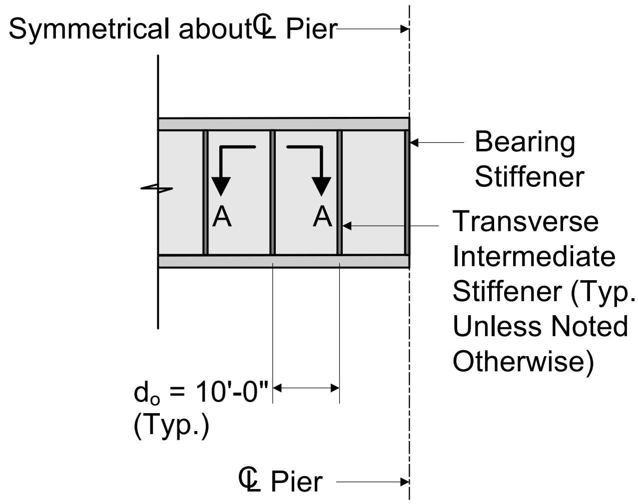
$\boxed{V_r = 515.86}$ kips OK

Therefore, the girder design section at the pier satisfies the shear resistance requirements for the web.

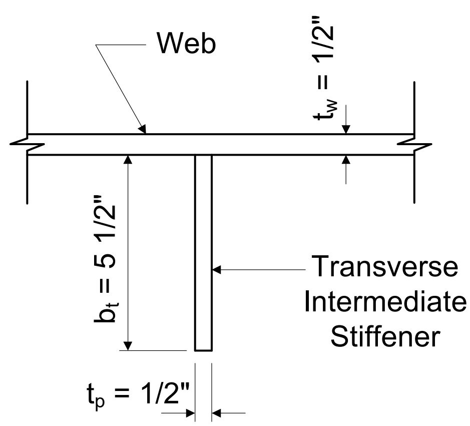
E24-1.22 Design Transverse Intermediate Stiffeners - Negative Moment Region

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]**:

$b_t \geq 2.0 + \frac{D}{30.0}$ 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

$\boxed{2.0 + \frac{D}{30.0} = 3.80}$ 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 :

$$I_t \geq I_{t1} + (I_{t2} - I_{t1}) \left(\frac{V_u - \phi_v \cdot V_{cr}}{\phi_v \cdot V_n - \phi_v \cdot V_{cr}} \right)$$

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

$b := \min(d_o, D)$ $b = 54.00$ in

$$J := \max\left[\frac{2.5}{\left(\frac{d_o}{D}\right)^2} - 2.0, 0.5\right] \qquad J = 0.50$$

$$I_{t1} := b \cdot t_w^{\,3} \cdot J = 3.38 \qquad in^4$$

$$I_{t2} = \frac{D^4 \cdot \rho_t^{\,1.3}}{40} \cdot \left(\frac{F_{yw}}{E}\right)^{1.5}$$

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)

$$\boxed{\frac{0.31 \cdot E_s}{\left(\frac{b_t}{t_p}\right)^2} = 74.30} \qquad ksi$$

$$F_{ys} := 50.00 \qquad ksi$$

Use

$$F_{crs} := \min\left[F_{ys}, \frac{0.31 \cdot E_s}{\left(\frac{b_t}{t_p}\right)^2}\right] \qquad \boxed{F_{crs} = 50.00} \qquad ksi$$

$$\rho_t := \max\left(\frac{F_{yw}}{F_{crs}}, 1.0\right) \qquad \boxed{\rho_t = 1.00}$$

$$\boxed{I_{t2} := \frac{D^4 \cdot \rho_t^{\,1.3}}{40} \cdot \left(\frac{F_{yw}}{E_s}\right)^{1.5} = 15.22} \qquad in^4$$

Since $I_{t2} > I_{t1}$, the moment of inertia, I_t, of the transverse stiffener must satisfy:

$$I_t \geq I_{t1} + (I_{t2} - I_{t1})\left(\frac{V_u - \phi_v \cdot V_{cr}}{\phi_v \cdot V_n - \phi_v \cdot V_{cr}}\right)$$

$V_u := 410.8$ kip

$V_{cr} := C \cdot V_p = 367.53$ kip

$V_n = 515.86$ kip

$$I_{t1} + (I_{t2} - I_{t1})\left(\frac{V_u - \phi_v \cdot V_{cr}}{\phi_v \cdot V_n - \phi_v \cdot V_{cr}}\right) = 6.83 \quad in^4$$

$I_t := \dfrac{t_p \cdot b_t^3}{3}$ $\boxed{I_t = 27.73}$ in^4

Therefore,

$$I_t \geq I_{t1} + (I_{t2} - I_{t1})\left(\frac{V_u - \phi_v \cdot V_{cr}}{\phi_v \cdot V_n - \phi_v \cdot V_{cr}}\right) \quad \text{OK}$$

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:

$V_u = V_{noncomp} + V_{par} + V_{fws} + 1.75V_{LLfatiguerange}$

$V_u := 111.5 + 14.5 + 13.8 + (1.75 \cdot 58.8)$ $\boxed{V_u = 242.70}$ 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:

$f_{botgdr} := -31.45$ ksi

$f_{topgdr} := 21.80$ 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)

$$Depth_{comp} := \frac{-f_{botgdr}}{|f_{botgdr}| + f_{topgdr}} \cdot Depth_{gdr} \qquad \boxed{Depth_{comp} = 34.99} \text{ in}$$

$t_{botfl} := 2.75$ in

$D_c := Depth_{comp} - t_{botfl}$ $\boxed{D_c = 32.24}$ in

$D := 54.0$ in

$$k := \frac{9.0}{\left(\frac{D_c}{D}\right)^2}$$ $\boxed{k = 25.24}$

$$\boxed{\frac{0.9 \cdot E_s \cdot k}{\left(\frac{D}{t_w}\right)^2} = 56.49}$$ 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

$$f_c := f_{botgdr} \cdot \left(\frac{D_c}{D_c + t_{bf}}\right)$$ $\boxed{f_c = -28.98}$ 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 construct bility 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

$$V_{DZ} := 2.5 \cdot V_o \cdot \left(\frac{V_{30}}{V_B}\right) \cdot \ln\left(\frac{Z}{Z_o}\right) \qquad \boxed{V_{DZ} = 26.12} \quad \text{MPH}$$

$$P_D := P_B \cdot \left(\frac{V_{DZ}}{V_B}\right)^2 \qquad \boxed{P_D = 0.0034} \quad \text{ksf}$$

$d := 8.45$ ft from bottom of girder to top of barrier

$$W := P_D \cdot d \qquad \boxed{W = 0.0288} \quad \text{kips/ft}$$

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

$$P_D := \frac{W}{d} \qquad \boxed{P_D = 0.0355} \quad \text{ksf}$$

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]**:

$$W := \frac{\eta \cdot \gamma \cdot P_D \cdot d}{2} \qquad \boxed{W = 0.060} \quad \text{kips/ft}$$

Next, the maximum lateral moment in the flange due to the factored wind loading is computed as follows:

$$M_w := \frac{W \cdot \left(\frac{L_b}{12}\right)^2}{10} \qquad \boxed{M_w = 2.40} \quad \text{kip-ft}$$

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

$$f_l := \frac{6 \cdot -M_w \cdot 12}{t_{fb} \cdot b_{fb}^{\,2}} \qquad \boxed{f_l = -0.321} \quad \text{ksi}$$

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:

$$f_{bu} := 1.25\cdot(-16.56 + -2.05) + 1.50(-1.94) + 1.35(-10.41)$$

$\boxed{f_{bu} = -40.23}$ ksi

$\boxed{f_{bu} + \frac{1}{3}f_l = -40.33}$ 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 (except as noted in Section E24-1.13), 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:

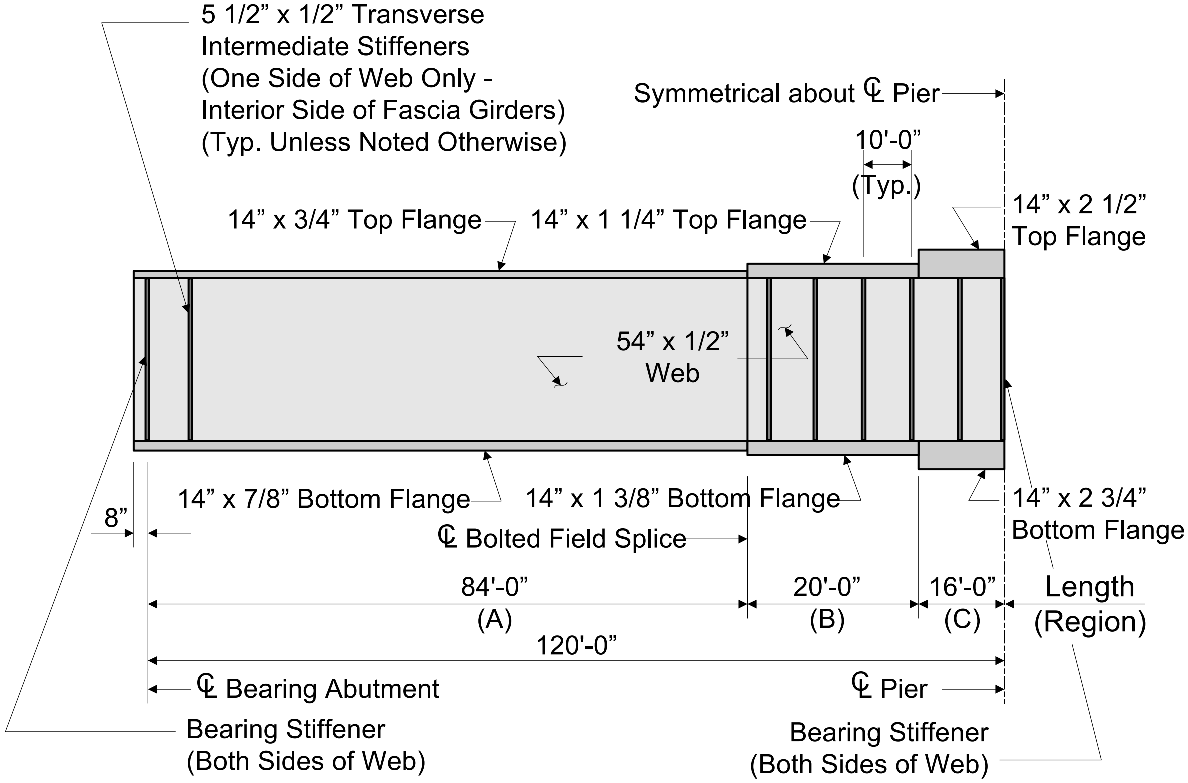


Figure E24-1.27-1
Final Plate Girder Elevation

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.

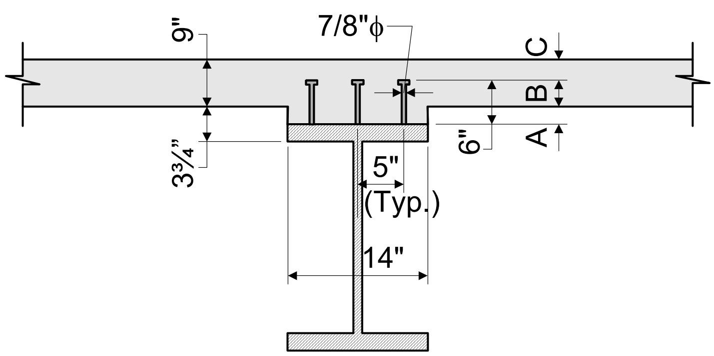


Figure E24-1.28-1
Stud Shear Connectors

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

$$\boxed{\frac{Height_{stud}}{Diameter_{stud}} = 6.86}$$ 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:

$V_f := 1.75 \cdot (43.60)$ $\boxed{V_f = 76.30}$ 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

$V_{fat} := \frac{V_f \cdot Q}{I}$ $\boxed{V_{fat} = 1.39}$ kip/in

$V_{sr} := V_{fat}$ $\boxed{V_{sr} = 1.39}$ kip/in

$d := 0.875$ in

$Z_r := 5.5 \cdot d^2$ $\boxed{Z_r = 4.21}$ kips

$p := \frac{n \cdot Z_r}{V_{sr}}$ $\boxed{p = 9.09}$ 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)

$Q := \left[\frac{(8.5) \cdot (120)}{8}\right] \cdot (64.750 - 48.868)$ $\boxed{Q = 2024.95}$ in^3

$V_f := 1.75 \cdot (58.8)$ $\boxed{V_f = 102.90}$ kips

$V_{fat} := \frac{V_f \cdot Q}{I}$ $\boxed{V_{fat} = 1.50}$ kip/in

$V_{sr} := V_{fat}$ $\boxed{V_{sr} = 1.50}$ kip/in

$p := \frac{n \cdot Z_r}{V_{sr}}$ $\boxed{p = 8.44}$ in

Therefore, based on the above pitch computations to satisfy the fatigue limit state, use the following pitch throughout the entire girder length:

$p := 8$ 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 8 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]**:

$n := \frac{P}{Q_r}$ $\boxed{n = 141.0}$

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 8 inches, as previously computed for the fatigue limit state, and using the above length, the number of shear connectors provided is as follows:

$n := 3 \cdot \frac{L \cdot (12)}{p}$ $\boxed{n = 216.0}$ 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 8 inches, as previously computed for the fatigue limit state, and using the above length, the number of shear connectors provided is as follows:

$n := 3 \cdot \frac{L \cdot (12)}{p}$ $\boxed{n = 324.0}$ OK

Therefore, using a pitch of 8 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.

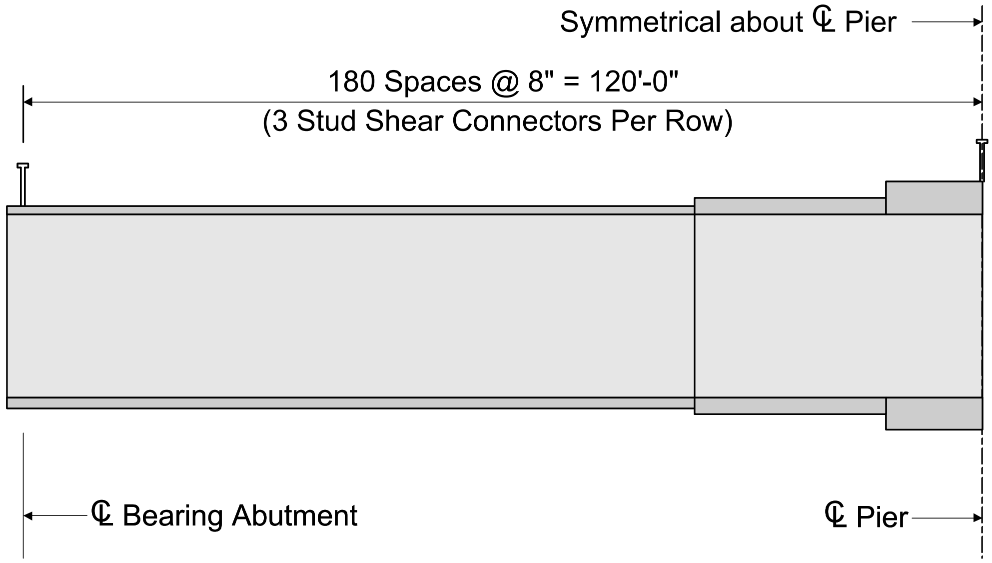


Figure E24-1.28-2
Shear Connector Spacing

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]**.

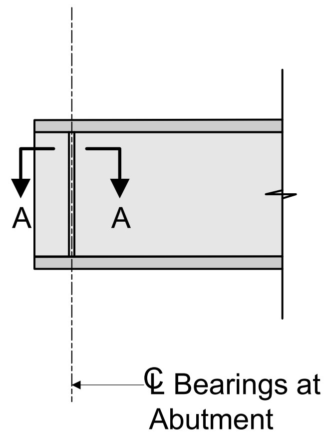
For plate girders, bearing stiffeners are required to be placed on the webs at all bearing locations. At all locations supporting concentrated loads where the loads are not transmitted through a deck or deck system, either bearing stiffeners are to be provided or the web must satisfy the provisions of **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.

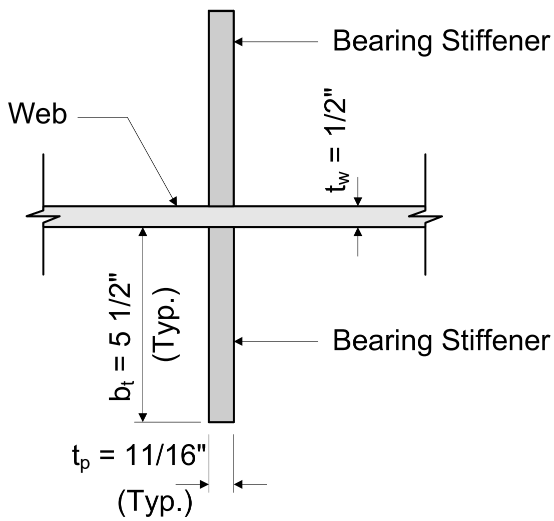
The bearing stiffeners in this design example consist of one plate welded to each side of the web. The connections to the web will be designed to transmit the full bearing force due to factored loads and is presented in E24-1.30.

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:

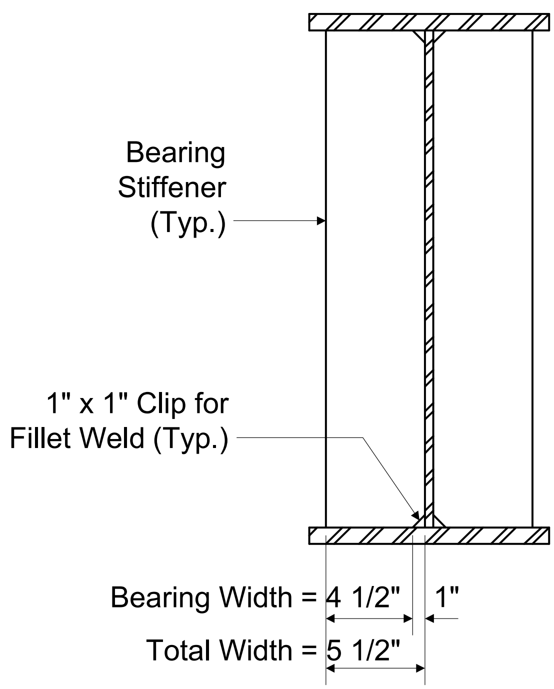


Figure E24-1.29-2
Bearing Width

$b_{brg} := b_t - 1.0$ $\boxed{b_{brg} = 4.50}$ 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:

$React_{Factored} := (1.25 \cdot 63.7) + (1.50 \cdot 7.4) + (1.75 \cdot 114.4)$

$\boxed{React_{Factored} = 290.93}$ 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:

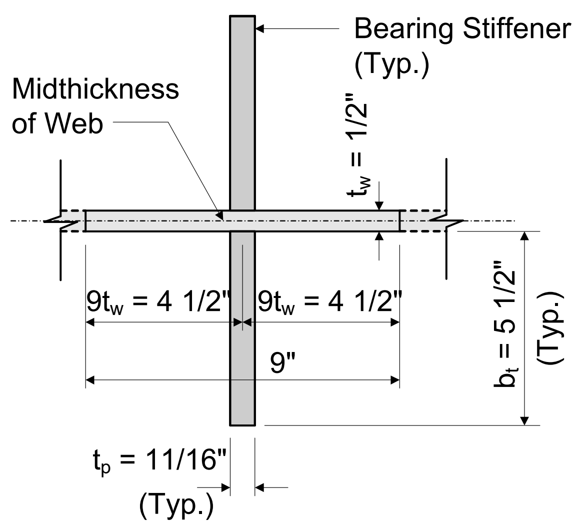


Figure E24-1.29-3
Bearing Stiffener Effective Section

$P_r = \phi_c \cdot P_n$ (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

$P_o := 1.0 F_y \cdot A_g$ $P_o = 603$ 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

$\boxed{React_{Factored} = 290.93}$ 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 Introduction

This splice design example shows design calculations conforming to the *AASHTO LRFD Bridge Design Specifications (Eighth Edition - 2017)* as supplemented by the *WisDOT Bridge Manual (January 2019)*.

According to **LRFD [6.13.6.1.3a] & LRFD [6.13.6.1.3b]**

- Splices should be made at or near points of dead load contraflexure.
- Inside and outside splice plates are used for flange splices, and two splice plates are used at both sides for the web splice
- The combined area of the flange and web splices plates often equal or exceed the areas of the smaller flanges and web to which they are attached
- Bolted splices for flexural members shall be designed using slip-critical connections as specified in **LRFD[6.13.2.1.1.]**
- Oversize or slotted holes are not permitted to be used for bolted splices
- Web and flange splices in areas of stress reversal shall be investigated for both positive and negative flexure to determine the governing condition.
- All the moments are assumed to be resisted by the flange splices. Should the factored moments exceed the moment resistance provided by the flange splices, the web splice is assumed to resist the additional moment in addition to its design shear.

As per **LRFD [C6.13.6.1.3a],** the method specified below ignores the moment due to eccentricity of the shear

E24-2.2 Obtain Design Criteria

Note: This example uses the girder from example E24-1
Presented in Figure E24-2.2-1 is the steel girder configuration and the bolted field splice location.

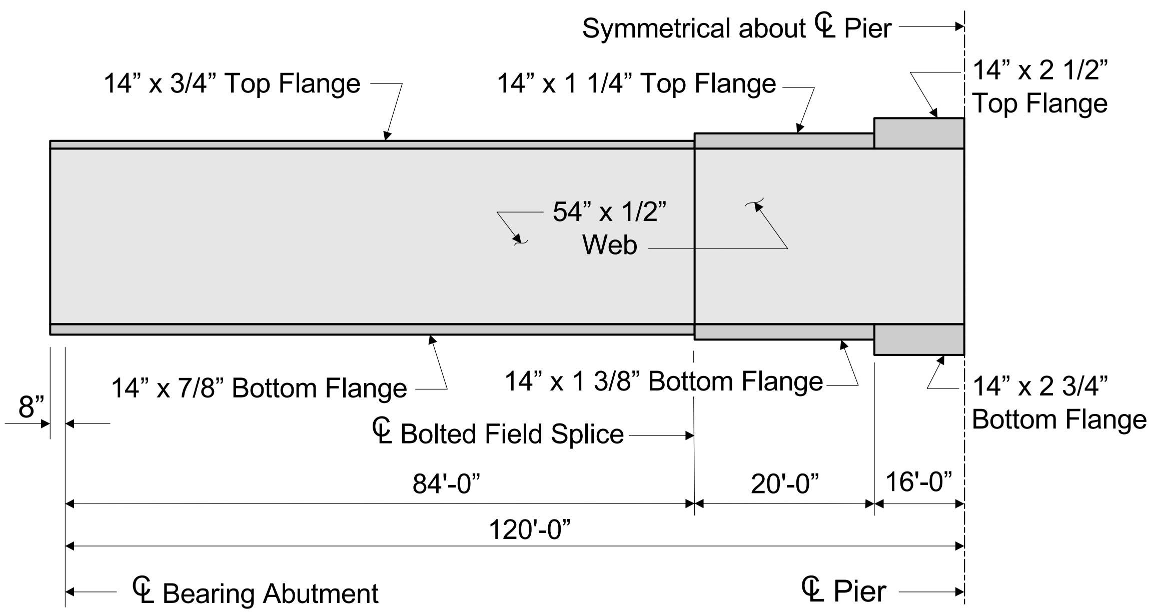

Figure E24-2.2-1
Plate Girder Elevation

Filler plate thickness: $t_{fill} := 0.50$ in

Filler plate width: $b_{fill} := 14$ in

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 E24-2.2-1 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 E24-2.2-1 are as follows:

Web thickness: $t_w = 0.50$ in

Web depth: $D = 54.00$ 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: $d_b := 0.875$ in **LRFD [6.13.2.5]**

Bolt cross area $A_b := \pi \cdot \frac{d_b^2}{4} = 0.60$ in^2

Bolt hole diameter (for design purposes add 1/16" to standard hole diameter):

$d_{hole} := \frac{15}{16}$ in **LRFD Table [6.13.2.4.2-1]**

Bolt tensile strength: $F_{ub} := 120$ ksi **LRFD table [6.4.3.1.1-1]**

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:

$$A_{deckreinftop} := (0.44) \cdot \frac{W_{eff}}{7.5} = 7.04 \quad in^2$$

For the bottom steel:

$$A_{deckreinfbot} := (0.44) \cdot \frac{W_{eff}}{7.5} = 7.04 \quad in^2$$

Resistance factors **LRFD [6.5.4.2]**:

Flexure: $\phi_f := 1.00$

Shear: $\phi_v := 1.00$

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: $\phi_{bb} := 0.80$

ASTM F3125 Grade A325 and A490 bolts in shear: $\phi_s := 0.80$

Block shear: $\phi_{bs} := 0.80$

For shear, rupture in connection element $\phi_{vu} := 0.80$

E24-2.3 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 in this example 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.

E24-2.4 Flange Splice Design Loads

A summary of the unfactored moments at the splice from example 24-1 are listed below. The live loads include dynamic load allowance and distribution factors.

The moments due to fatigue are not listed below as **LRFD [C6.13.6.1.3a]** states that the combined area of the flange and web splices plates often equal or exceed the areas of the smaller flanges and web to which they are attached, and the flanges and web are usually checked separately for either equivalent or more critical fatigue category details. Therefore, fatigue of the splices will not control and not need to be checked.

Dead load moments:

Non-composite: $M_{NDL} := -107.8$ kip-ft

Composite: $M_{CDL} := -2.8$ kip-ft

Future wearing surface: $M_{FWS} := -2.6$ kip-ft

Live load moments:

HL-93 positive: $M_{PLL+IL} := 1384.6$ kip-ft

HL-93 negative: $M_{NLL+IL} := -804.3$ kip-ft

E24-2.5 Loads Factors

Bolted splices for flexural members shall be designed using slip critical connection. Slip critical connections are proportioned to prevent slip under load combination Service II and to provide bearing and shear resistance under the applicable strength limit state load combinations. The load factors for these load combinations are selected based on **LRFD Tables [3.4.1-1] & [3.4.1-2]**:

Load Factors				
State	Strength I		Service II	
Load	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	1.35	1.30	1.30

Table E24-2.5-1
Load Factors

E24-2.5.1 Strength I Limit State

Both positive and negative moments are investigated in Strength I and Service II limit state. Load factors are selected from the above table to produce the largest moments.

Max. Positive Moment

$$M_{u+} := 0.9\left(M_{NDL} + M_{CDL}\right) + 0 \cdot M_{FWS} + 1.75 \cdot M_{PLL+IL} = 2323.51 \quad \text{kip-ft}$$

Max. Negative Moment

$$M_{u-} := 1.25 \cdot \left(M_{NDL} + M_{CDL}\right) + 1.5 \cdot M_{FWS} + 1.75 \cdot M_{NLL+IL} = -1549.67 \quad \text{kip-ft}$$

The future wearing surface is excluded to get the largest negative moment

E24-2.5.2 Service II Limit State

Max. Positive Moment

$$M_{+} := 1 \cdot \left(M_{NDL} + M_{CDL}\right) + 0 \cdot M_{FWS} + 1.3 \cdot M_{PLL+IL} = 1689.38 \quad \text{kip-ft}$$

Max. Negative Moment

$$M_{-} := 1 \cdot \left(M_{NDL} + M_{CDL}\right) + 1 \cdot M_{FWS} + 1.3 \cdot M_{NLL+IL} = -1158.79 \quad \text{kip-ft}$$

Type	M(+) [K.ft]	M(-) [K.ft]
Strength I	2323.51	-1549.67
Service II	1689.38	-1158.80

Table E24-2.5.2-1
Summary of Design Moments
From 24E-1

E24-2.6 Flange Splice Plates Dimensions

LRFD [C6.13.6.1.3a]: the combined area of the flange and web splices plates often equal or exceed the areas of the smaller flanges and web to which they are attached

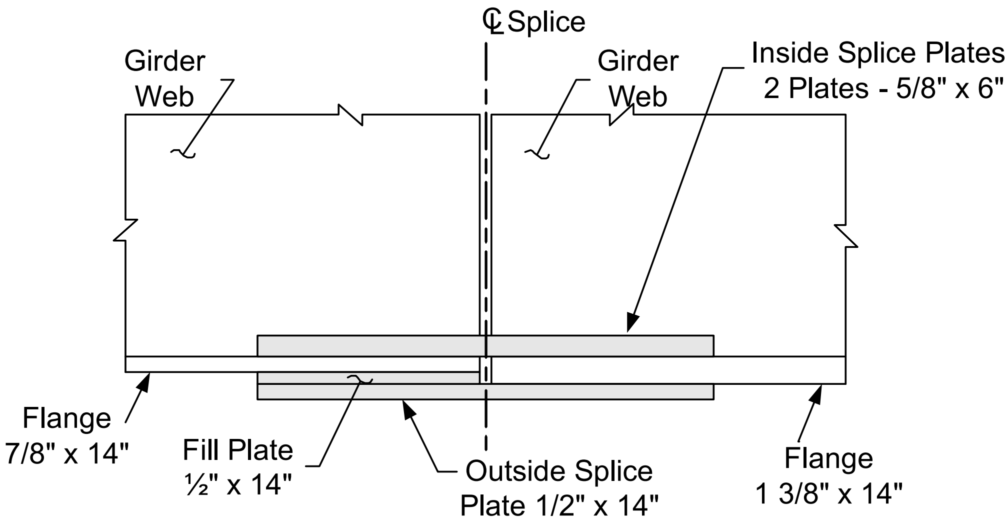

Figure E24-2.6-1
Bottom Flange Splice

The dimensions of the elements involved in the bottom flange splice from Figure E24-2.6-1 are:

Thickness of inside splice plate:	$t_{in} := 0.625$	in
Width of inside splice plate:	$b_{in} := 6$	in
Number of inside plates:	$N_{inp} := 2$	Plates
Thickness of outside splice plate:	$t_{out} := 0.5$	in
Width of outside splice plate:	$b_{out} := 14$	in
Thickness of the filler plate:	$t_{fill} = 0.50$	in
Width of the filler plate:	$b_{fill} = 14.00$	in

E24-2.7 Strength Limit State Design of Flange Splice Plates

E24-2.7.1 Bolt Design

E24-2.7.1.1 Bottom Flange Bolts

According to **LRFD [6.13.6.1.3b]** the flange splice plates and their connections shall be designed to develop the smaller design yield resistance of the flanges at the point of splice. The total number of bolts on one side of the splice are determined by dividing the smaller design yield resistance at the point of splice, P_{fy}, by the factored shear resistance of the bolts. Then the bearing resistance of the flange splice bolts holes shall be checked at the strength limit state.

E24-2.7.1.1.1 Design Yield Resistance of the Bottom Flange at the Point of the Splice

The design yield resistance of each flange, P_{fy}, at the point of splice shall be taken as:

$$P_{fy} = F_{yf} \cdot A_e$$ **LRFD [6.13.6.1.3b-1]**:

Where a section changes at splice, the smaller P_{fy} of the two connected sections shall be used in the design. In this example, the bottom flange on the left has a smaller area with the same F_y.

NOTE: A minimum two rows of bolts on each side of the joint to be used to ensure proper alignment and stability of the girder during construction.
Assuming 4 rows of bolts across the width of the flange

$$Row\_No := 4$$

The effective area of flange A_e:

$$A_e = \left(\frac{\phi_u \cdot F_u}{\phi_y \cdot F_{yf}}\right) \cdot A_n \le A_g$$ **LRFD [6.13.6.1.3b-2]**:

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_{yf}	= Specified minimum yield strength of the flange under consideration (ksi)

The net area of bottom flange A_{n_bot}:

$$A_{n\_bot} := \left(b_{flbL} - Row\_No \cdot d_{hole}\right) \cdot t_{flbL} = 8.97 \qquad in^2$$

The gross area of bottom flange A_{g_bot}:

$A_{g\_bot} := b_{flbL} \cdot t_{flbL} = 12.25$ in^2

The effective area of bottom flange A_{e_bot}:

$$A_{e\_bot} := \min\left(\frac{\phi_u}{\phi_y}\frac{F_u}{F_y} \cdot A_{n\_bot}, A_{g\_bot}\right) = 9.82 \quad \text{in}^2$$

The design yield resistance of bottom flange, P_{fy_bot}

$P_{fy\_bot} := A_{e\_bot} \cdot F_{yf} = 490.92$ Kips

E24-2.7.1.1.2 The Shear Resistance of the Bolt

LRFD [6.13.2.7] Factored shear resistance of bolt (ASTM F3125) at the strength limit state in joints whose length between the extreme fasteners measured parallel to the line of action of force is less than 38.0 in shall be taken as:

$R_{n1} = 0.56A_b \cdot F_{ub} \cdot N_{st}$ When threads are excluded **LRFD Eq. [6.13.2.7.1]**

$R_{n2} = 0.45A_b \cdot F_{ub} \cdot N_{st}$ When threads are included **LRFD Eq. [6.13.2.7.2]**

ϕ_s = Resistance factor for bolt in shear **LRFD [6.5.4.2]**

A_b = Area of the bolt corresponding to the nominal diameter (in^2)

F_{ub} = Specified minimum tensile strength of the bolt specified in **LRFD [6.4.3]** (Ksi)

N_{st} = Number of shear planes per bolt

LRDF[6.13.2.7]: When joint length exceeds 38.0 in., reduction factor of 0.83 is applied to $\phi_s \cdot R_n$. This reduction is applied only to lap splice tension connection.

Number of shear planes at bottom flange N_{sb}:

LRFD C6.13.6.1.3b

- If inner and outer flange splice plates do not differ by more than 10%, the connections are proportioned assuming double shear connection (N_s=2) and P_{fy} at the strength limit state is assumed divided equally to the inside and outside plates and their connections.

- When the inner and outer flange splice plates differ by more than 10%, the design force P_{fy} in each splice plates and its connection at the strength limit state should determined by multiplying P_{fy} by the ratio of the area of the splice plate under consideration to the total area of inner and outer splice plates and the connection are proportioned for the maximum calculated splice plate force acting on a single shear plane (N_s=1).

The area of inner splice plates at bottom flange A_{inn_bot}:

$A_{inn\_bot} := N_{inp} \cdot (t_{in} \cdot b_{in}) = 7.50$ in^2

The area of outside splice plate at bottom flange A_{out_bot}:

$A_{out\_bot} := t_{out} \cdot b_{out} = 7.00$ in^2

$$\left(1 - \frac{A_{out\_bot}}{A_{inn\_bot}}\right) = 0.07$$

The difference between the outer and inner flange splice plates is less than 10%, therefore, P_{fy} will be divided equally to the inner and outer splice plates and their connections and the connections are proportioned assuming a double shear connection.

$N_{sb} := 2$ Plates

Total splice area at bottom flange:

$A_{Bot\_splice} := A_{inn\_bot} + A_{out\_bot} = 14.50$ in^2 .> $A_{BF} := t_{flbL} \cdot b_{flbL} = 12.25$ in^2

See **LRFD [C6.13.6.1.3b]** to determine if the bolt threads are included or excluded from the shear plane.

In this example, the bolt diameter = 0.875 less than 1.0 in., so the threads are excluded from the shear planes.

Therefore

$$\phi_s \cdot R_n = \phi_s \cdot R_{n1} = 64.65 \quad \text{kips}$$

Due to unequal thickness of the top and bottom flanges on the left and right side of the splice, filler plates need to be used. When filler plate is 0.25 in. or more in thickness there are two options **LRFD [6.13.6.1.4]**:

- Either the fillers shall be extended and secured by additional bolts and no need to reduce the factored shear resistance of the bolts
- Or the filler need not be extended and the strength limit state of the bolts in shear will be reduced by the following factor:

$$R := \left(\frac{1 + \gamma}{1 + 2\gamma}\right)$$

The reduction factor is only applied on the side of the connection with the filler.

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 (in^2)

The outer flange splice plate and flange width will be equal in the splice.

Sum of the area of the fillers on the top and bottom of the connected plate:

$A_f := b_{fill} t_{fill}$ $\quad A_f = 7.00$ $\quad in^2$

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 A_{g_bot}:

$A_{g\_bot} = 12.25$ $\quad in^2$

Sum of splice plate areas is equal to the gross areas of the inside and outside splice plates:

$A_{Bot\_splice} = 14.50$ $\quad in^2$

The minimum of the areas is:

$A_{p\_b} := \min\left(A_{BF}, A_{Bot\_splice}\right)$ $\quad A_{p\_b} = 12.25$ $\quad in^2$

Therefore:

$$\gamma := \frac{A_f}{A_{p\_b}} \qquad \gamma = 0.57$$

The reduction factor due to the filler is determined to be:

$$R_{fill\_bot} := \left(\frac{1+\gamma}{1+2\gamma}\right) \qquad R_{fill\_bot} = 0.73$$

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:

$R_{bot} = \phi_s \cdot R_{nb} \cdot R_{fill\_bot}$ $\quad R_{bot} := 47.41$ $\quad$ kips

E24-2.7.1.1.3 Number of Bolts

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:

The number of bolts required per side is:

$$N_{bot\_calculated} := \frac{P_{fy\_bot}}{R_{bot}} \qquad N_{bot\_calculated} = 10.35 \quad \text{Bolts}$$

Use 4 rows with 3 bolts per row for bottom flange without stagger on each side of the splice to resist the maximum Strength I flange design force in shear is twelve.

$N_{bot} := 12$ $\quad$ Bolts

E24-2.7.1.1.4 Bolts Spacing

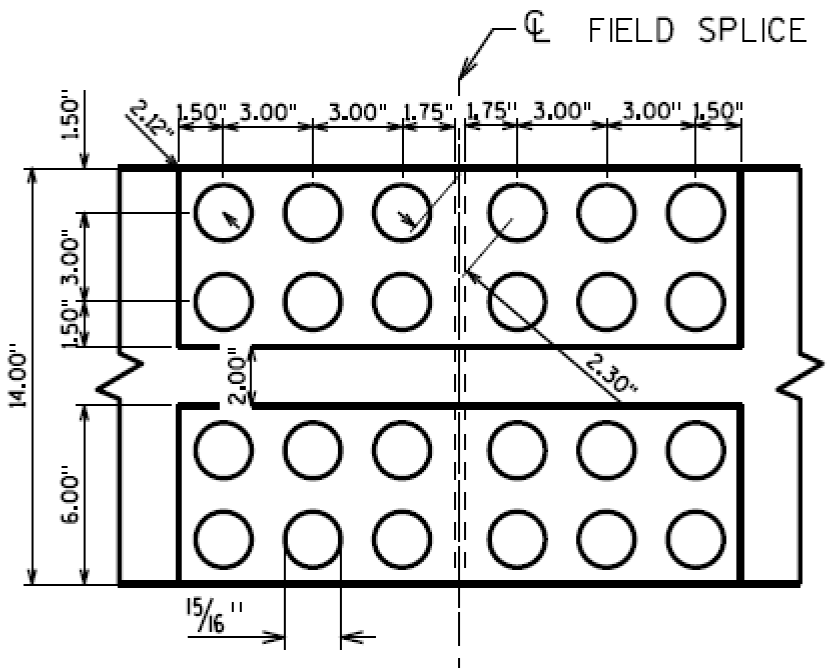

Figure E24-2.7.1.1.4-1
Bottom Flange Inside Splice

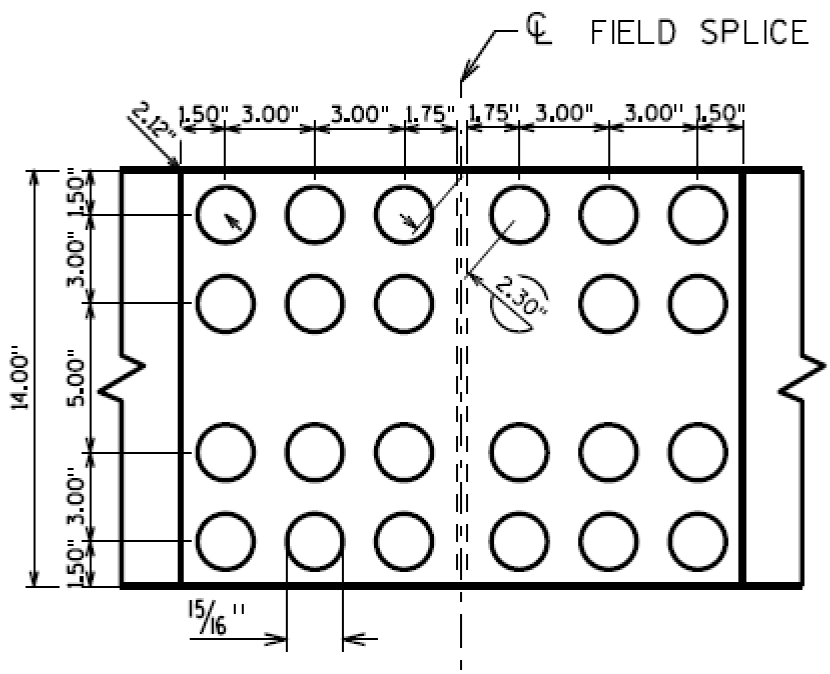

Figure E24-2.7.1.1.4-2
Bottom Flange Outside Splice

The 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 **LRFD[6.13.2.6.1]**.

$d_b = 0.875$ in

$s_{min} := 3 \cdot d_b$ $s_{min} = 2.63$ in

Use $s = 3.00$ in See figures E24-2.7.1.1.4-1 and E24-2.7.1.1.4-2

The minimum spacing requirement is satisfied.

The 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 when the bolts are not staggered:

$s \le (4.0 + 4.0 \cdot t) \le 7.0$ When the bolts are not staggered **LRFD [6.13.2.6.2-1]**:

Where:

t = Thickness of the thinner outside plate or shape (in)

$t_{out} = 0.5000$ in

Maximum spacing for sealing at the edge:

$4 + 4 \cdot t_{out} = 6.00$ in

$s \le 6 \le 7.00$ 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:

$s_{end} := 5.00$ in

Maximum spacing for sealing at the end of the splice plate:

$4.0 + 4.0 \cdot t_{out} = 6.00$ in

$s_{end} \le 6 \le 7.00$ OK

The 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.

The end distance LRFD [6.13.2.6.5]:

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 1/8" **LRFD Table [6.13.2.6.6-1]**. Referring to Figures E24-2.7.1.1.4-1 thru E24-2.7.1.1.4-2, 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 D_{max} shall not be more than eight times the thickness of the thinnest outside plate or five inches.

Usually the maximum distance is measured prependicular to the edge of the flange plate or the splice plate. However, this example check the maximum distance from the corner of the bolt to the corner of the flange plate and the corner of the splice plate.

$D_{max} \leq 8 \cdot t \leq 5.00$ in

$t := t_{out}$ $\quad t_{out} = 0.5000$ in

$8 \cdot t_{out} = 4.00$ in

The maximum distance from the bolts to the corner of the girder flange is:

$D_{max} := \sqrt{1.50^2 + 1.75^2} = 2.30$ in

$2.30 \cdot in \leq 4.0 \cdot in$ OK

The maximum distance from the corner bolts to the corner of the splice plate is equal to:

$\sqrt{1.5^2 + 1.5^2} = 2.12$ in

$2.12 \cdot in \leq 4.0 \cdot in$ OK

E24-2.7.1.1.5 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, as calculated before, is the smaller of the P_{fy} of the two connected sections at the splice:

$$P_{cu} := 490.92 \qquad \text{kips}$$

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 flange plate on the left side.

For standard holes, oversize holes, short-slotted holes loaded in any direction, and long-slotted holes parallel to the applied bearing force, the nominal resistance of interior and end bolt hole at the strength limit state, R_n, shall be taken as:

- With bolts spaced at a clear distance between holes not less than 2.0d with a clear end distance not less than 2.0d:

  $R_n=2.4dtF_u$ **LRFD [6.13.2.9-1]**

- If either the clear distance between holes is less than 2.0d, or the clear end distance less than 2.0d:

  $R_n=1.2L_ctF_u$ **LRFD [6.13.2.9-2]**

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)

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:

$d_b = 0.875$ in $\qquad 2 \cdot d_b = 1.75$ in

$d_{hole} = 0.938$ in

For the bolts adjacent to the end of the flange 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:

$$L_{c_1} := 1.75 - \frac{d_{hole}}{2} \qquad L_{c_1} = 1.28 \text{ in}$$

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:

$$L_{c_2} := 3.00 - d_{hole} \qquad L_{c_2} = 2.06 \text{ in}$$

For the flange plate on the left side:

$t_{flbL} = 0.875$ in

$F_u = 65.00$ ksi

The nominal resistance for the end row of bolt holes is computed as follows:

$R_{n_1} := 4 \cdot \left(1.2 \cdot L_{c_1} \cdot t_{flbL} \cdot F_u\right)$ $R_{n_1} = 349.78$ kips

The nominal resistance for the remaining bolt holes is computed as follows:

$R_{n_2} := 8 \cdot \left(2.4 \cdot d_b \cdot t_{flbL} \cdot F_u\right)$ $R_{n_2} = 955.50$ kips

The total nominal resistance of the bolt holes is:

$R_n := R_{n_1} + R_{n_2}$ $R_n = 1305.28$ kips

$\phi_{bb} = 0.80$

$R_r := \phi_{bb} \cdot R_n$ $R_r = 1044.23$ kips

Check:

$P_{cu} = 490.92$ kips $<$ $R_r = 1044.23$ kips OK

E24-2.7.1.2 Top Flange Bolts

E24-2.7.1.2.1 Design Yield Resistance of the Top Flange

The top flange on the left has a smaller area with the same F_y, so the top flange on the left will control

The net area of top flange A_{n_top}:

$A_{n\_top} := (b_{fltL} - Row\_No \cdot d_{hole}) \cdot t_{fltL} = 7.69$ in^2

The gross area of top flange A_{g_top}:

$A_{g\_top} := b_{fltL} \cdot t_{fltL} = 10.50$ in^2

The effective area of top flange A_{e_top}:

$A_{e\_top} := \min\left(\frac{\phi_u}{\phi_y} \frac{F_u}{F_y} \cdot A_{n\_top}, A_{g\_top}\right) = 8.42$ in^2

The design yield resistance of top flange, P_{fy_top}

$P_{fy\_top} := A_{e\_top} \cdot F_{yf} = 420.79$ Kip

E24-2.7.1.2.2 Shear Resistance of the Bolts

The area of inner splice plates at top flange A_{inn_top}:

$A_{inn\_top} := N_{inp} \cdot t_{in} \cdot b_{in} = 7.50$ in^2

The area of outside splice plate at top flange A_{out_top}:

$A_{out\_top} := t_{out} \cdot b_{out} = 7.00$ in^2

$$\left(1 - \frac{A_{out\_top}}{A_{inn\_top}}\right) = 0.07$$

Total splice area at top flange:

$A_{Top\_splice} := A_{inn\_top} + A_{out\_top} = 14.50$ in^2 .> $A_{g\_TF} := t_{fltL} \cdot b_{fltL} = 10.50$ in^2

The difference between the outer and inner flange splice plates is less than 10%, therefore, P_{fy} will be divided equally to the inner and outer splice plates and their connections and the connections are proportioned assuming a double shear connection (N_s=2).

$N_{st} := 2$ Planes

the bolt diameter = 0.875 less than 1.0 in., so the threads are excluded from the shear planes.

Threads_bottom is excluded

$\phi_s \cdot R_{nt} = 64.65$ Kips

The outer flange splice plate and flange width will be equal in the splice.

There is reduction factor that needs to be applied due to filler plate

Sum of the area of the fillers on the top and bottom of the connected plate:

$A_f := b_{fill} t_{fill}$ $A_f = 7.00$ in^2

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.

Top flange area A_{g_TF}:

$A_{g\_TF} = 10.50$ in^2

Sum of splice plate areas is equal to the gross areas of the inside and outside splice plates:

$A_{Top\_splice} = 14.50$ in^2

The minimum of the areas is:

$A_{p\_t} := \min(A_{Top\_splice}, A_{g\_TF})$ $A_{p\_t} = 10.50$ in^2

Therefore:

$$\gamma := \frac{A_f}{A_{p\_t}} \qquad \gamma = 0.67$$

The reduction factor is determined to be:

$$R_{fill\_top} := \left(\frac{1+\gamma}{1+2\gamma}\right) \qquad R_{fill\_top} = 0.71$$

$R_{top} := \phi_s \cdot R_n \cdot R_{fill\_top}$ $R_{top} = 46.18$ kips

E24-2.7.1.2.3 Number of Bolts

$$N_{top\_calculated} := \frac{P_{fy\_top}}{R_{top}} \qquad N_{top\_calculated} = 0.56 \text{ bolts}$$

Use 4 rows with 3 bolts per row on each side of the splice of the top flange

$N_{top} := 12$ Bolts

E24-2.7.2 Moment Resistance

E24-2.7.2.1 Positive Moment

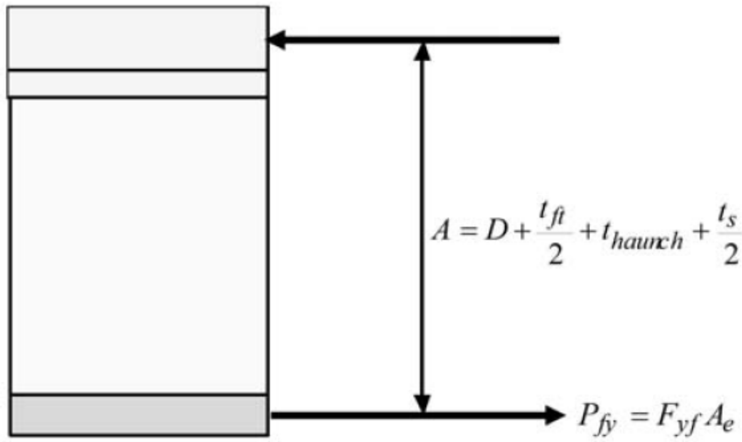

Figure E24-2.7.2.1-1
LRFD Figure [C6.13.6.1.3b-1] Calculation of the Moment Resistance Provided by the Flange Splices for Composite Sections Subject to Positive Flexure

LRFD [6.13.6.1.3b]: For composite sections subject to positive flexure, the moment resistance provided by the flange splices at the strength limit state shall be computed as **P_{fy} for the bottom flange** times the moment arm taken as the vertical distance from the mid-thickness of the bottom flange to the mid thickness of the concrete deck including the concrete haunch.

Use P_{fy} for the bottom flange = 490.92 Kip

Flange moment arm: $A_{+} := D + \frac{t_{flbL}}{2} + d_{haunch} + \frac{t_{seff}}{2} = 62.44$ in

The haunch thickness d_{haunch} is measure from the top of the web to the bottom of concrete deck

$$M_{f+} := P_{fy\_bot} \cdot \frac{A_{+}}{12} = 2554.32 \quad \text{Kip.ft}$$

$$M_{u+} = 2323.51 \quad \text{kip-ft}$$

$$M_{f+} > M_{u+} \quad \text{OK}$$

Hence, the flange splices are able to resist the applied positive moment, and the web splice will not contribute to resist any portion of moment

E24-2.7.2.2 Negative Moment

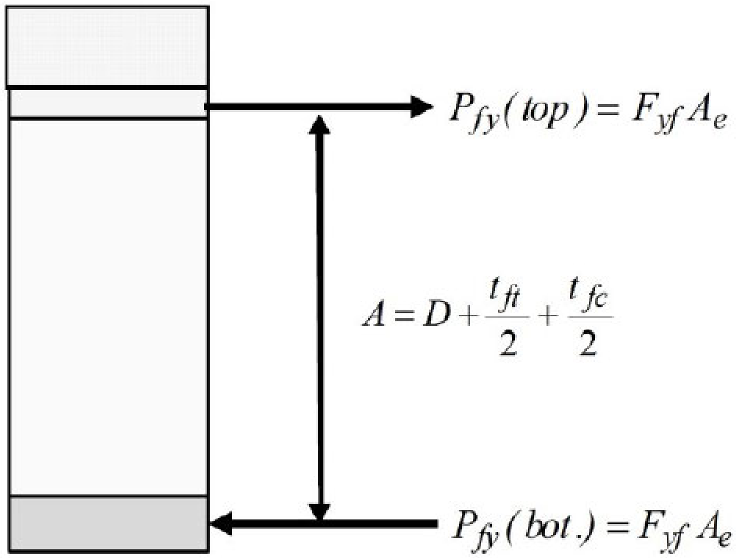

Figure E24-2.7.2.2-1
LRFD Figure [C6.13.6.1.3b-2] Calculation of the Moment Resistance Provided by the Flange Splices for Composite Sections Subject to Negative Flexure and Non-composite Sections

LRFD [6.13.6.1.3b]: For composite sections subject to negative flexure and non-composite sections subject to positive or negative flexure, the moment resistance provided by the flange splices at the strength limit state shall be computed as **P_{fy} for the top or bottom flange**, which is smaller, times the moment arm taken as the vertical distance between the mid-thickness of the top and bottom flanges.

Use the smaller value of P_{fy} for the top and bottom flange = 420.79 Kip

$P_{fy\_N} := \min(P_{fy\_top}, P_{fy\_bot}) = 420.79$ OK

Flange negative moment arm: $A_- := D + \frac{t_{flbL}}{2} + \frac{t_{fltL}}{2} = 54.81$ in

$M_{f-} := P_{fy\_N} \cdot \frac{A_-}{12} = 1922.04$ Kip.ft

$M_{u-} = -1549.67$ kip-ft

$M_{f-} > |M_{u-}|$ OK

The flange splices are able to resist the applied negative moment, and the web splice will not contribute to resist any portion of moment

E24-2.7.3 Bottom Splice Plates

E24-2.7.3.1 - Tension **LRFD [6.13.5.2]**:

LRFD [C6.13.6.1.3b] Splice plate subjected to tension is to be checked at the strength limit state for:

- Yielding on the gross section
- Fracture on the net section
- Block shear rupture

Cross section yielding

As the inner and outer splice plates do not differ by more than 10%, P_{fy} is equally divided to the inner and the outer flange splice plates

$P_{cu} := P_{fy\_bot} = 490.92$ kips

The factored tensile resistance for yielding on the gross section, P_r, is taken from **LRFD [6.8.2.1]**:

$P_r = \phi_y \cdot P_{ny}$ **LRFD [6.8.2.1-1]**

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)

ϕ_y = Resistance factor for yielding of tension members

$P_r = \phi_y \cdot F_y \cdot A_g$

$F_y = 50.00$ ksi

$\phi_y = 0.95$

For yielding of the outside splice plate P_{ro}:

$A_g := A_{out\_bot}$ $A_g = 7.00$ in^2

$P_r := \phi_y \cdot F_y \cdot A_g$ $P_r = 332.50$ kips

The outside splice plate takes half of the design load:

$P_r = 332.50$ > $\frac{P_{cu}}{2} = 245.46$ OK

For yielding of the inside splice plates P_{ri}:

$A_g := A_{inn\_bot}$ $\qquad A_g = 7.50$ $\qquad in^2$

$P_r := \phi_y \cdot F_y \cdot A_g$ $\qquad P_r = 356.25$ $\qquad$ kips

The inside splice plate takes half of the design load:

$$P_r = 356.25 \quad > \quad \frac{P_{cu}}{2} = 245.46 \qquad \text{OK}$$

Fracture in net section

The factored tensile resistance for fracture on the net section, P_r, is calculated by:

$P_r = \phi_u \cdot P_{nu}$ $\qquad$ **LRFD [6.8.2.1-2]**

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

ϕ_u = Resistance factor for fracture of tension members

$P_r = \phi_u \cdot F_u \cdot A_n \cdot R_p \cdot U$

$F_u = 65.00$

$\phi_u = 0.80$

$U := 1.0$

$R_p := 1.0$

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:

$d_{hole} = 0.938$ in

The nominal area of the outside splice plate is determined to be:

$$A_{n(out\_b)} := (b_{out} - Row\_No \cdot d_{hole}) \cdot t_{out} = 5.13 \quad in^2$$

The net area of the connecting element is limited to $0.85A_g$ **LRFD [6.13.5.2]**:

$$A_n \le 0.85 \cdot A_g$$

$$A_{g(out\_b)} := t_{out} \cdot b_{out} = 7.00 \quad in^2$$

$$A_{n(out\_b)} = 5.13 \quad in^2 \quad < \quad 0.85 \cdot A_{g(out\_b)} = 5.95 \quad in^2 \quad OK$$

$$P_r := \phi_u \cdot F_u \cdot A_{n(out\_b)} \cdot U \qquad P_r = 266.50 \quad kips$$

The outside splice plate takes half of the design flange force:

$$P_r = 266.50 \quad kips \quad > \quad \frac{P_{cu}}{2} = 245.46 \quad kips \quad OK$$

For fracture of the inside splice plates:

The nominal area is determined to be:

$$A_{n(in\_b)} := N_{inp}(b_{in} - 2 \cdot d_{hole}) \cdot t_{in} = 5.16 \quad in^2$$

The net area of the connecting element is limited to $0.85A_g$:

$$A_n \le 0.85 \cdot A_g$$

$$A_{g(in\_b)} := N_{inp} \cdot b_{in} \cdot t_{in} = 7.50$$

$$A_{n(in\_b)} = 5.16 \quad in^2 \quad < \quad 0.85 \cdot A_{g(in\_b)} = 6.38 \quad in^2 \quad OK$$

$$P_r := \phi_u \cdot F_u \cdot A_{n(in\_b)} \cdot U \qquad P_r = 268.13 \quad kips$$

The inside splice plates take half of the design flange force:

$$P_r = 268.13 \quad kips \quad > \quad \frac{P_{cu}}{2} = 245.46 \quad kips \quad OK$$

Block shear rupture LRFD [6.13.4]

A) 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 Figure E24-2.7.3.1-1. The outside splice plate will now be checked for block shear.

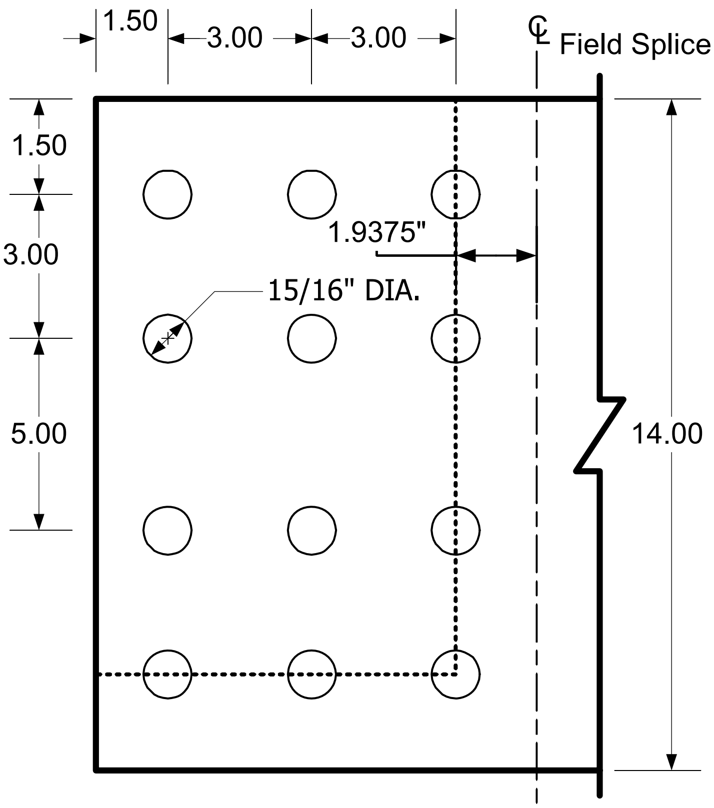

Figure E24-2.7.3.1-1
Outside Splice Plate - Failure Mode 1

Applying the factored resistance equations presented previously to the outside splice plate for failure mode 1:

Gross area along the plane resisting shear stress:

$$A_{vg} := [2\cdot(3.00) + 1.50]\cdot t_{out} \qquad A_{vg} = 3.75 \qquad in^2$$

Net area along the plane resisting shear stress:

$$A_{vn} := \left[2\cdot(3.00) + 1.50 - 2.5\cdot d_{hole}\right]\cdot t_{out} \qquad A_{vn} = 2.58 \qquad in^2$$

Net area along the plane resisting tension stress:

$$A_{tn} := \left[[2\cdot(3.00) + 5.00 + 1.50] - 3.5\cdot d_{hole}\right]\cdot t_{out} \qquad A_{tn} = 4.61 \qquad in^2$$

$U_{bs} := 1.0$

$R_{r1} := \phi_{bs} \cdot R_p \cdot (0.58 \cdot F_u \cdot A_{vn} + U_{bs} \cdot F_u \cdot A_{tn}) = 317.44$

$R_{r2} := \phi_{bs} \cdot R_p \cdot (0.58 \cdot F_y \cdot A_{vg} + U_{bs} \cdot F_u \cdot A_{tn}) = 326.69$

$R_r := \min(R_{r1}, R_{r2})$

$R_r = 317.44$ kips

Check:

$R_r = 317.44$ kips $> \dfrac{P_{cu}}{2} = 245.46$ kips OK

Failure mode 2:

See Figure Figure E24-2.7.3.1-2 for failure mode 2:

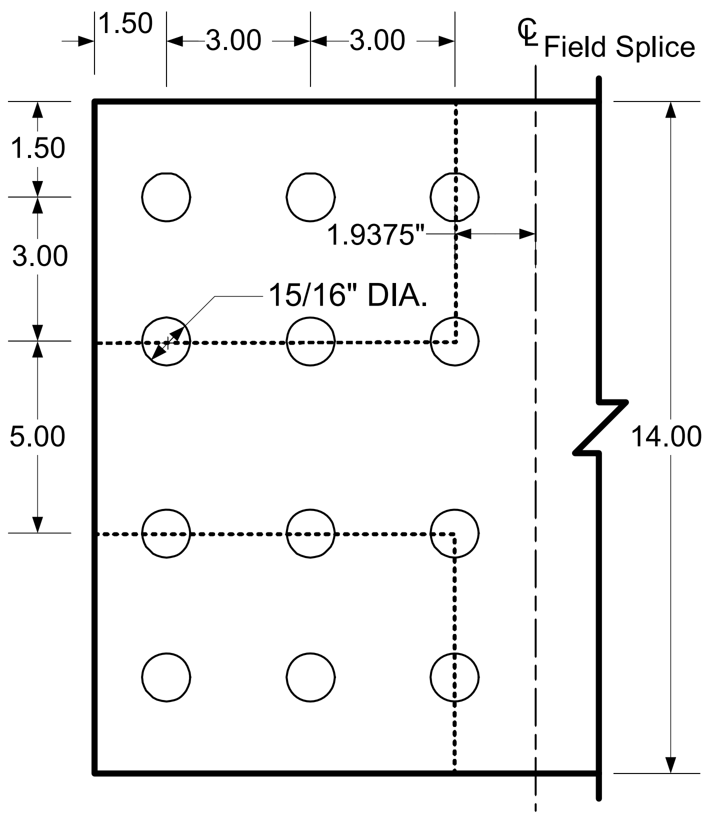

Figure E24-2.7.3.1-2
Outside Splice Plate - Failure Mode 2

$$R_r = 316.39 \quad \text{kips} \; > \; \frac{P_{cu}}{2} = 245.46 \quad \text{kips} \quad \text{OK}$$

B) Inside splice plates:

The inside splice plates will now be checked for block shear. See Figure Figure E24-2.7.3.1-3 for the assumed failure mode:

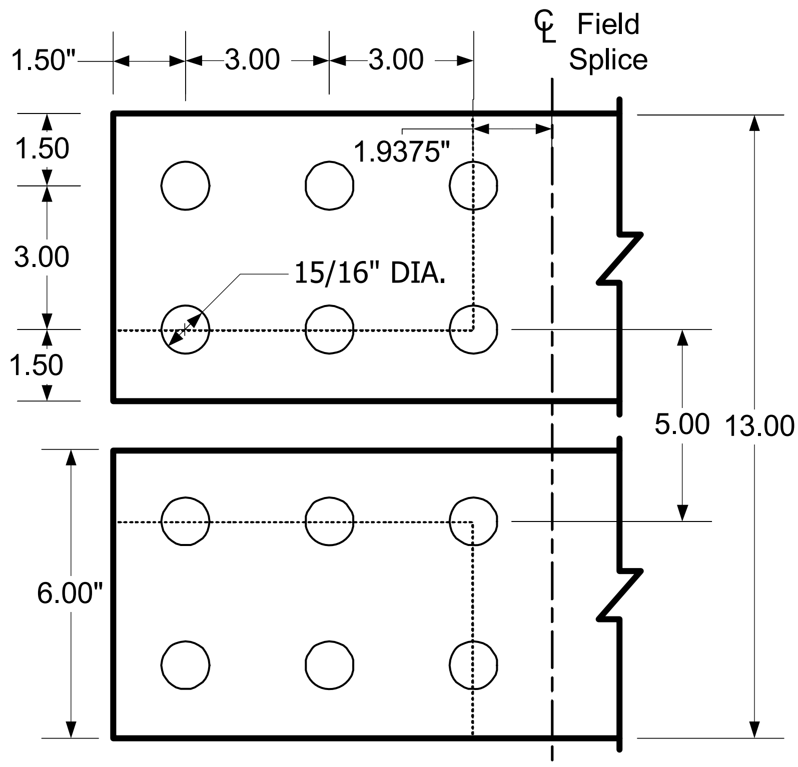

Figure E24-2.7.3.1-3
Inside Splice Plates - Block Shear Check

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:

$$R_r = 395.48 \text{ kips} > \frac{P_{cu}}{2} = 245.46 \text{ kips} \quad \text{OK}$$

C) Girder bottom flange:

The girder bottom flange will now be checked for block shear. See Figure Figure E24-2.7.3.1-4 for the assumed failure mode:

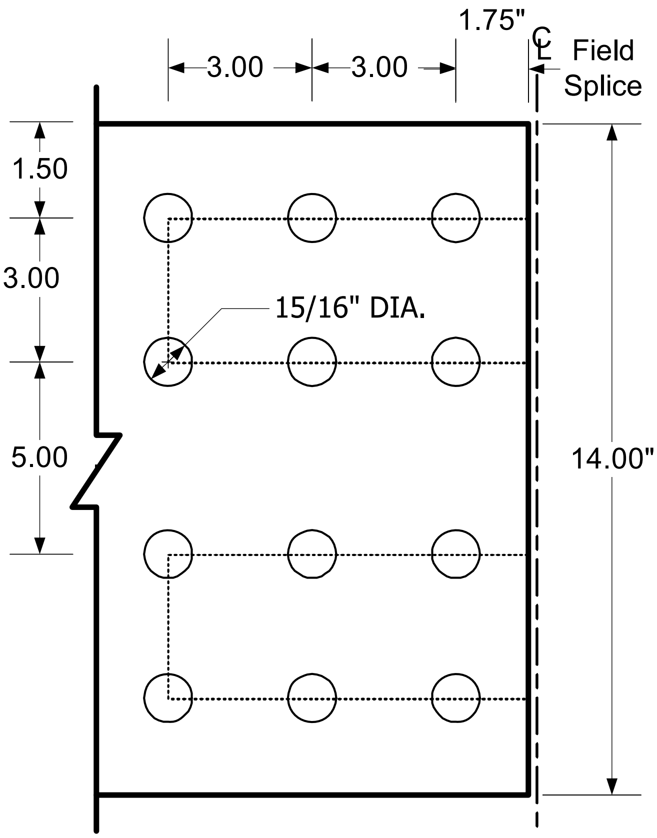

Figure E24-2.7.3.1-4
Bottom Flange - Block Shear Check

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:

$$R_r = 758.37 \text{ kips} > P_{cu} = 490.92 \text{ kips} \quad \text{OK}$$

E24-2.7.3.2 - Compression

Flange splice plate subjected to compression at the strength limit state is to be checked for yielding on the gross section of the plates. However, no need to check this requirement as it is satisfied in the tension check

Also, no need to check the plate buckling due to the compression load, as the bolt spacing is close.

LRFD [6.13.2.6.3] To prevent buckling in compression, the maximum spacing between bolts shal not exceed 12.t, and the gage, g, between adjacent lines of bolts shall not exceed 24.t

Where:

t = Thickness of the thinner outside plate or shape (in)

$t_{buckling} := \min\left(t_{out}, t_{fltL}\right) = 0.50$ in

$12 \cdot t_{buckling} = 6.00$ in

$24 \cdot t_{buckling} = 12.00$ in

Both requirements are met and buckling will not occure.

E24-2.7.4 - Checking Flexural Members at the Strength Limit for Constructibility

LRFD 6.10.1.8-1 should be satisfied at all cross-sections containing holes in the tension flange. In this example, this equation was checked in separate calculation and it is satisfied for both flanges of the girder at the splice at the stength limit state.

E24-2.8 Service Limit State Design of the Flange Splice Plates

E24-2.8.1 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 during deck casting, whichever governs **LRFD [6.13.6.1.3a]**.

LRFD [C6.13.6.1.3b] 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. Slip of the connection cannot occur unless slip occurs on both planes.

Furthermore, for slip-critical connections, the nominal slip resistance of a bolt shall not be adjusted for the effect of the fillers. The resistance to slip between filler and either connected part is comparable to that which would exist between the connection parts if fillers were not present.

The factored resistance of bolt for slip-critical connections, R_r, is calculated from **LRFD [6.13.2.2 & 6.13.2.8]**:

$$R_r = R_n \cdot 1$$

Where R_n is the nominal resistance:

$$R_n = K_h \cdot K_s \cdot N_s \cdot P_t \qquad \textbf{LRFD [6.13.2.8-1]}$$

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 [6.13.6.1.3a]**) 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]**.

$$K_h := 1.0$$

$$K_s := 0.50$$

$$N_s := 2$$

$$P_t := 39.0 \qquad \text{kips}$$

$R_n := K_h \cdot K_s \cdot N_s \cdot P_t$ $R_n = 39.00$ kip / Bolt

E24-2.8.1.1 Service II Positive Moment

For composite section subject to positive moment, use slip resistance of the bottom flange splice bolts.

The factored slip resistance of the bottom flange splice with 12 bolts R_r

$R_r := R_n \cdot 12 = 468.00$ kip

Flange Moment Arm (A_+) as calculated before:

$A_+ = 62.44$ in

Service II Positive Moment M_+:

$M_+ := 1 \cdot (M_{NDL} + M_{CDL}) + 0 \cdot M_{FWS} + 1.3 \cdot M_{PLL+IL} = 1689.38$ kip-ft

$M_{slip\_bot} := R_r \cdot \frac{A_+}{12} = 2435.06$ kip-ft $>$ $M_+ = 1689.38$ kip-ft OK

E24-2.8.1.2 Service II Negative Moment

For composite section subject to negative moment, use slip resistance of the bottom or top flange splice bolts, which is smaller.

The factored slip resistance of the top flange splice with 12 bolts R_r

$R_r := R_n \cdot 12 = 468.00$ kip

Flange Moment Arm (A_-) as calculated before:

$A_- = 54.81$ in

Service II Negative Moment M_+:

$M_- := 1 \cdot (M_{NDL} + M_{CDL}) + 1 \cdot M_{FWS} + 1.30 \cdot M_{NLL+IL} = -1158.79$ kip-ft

$M_{slip\_top} := R_r \cdot \frac{A_-}{12} = 2137.69$ kip-ft $>$ $M_- = -1158.79$ kip-ft OK

E24-2.8.1.3 Deck Casting

For non-composite section, use the slip resistance of the bottom or top flange splice bolts, which is smaller. The deck casting will not control in this example.

E24-2.8.2 Control of Permanent Deformation

When the combined area of the inside and outside flange splice plates is greater than the area of the smaller bottom flange at the point of splice, the permanent deflection under the Service II load combination need not be checked.

E24-2.9 Filler Plates

LRFD [6.13.6.1.4] The specified minimum yield strength of the fillers 0.25 inch or greater in thickness should not be less than the larger of 70 percent of the specified minimum yield strength of the connected plate and 36 ksi.

E24-2.10 Web Design

E24-2.10.1 Web Splice Design Loads

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 distribution factors.

Dead load shears:

Non-composite:

$V_{NDL} := -58.4$ kips

Composite:

$V_{CDL} := -7.8$ kips

Future wearing surface:

$V_{FWS} := -7.4$ kips

Live Load shears:

HL-93 positive:

$V_{PLL} := 16.2$ kips

HL-93 negative:

$V_{NLL} := -91.6$ kips

E24-2.10.2 Web Splice Configuration

Two vertical rows of bolts with sixteen bolts per row will be used. The typical bolt spacings, both horizontally and vertically, are as shown in Figure E24-2.10.2-1. 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.

The splice plates shall be extended as near as practical for the full depth between flanges without impinging on bolt assembly clearance

For bolted web splices with thickness differences of 0.0625 inch or less, filler plates should not be provided

Web splice plate thickness: $t_{wp} := 0.375$ in

Web splice plate length: $L_{wp} := 48$ in

Number of web splice plates: $N_{wp} := 2$ Plates

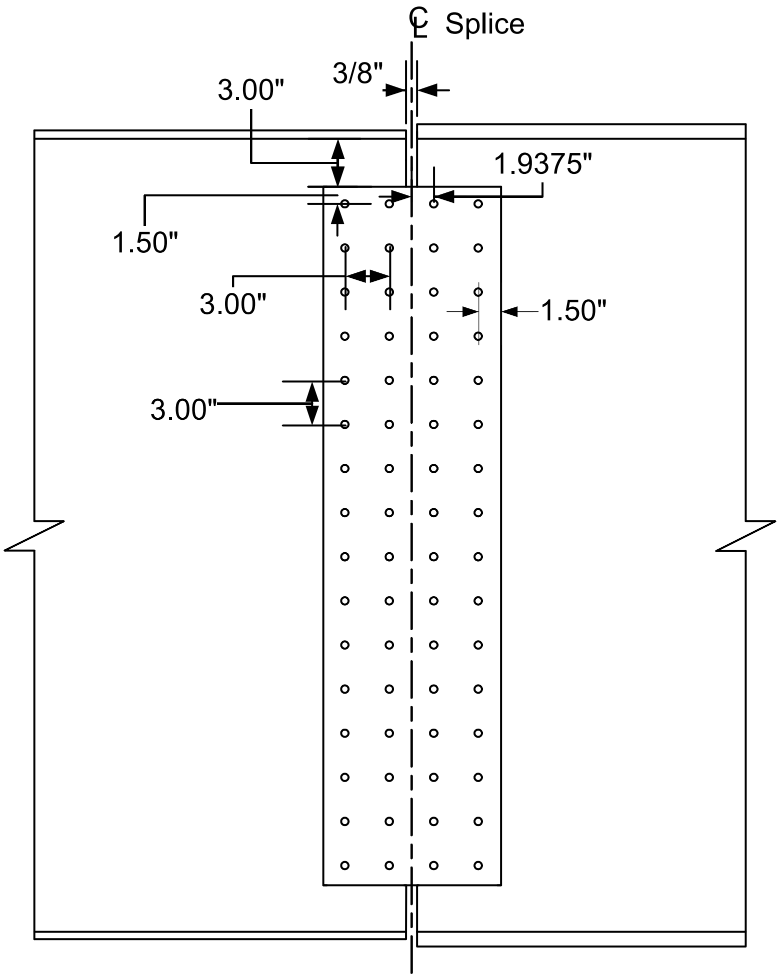

Figure E24-2.10.2-1
Web Splice

3" spacing was selected, but 4.5" may work.

E24-2.10.3 Strength Limit State Design of the Web Plates

In this example, the moment resistance of the flanges is sufficient to resist the factored moment at the strength limit state, so the web will not contribute to resist any moment.

Should the factored moments exceed the moment resistance provided by the flange splices, the web splice is assumed to resist the additional moment as addressed in **LRFD [6.13.6.1.3c]** in addition to its design shear.

E24-2.10.3 .1 Bolt Design

E24-2.10.3.1.1 Number of Bolts and Spacing

- **LRFD [6.13.6.1.3c]** As a minimum, web splice plates and their connections shall be designed at the strength limit state for a design web force taken equal to the smaller factored shear resistance of the web at the point of splice
- The factored shear resistance of the bolts should be based on threads included in the shear planes, unless the web splice-plate thickness exceeds 0.5 inch.
- The small moment induced by the eccentricity of the web may be ignored at all limit states.
- As a minimum, two vertical rows of bolts spaced at maximum spacing for sealing bolts specified in **LRFD [6.13.2.6.2]** should be provided, with a closer spacing and or additional rows provided only as needed.
- Unlike the tension splice connection, the length reduction factor of 0.83 is not applied to the web splice when the web splice connection exceeds 38.0 inch.
- The total number of bolts on one side of the splice are determined by dividing the smaller shear resistance at the point of splice, V_r, by the factored shear resistance of the bolts. Then the bearing resistance of the flange splice bolts holes shall be checked at the strength limit state.

The smaller shear resistance of web as calculated in EX24-1:

$V_n := 305.6$ kips

$V_r := \phi_v \cdot V_n = 305.60$ kips

Using a splice plate at each side of the web, the connection is double shear connection

$N_{st} = 2.00$ Planes

When threads are included **LRFD Eq. [6.13.2.7.2]**

$R_n := 0.45 A_b \cdot F_{ub} \cdot N_{st} = 64.94$ kips

$\phi_s \cdot R_n = 51.95$ kips

Number of bolts in web:

$$N_b := \frac{V_r}{(\phi_s \cdot R_n)} = 5.88 \quad \text{Bolts}$$

The minimum spacing LRFD[6.13.2.6.1]:
The minimum spacing between centers of bolts in standard holes for sealing against the penetration shall be no less than three times the diameter of the bolt.

$d_b = 0.875$ in

$s_{min} := 3 \cdot d_b$ $s_{min} = 2.63$ in

Using 3 inch spacing will meet the minimum spacing requirement

$s := 3.00$ in

The 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 when the bolts are not staggered (for example, the bolts along the edges of the plate parallel to the direction of the applied force):

$s \le (4.0 + 4.0 \cdot t) \le 7.0$ When the bolts are not staggered **LRFD [6.13.2.6.2-1]**:

Where:

t = Thickness of the splice plate (in)

$t_{wp} = 0.3750$ in

Maximum spacing for sealing at the edge parallel to the applied force:

$4 + 4 \cdot t_{wp} = 5.50$ in

$s \le 5.5 \le 7.00$ 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:

$s_{end} := 3.875$ in

Maximum spacing for sealing at the end of the splice plate:

$4.0 + 4.0 \cdot t_{wp} = 5.50$ in

$s_{end} \le 5.5 \le 7.00$ OK

The 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.

The end distance LRFD [6.13.2.6.5]:

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 Figure E24-2.10.2-1, 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.

$D_{max} \le 8 \cdot t \le 5.00$ in

$t := t_{wp}$ $t_{wp} = 0.3750$ in

$8 \cdot t_{wp} = 3.00$ in

The maximum distance from the corner bolts to the corner of the splice plate is equal to:

$D_{max} := \sqrt{1.5^2 + 1.5^2} = 2.12$ in

$2.12 \cdot in \le 4.0 \cdot in$ OK

Therefore, the total number of web bolts on each side of the splice required to meet the maximum bolt spacing, assuming two vertical rows per side with sixteen bolts per row :

$N_b := 32$ Bolts per side

$N_{bl} := 16$ Bolts per line

E24-2.10.3.1.2 Bearing at Bolt Holes

LRFD [6.13.2.9]

Since the sum of splice plates thickness times F_u is greater than the web splice plate times F_u, the left girder web controls the bearing resistance of the connection. In addition, the flange splice plates are sufficient to resist the moment without cotribution from the web, therefore, only bearing parallel to the shear resistance is to be checked.

The flange is sufficient to resist the moment, then:

$T := V_r$

$d_b = 0.875$ in $\qquad 2 \cdot d_b = 1.75$ in

$d_{hole} = 0.938$ in

For the two bolts at the bottom of the web plate, the edge distance is 3". Therefore, the clear end distance between the edge of the hole and the end of the web in the direction of the applied force:

$$L_{c_1} := 3 - \frac{d_{hole}}{2} \qquad L_{c_1} = 2.53 \text{ in}$$

$L_{c_1} > 2 \cdot d_b$ Then $R_n = 2.4dtF_u$ **LRFD [6.13.2.9-1]**

The nominal resistance of the bottom bolt holes (two holes) is computed as follows:

$$R_{n_1} := 2 \cdot \left(2.4 \cdot d_b \cdot t_w \cdot F_u\right) \qquad R_{n_1} = 136.50 \text{ kips}$$

The vertical 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:

$$L_{c_2} := 3.00 - d_{hole} \qquad L_{c_2} = 2.06 \text{ in}$$

$L_{c_2} > 2 \cdot d_b$ Then $R_n = 2.4dtF_u$ **LRFD [6.13.2.9-1]**

The nominal resistance for the remaining bolt holes (30 holes) is computed as follows:

$$R_{n_2} := 30 \cdot \left(2.4 \cdot d_b \cdot t_w \cdot F_u\right) \qquad R_{n_2} = 2047.50 \text{ kips}$$

The total nominal resistance of the bolt holes is:

$R_n := R_{n_1} + R_{n_2}$ $R_n = 2184.00$ kips

$\phi_{bb} = 0.80$

$R_r := \phi_{bb} \cdot R_n$ $R_r = 1747.20$ kips

Check:

$V_r = 305.60$ kips $<$ $R_r = 1747.20$ kips OK

E24-2.10.3.2 Shear Resistance of the Connection Element

LRFD [6.13.6.1.3c]

The design web force at the strength limit state shall not exceed the lesser of the factored shear resistance of the web splice plates determined from:

Shear yielding of the connection element
Shear rupture of the connection element
Block shear resistance of the connection element (normally does not govern)

Shear yielding of the connection element LRFD [6.13.5.3]

For shear yielding, the factored shear resistance of the connection element shall be taken as:

$R_r := \phi_v \cdot 0.58 \cdot F_y \cdot A_{vg}$

ϕ_v = Resistance factor for shear **LRFD [6.5.4.2]**

A_{vg} = Gross area of the connection element subject to shear (in^2)

F_y = Specified minimum yield strength of the connection element (ksi)

$A_{vg} := t_{wp} \cdot L_{wp} = 18.00$ in^2

Using two plates total ($N_{wp} = 2$) with one plate on each side of the web

the shear yielding resistanceis

$R_r := \phi_v \cdot 0.58 \cdot F_y \cdot N_{wp} A_{vg} = 1044.00$ kips

$R_r > V_r$ kips OK

Shear rupture of the connection element LRFD [6.13.5.3]

For shear rupture, the factored shear resistance of the connection element shall be taken as:

$R_r := \phi_{vu} \cdot 0.58 \cdot R_p \cdot F_u \cdot A_{vn}$ **LRFD [6.13.5.3-2]**

ϕ_{vu} = Resistance factor for shear rupture of connection elements **LRFD [6.5.4.2]**

A_{vn} = Net area of the connection element subject to shear (in^2)

F_u = Tensile strength of the connection element (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

$A_{vn} := t_{wp} \cdot (L_{wp} - 16 \cdot d_{hole}) = 12.38$ in^2

Using two plates with one plate on each side of the web

the shear yielding resistance

$R_r := \phi_{vu} \cdot 0.58 \cdot R_p \cdot F_u \cdot N_{wp} A_{vn} = 746.46$ kips

$R_r > V_r$ kips OK

Block shear rupture of the connection element LRFD [6.13.4]

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 shear resistance was determined to be:

$V_r = 305.60$ kips

Gross area along the plane resisting shear stress:

$A_{vg} := N_{wp} \cdot (L_{wp} - 1.50) \cdot t_{wp}$ $A_{vg} = 34.88$ in^2

Net area along the plane resisting shear stress:

$A_{vn} := N_{wp} \cdot \left[L_{wp} - 1.50 - 15.50 \cdot (d_{hole})\right] \cdot t_{wp}$ $A_{vn} = 23.98$ in^2

Net area along the plane resisting tension stress:

$A_{tn} := N_{wp} \cdot \left[1.50 + 3.0 - 1.5 \cdot \left(d_{hole}\right)\right] \cdot t_{wp}$ $\qquad A_{tn} = 2.32$ $\qquad in^2$

$U_{bs} := 1.0$

$R_{r1} := \phi_{bs} \cdot R_p \cdot \left(0.58 \cdot F_u \cdot A_{vn} + U_{bs} \cdot F_u \cdot A_{tn}\right) = 843.79$

$R_{r2} := \phi_{bs} \cdot R_p \cdot \left(0.58 \cdot F_y \cdot A_{vg} + U_{bs} \cdot F_u \cdot A_{tn}\right) = 929.76$

$R_r := \min\left(R_{r1}, R_{r2}\right)$

$R_r = 843.79$ kips

Check:

$V_r = 305.60$ kips $<$ $R_r = 843.79$ kips OK

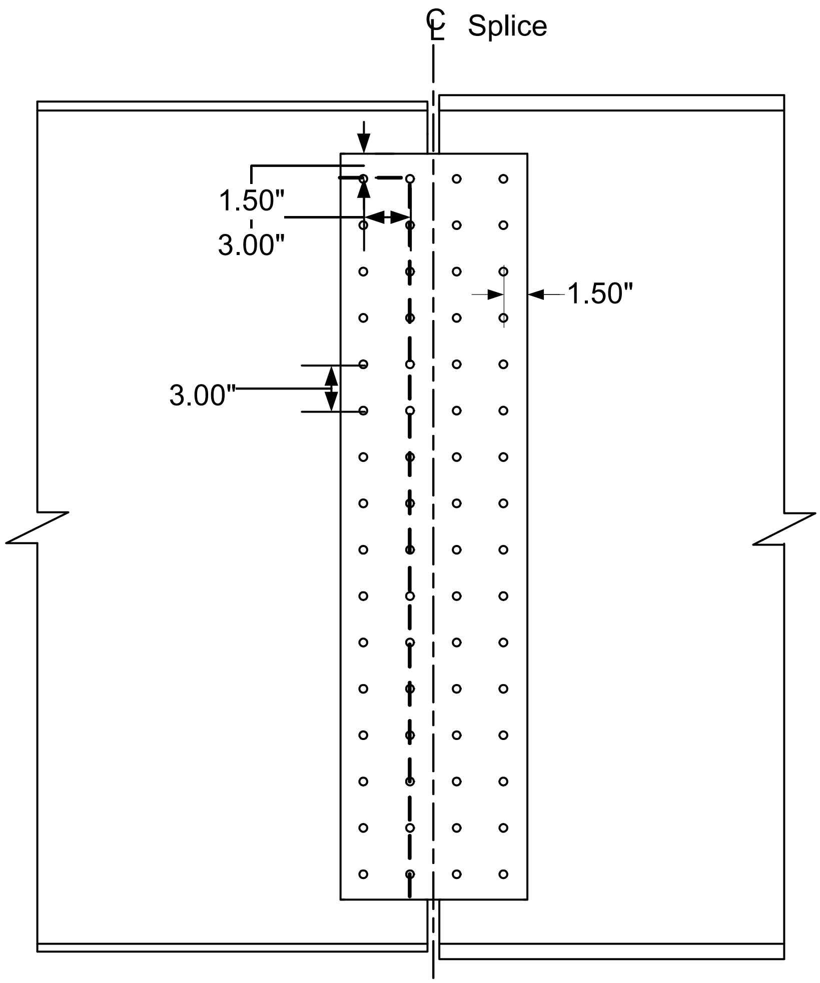

Figure E24-2.10.3.2-1
Block Shear Failure Mode - Web Splice Plate

E24-2.10.4 Service Limit State Design of the Flange Splice Plates

LRFD [6.13.6.1.3c] The factored shear for checking slip shall be taken as the shear in the web at the point of the splice under Load Combination Service II or the shear in the web due to the deck casting sequence, whichever governs.

Should the nominal slip resistance provided by the flange bolts not be sufficient to resist the flange slip force due to the factored moment at the point of splice as determined in Article **LRFD [6.13.6.1.3b]**, the web splice bolts shall be, instead, be checked for slip under a web slip force taken equal to the vector sum of the factored shear and the portion of the flange slip force that exceeds the nominal slip resistance of the flange bolts.

Furthermore, Positive and negative shear under Load Combination Service II, which is greater, should be investigated.

By inpsection, the Service II negative shear controls

$$V_{N\_ServiceII} := 1 \cdot (V_{NDL} + V_{CDL}) + 1 \cdot (V_{FWS}) + 1.3 \cdot (V_{NLL}) = -192.68$$

The nominal resistance of one bolt:

$$R_n := K_h \cdot K_s \cdot N_s \cdot P_t$$ **LRFD [6.13.2.8-1]**

$R_n = 39.00$ kip/Bolt

The factored slip resistance of the web splice of 32 bolts R_r

$R_r := R_n \cdot 32 = 1248.00$ kip

$R_r > |V_{N\_ServiceII}|$ kip OK

E24-2.11 Draw Schematic of Final Bolted Field Splice Design

Figure E24-2.11-1 shows the final bolted field splice as determined in this design example.

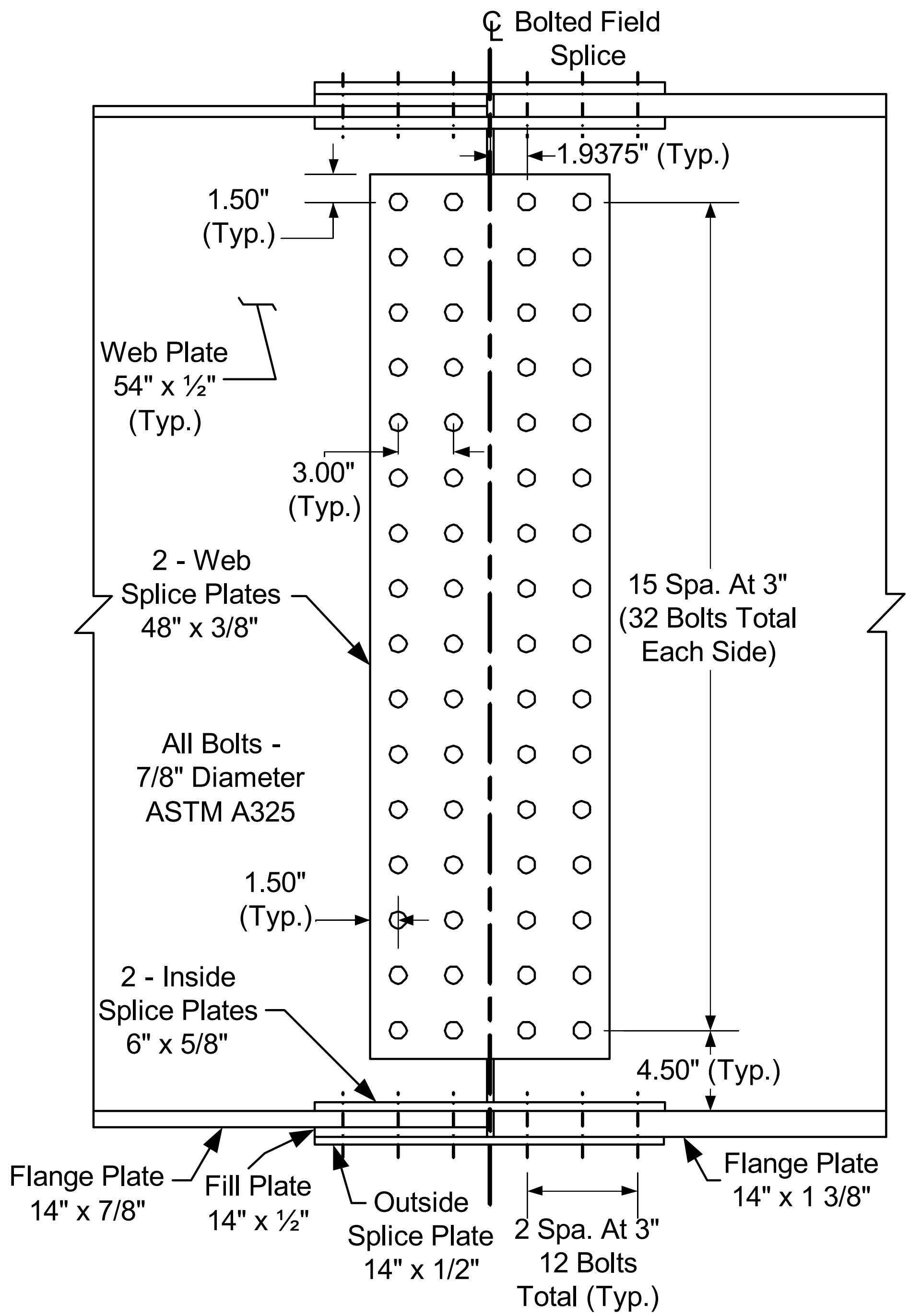

Figure E24-2.11-1
Final Bolted Field Splice Design

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27.1 General

Bridges supported in the conventional way by abutments and piers require bearings to transfer girder reactions without overstressing the supports, ensuring that the bridge functions as intended. Bridges usually require bearings that are more elaborate than those required for building columns, girders and trusses. Bridge bearings require greater consideration in minimizing forces caused by temperature change, friction and restraint against elastic deformations. A more detailed analysis in bridge bearing design considers the following:

- Bridges are usually supported by reinforced concrete substructure units, and the magnitude of the horizontal thrust determines the size of the substructure units. The coefficient of friction on bridge bearings should be as low as possible.

- Bridge bearings must be capable of withstanding and transferring dynamic forces and the resulting vibrations without causing eventual wear and destruction of the substructure units.

- Most bridges are exposed to the elements of nature. Bridge bearings are subjected to more frequent and greater total expansion and contraction movement due to changes in temperature than those required by buildings. Since bridge bearings are exposed to the weather, they are designed as maintenance-free as possible.

WisDOT policy item:

The temperature range considered for steel girder superstructures is -30°F to 120°F. A temperature setting table for steel bearings is used for steel girders; where 45°F is the neutral temperature, resulting in a range of 120° - 45° = 75° for bearing design.

The temperature range considered for prestressed concrete girder superstructures is 5°F to 85°F. Using an installation temperature of 60° for prestressed girders, the resulting range is 60° - 5° = 55° for bearing design. For prestressed girders, an additional shrinkage factor of 0.0003 ft/ft shall also be accounted for. (Do not include prestressed girder shrinkage when designing bearings for bridge rehabilitation projects). No temperature setting table is used for prestressed concrete girders.

See the Standard for Steel Expansion Bearing Details to determine bearing plate “A” sizing (steel girders) or anchor plate sizing (prestressed concrete girders). This standard also gives an example of a temperature setting table for steel bearings when used for steel girders.

WisDOT policy item:

According to **LRFD [14.4.1]**, the influence of dynamic load allowance need not be included for bearings. However, dynamic load allowance shall be included when designing bearings for bridges in Wisconsin. Apply dynamic load allowance in **LRFD [3.6.2]** to HL-93 live loads as stated in **LRFD [3.6.1.2, 3.6.1.3]** and distribute these loads, along with dead loads, to the bearings.

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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.

WisDOT policy item:

For all Interstate structures, the 42SS parapet shall be used. For all STH and USH structures with a posted speed >= 45 mph, the 42SS parapet shall be used.

The timeline for implementation of the above policy is:

- All contracts with a letting date after December 31, 2019.
  (This is an absolute, regardless of when the design was started.)

- All preliminary designs starting after October 1, 2017
  (Even if the let is anticipated to be prior to December 31, 2019.)

Contact BOS should the 42" height adversely affect sight distance, a minimum 0.5% grade for drainage cannot be achieved, or for other non-typical situations.

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 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 is TL-3 under MASH.

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 has not been rated under MASH.

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. Although 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), FHWA has since restricted its use as indicated above.

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 is TL-2 under MASH.

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 are TL-2 under MASH.

14. The type "F" steel railing, as shown in the Standard Details of Chapter 40, shall not 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 informational purposes only.

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) 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 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 FDM 11-45-1 and 11-45-2 for Traffic Barrier, Crash Cushions, and Roadside Barrier Design Guidance. See FDM 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 30.2 (i.e., cast-in-place anchors are used at exterior parapet location). See Standards for Parapet Footing and Lighting Detail 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 11-45-2.3.1.1 and 11-45-2.3.6.2.3 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 30.4 and what is required in FDM 11-45-2.3.1.1 and 11-45-2.3.6.2.3 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 for 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 for 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.

$f'_c := 3.5$ culvert concrete strength, ksi

$f_y := 60$ reinforcement yield strength, ksi

$E_s := 29000$ modulus of elasticity of steel, ksi

$skew = 0.0$ skew angle, degrees

$H_s = 4.00$ depth of backfill above top edge of top slab, ft

$w_c := 0.150$ weight of concrete, kcf

$cover_{bot} := 3$ concrete cover (bottom of bottom slab), in

$cover := 2$ concrete cover (all other applications), in

$LS_{ht} := 2.2$ live load surcharge height, ft (See Sect. 36.4.4)

Resistance factors, reinforced concrete cast-in-place box structures, **LRFD [Table 12.5.5-1]**

$\phi_f := 0.9$ resistance factor for flexure

$\phi_v := 0.85$ resistance factor for shear

Calculate the span lengths for each cell (measured between centerlines of walls)

$$S_1 := W_1 + \frac{1}{12}\left(\frac{t_{win}}{2} + \frac{t_{wex}}{2}\right) \qquad \boxed{S_1 = 13.00} \text{ ft}$$

$$S_2 = W_2 + \frac{1}{12}\left(\frac{t_{wex}}{2} + \frac{t_{win}}{2}\right) \qquad \boxed{S_2 = 13.00} \text{ ft}$$

Verify that the box culvert dimensions fall within WisDOT's minimum dimension criteria. Per Sect. 36.2, the minimum size for pedestrian underpasses is 8 feet high by 10 feet wide. The minimum size for cattle underpass is 6 feet high by 5 feet wide. A minimum height of 5 feet is desirable for cleanout purposes.

Does the culvert meet the minimum dimension criteria? $\boxed{check = \text{"OK"}}$

Verify that the slab and wall thicknesses fall within WisDOT's minimum dimension criteria. Per Sect. 36.5, the minimum thickness of the top and bottom slab is 6.5 inches. Per Sect. 36.5 [Table 36.5-1], the minimum wall thickness varies with respect to cell height and apron wall height.

Minimum Wall Thickness (Inches)	Cell Height (Feet)	Apron Wall Height Above Floor (Feet)
8	< 6	< 6.75
9	6 to < 10	6.75 to < 10
10	10 to > 10	10 to < 11.75
11		11.75 to < 12.5
12		12.5 to 13

Table 36.5-1
Minimum Wall Thickness Criteria

Do the slab and wall thicknesses meet the minimum dimension criteria? $\boxed{check = \text{"OK"}}$

Since this example has more than 2.0 feet of fill, edge beams are not req'd, **LRFD [C12.11.2.1]**

E36-1.2 Modulus of Elasticity of Concrete Material

Per Sect. 36.2.1, use f'_c = 3.5 ksi for culverts. Calculate value of E_C per **LRFD [C5.4.2.4]**:

$\boxed{K_1 := 1}$ $E_{c\_calc} := 33000 \cdot K_1 \cdot w_c^{1.5} \cdot \sqrt{f'_c}$ $\boxed{E_{c\_calc} = 3586.616}$ ksi

$E_c := 3600$ ksi modulus of elasticity of concrete, per Sect. 9.2

E36-1.3 Loads

$\gamma_s := 0.120$ unit weight of soil, kcf

Per Sect. 36.5, a haunch is provided only when the slab depth required at the interior wall is more than 2 inches greater than that required for the remainder of the span. Minimum haunch depth and length is 6 inches. Haunch depth is increased in 3 inch increments. For the first iteration, assume there are no haunches.

$h_{hau} := 0.0$ haunch height, in

$l_{hau} := 0.0$ haunch length, in

$wt_{hau} = 0.0$ weight of one haunch, kip

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of concrete surfaces and exposure of reinforcing steel, or disintegration of masonry jointing.

- Dolphins - Where depth of water and other conditions are suitable, the driving of pile clusters may be considered. Such clusters have the piles lashed together with cable to promote integral action. The clusters should be flexible to be effective in absorbing impact through deflection.

- Cellular dolphins - May be filled with concrete, loose materials or materials suitable for grouting.

- Floating shear booms - Where the depth of water or other conditions precludes the consideration of dolphins or integral pier protection, floating sheer booms may be used. These are suitably shaped and positioned to protect the pier and are anchored to allow deflection and absorption of energy. Anchorage systems should allow for fluctuations in water level due to stream flow or tidal action.

- Hydraulic devices - Such as suspended cylinders engaging a mass of water to absorb or deflect the impact energy may be used under certain conditions of water depth or intensity of impact. Such cylinders may be suspended from independent caissons, booms projecting from the pier or other supports. Such devices are customarily most effective in locations subject to little fluctuations of water levels.

- Fender systems - Constructed using piling with horizontal wales, is a common means of protection where water depth is not excessive and severe impacts are not anticipated.

- Other types of various protective systems have been successfully used and may be considered by the Engineer. Criteria for the design of protective systems cannot be specified to be applicable to all situations. Investigation of local conditions is required in each case, the results of which may then be used to apply engineering judgment to arrive at a reasonable solution.

38.4 Overpass Structures

Highway overpass structures are placed when the incidences of train and vehicle crossings exceeds certain values specified in the *Facilities Development Manual* (*FDM*). The separation provides a safer environment for both trains and vehicles.

In preparing the preliminary plan which will be sent to the railroad company for review and approval several items of data must be determined.

- Track Profile - In order to maintain clearances under existing structures when the track was upgraded with new ballast, the railroad company did not change the track elevation under the structure causing a sag in the gradeline. The track profile would be raised with a new structure and the vertical clearance for the structure should consider this.

- Drainage - Hydraulic analysis is required if any excess drainage will occur along the rail line or into existing drainage structures. Deck drains shall not discharge onto railroad track beds.

- Horizontal Clearances - The railroad system is expanding just as the highway system. Contact the railroad company for information about adding another track or adding a switching yard under the proposed structure.

- Safety Barrier – The Commissioner of Railroads has determined that the Transportation Agency has authority to determine safety barriers according to their standards. The railroad overpass parapets should be designed the same as highway grade separation structures using solid parapets (Type “SS” or appropriate) and chain link fencing when sidewalks are present.

38.4.1 Preliminary Plan Preparation

Standard for Highway over Railroad Design Requirements shows the minimum dimensions for clearances and footing depths. These should be shown on the Preliminary Plan along with the following data.

- Milepost and Direction - Show the railroad milepost and the increasing direction.

- Structure Location - Show location of structure relative to railroad right of way. (Alternative is to submit Roadway Plan).

- Footings - Show all footing depths. Minimum footing depth requirements are shown on the Standard for Highway over Railroad Design Requirements.

- Drainage Ditches - Show ditches and direction of flow.

- Utilities - Show all utilities that are near structure footings and proposed relocation is required.

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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.

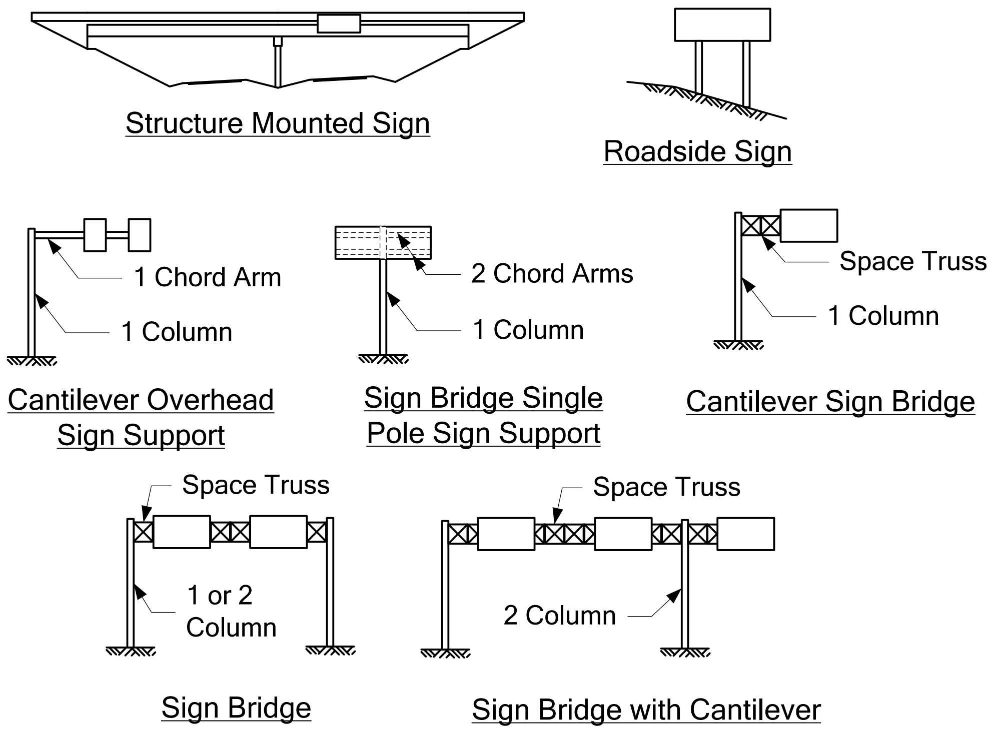

Figure 39.1-1
Sign Support Structures

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) 11-55-20.7 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 13.4.10.

Overhead sign structures, for new and replacement structures only, are to have a minimum vertical clearance of 20’-0” above the roadway for the Oversize/Overweight (OSOW) High Clearance Route and 18’-3” for all other routes. Reference 39.4.2 for additional vertical clearance requirements when catwalk or lighting is designed with a sign bridge. See FDM 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 directly to the side of a structure.

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 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.

39.5.2 Overhead Sign Supports

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 FDM 11-55-20 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 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 the Standard for Cantilever Truss Footing. The wings on this single shaft footing are used to help resist torsion. If a cantilever sign bridge exceeds the criteria/limitations (shown on the Standard for Galvanized Steel Cantilever Sign Truss), 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 FDM 11-55-20. In addition, Section 641 of the Standard Specifications outlines the design/construction aspects of these structures.

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40.1 General

New bridges are designed for a minimally expected life of 75 years. Preliminary design considerations are site conditions, structure type, geometrics, and safety. Refer to Bridge Manual Chapters 9 and 17 for Materials and Superstructure considerations, respectively. Comprehensive specifications and controlled construction inspection are paramount to obtaining high quality structures. Case history studies show that adequately consolidated and properly cured concrete with low water-cement ratios and good air void systems have lower absorption rates and provide greater resistance to scaling and chloride penetration under heavy traffic and exposure to de-icing chemicals. Applying protective surface treatments to new decks improves their resistance to first year applications of de-icing chemicals.

Most interstate and freeway structures are not subject to normal conditions and traffic volumes. Under normal environmental conditions and traffic volumes, original bridge decks have an expected life of 40 years. Deck deterioration is related to the deck environment which is usually more severe than for any of the other bridge elements. Decks are subjected to the direct effects of weather, the application of chemicals and/or abrasives, and the impact of vehicular traffic. For unprotected bar steel, de-icing chemicals are the primary cause of accelerated bridge deck deterioration. Chlorides cause the steel to corrode and the corrosion expansion causes concrete to crack along the plane of the top steel. Traffic breaks up the delaminated concrete leaving potholes on the deck surfaces. In general, deck rehabilitation on Wisconsin bridges has occurred after 15 to 22 years of service due to abnormally high traffic volumes and severe environment.

Full depth transverse floor cracks and longitudinal construction joints leak salt water on the girders below causing deterioration and over time, section loss.

Leaking expansion joints allow salt water seepage which causes deterioration of girder ends and steel bearings located under them. Also, concrete bridge seats will be affected in time. Concrete bridge seats should be finished flat, and sealed with a penetrating epoxy coating.

Bridges being designed with staged construction, whether new or rehabilitation, shall satisfy the requirements of LRFD (or LFD, if applicable) for each construction stage. Utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by Chief Structures Development Engineer at WisDOT.

40.3 Bridge Replacements

Bridge preservation and rehabilitation is preferred over bridge replacement if the final structure provides adequate serviceability. Ideal bridge preservation strategy is explained in the WisDOT Bridge Preservation Policy Guide (BPPG). This guide should be followed as closely as possible, considering estimated project costs and funding constraints.

See Faculties Design Manual (FDM) 11-40-1.5 for policies regarding necessary bridge width* and structural capacity.

* 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 ensuring some level of acceptable serviceability; however, structure preservation as explained in the Bridge Preservation Policy Guide (BPPG) should be followed as closely as possible, considering estimated project costs and funding constraints.

The first consideration for any bridge rehabilitation decision is whether its geometry and load carrying capacity are adequate to safely carry present and projected traffic. Information which is helpful in determining structure adequacy includes structure inspection history, inventory data, traffic projections, maintenance history, capacity and route designations. The methods of rehabilitation 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.

WisDOT policy item:

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.

The Regions are to evaluate bridge deficiencies when the bridge is placed in the program to ensure that rehabilitation will remove all structural deficiencies. Bureau of Structures (BOS) concurrence with all proposed bridge rehabilitation is required. See FDM 11-40-1.5 for policies regarding bridge rehabilitation.

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.

40.5 Deck Overlays

As a bridge deck ages, preservation and rehabilitation techniques are necessary to maximize the life of the deck and ensure a level of acceptable serviceability. Overlays can be a useful tool to extend the service life of structures. This section discusses several overlay methods, considerations, and guidelines for deck overlays. The provided information is intended for deck-girder structures and may be applicable for slab structures. Slab structures may have different condition triggers and may warrant additional considerations.

The following criteria should be met when determining if an overlay should be used:

- The structure is capable of carrying the overlay dead load
- The deck and superstructure are structurally sound
- The desired service life can be achieved with the considered overlay and existing structure
- The selected option is cost effective based on the anticipated structure life and funding constraints

Decks deteriorate at different rates depending on many factors, including deck materials, material quality, construction quality, structure geometry, exposure to deicing agents, and traffic demands. Additionally, there is a wide variance in the amount of structure preservation techniques utilized by different regions. While the deck age can be a useful parameter, it should not be the primary consideration for determining the eligibility of overlays. Recommended preservation techniques should rely heavily on quality inspection data to determine the appropriate course of action. For more information related to preservation techniques and practices, refer to the Bridge Preservation Policy Guide.

Overlays can be an effective tool to maximize the life of the deck. Figure 40.5-1 illustrates a possible preservation scenario using deck deterioration curves showing approximate deck NBI ratings at which the overlays would occur, and the benefit of performing these overlays. This scenario assumes that the underside of deck deterioration is significantly reduced due to the preservation techniques performed on the top side of the deck.

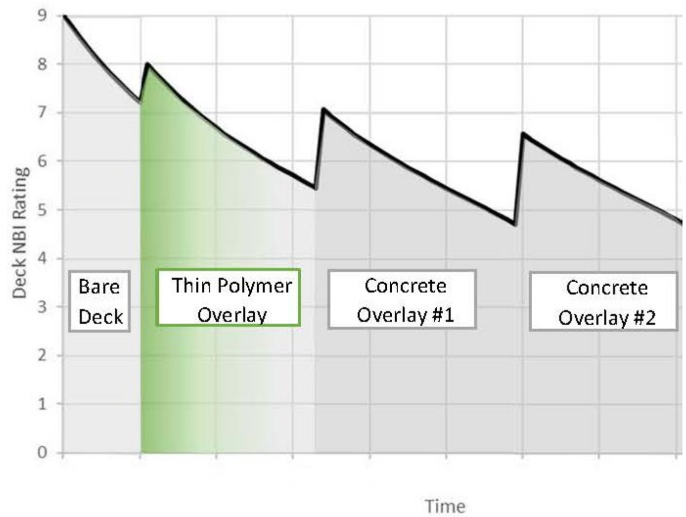

Figure 40.5-1
Deck Deterioration Curve

40.5.1 Overlay Methods

There are several commonly used overlay methods for the preservation and rehabilitation of decks. Generally, thin polymers overlays are recommended as preventative maintenance for decks with a minimal amount of deck distress. Ideally, thin polymer overlays are applied within the first couple of years to limit chloride infiltration. For decks with distress, the existing deck is typically milled and repaired with a low-slump concrete overlay as part of a more extensive bridge rehabilitation effort. For decks nearing replacement, asphaltic overlays maybe a cost effective option to improve ride quality. Refer to the following sections and Table 40.5-1 and Table 40.5-2 for a list of common overlay methods and additional information.

40.5.1.1 Thin Polymer Overlay

A thin polymer overlay (TPO) is expected to extend the service life of a bridge deck for 7 to 15 years. This overlay adds minimal dead load to the existing structure while providing an impermeable surface to prevent chlorides from infiltrating the deck. It can also be used to improve or restore friction on bridge decks.

In general, thin polymer overlays are defined as 1-inch thick or less overlays consisting of a polymer binder with aggregates and can be placed either as a multi-layer, slurry, or premixed system. Typical polymer binders are either epoxy, polyester, or methacrylate based. For WisDOT applications, TPO's consist of a two-layer, two-component epoxy polymer in conjunction with natural or synthetic aggregates for a 1/4-inch minimum total thickness. For dead load purposes, use 5 psf for thin polymer overlays. Refer to the approved products list for a list of pre-qualified polymer liquid binders.

Thin polymer overlays can be placed on new concrete once it has been fully cured and dried to an acceptable moisture content, which may be as soon as 21 days. However, cracks will develop in the concrete deck throughout the first couple of years in response to vehicular and environmental loads. As a result, the preferred time to place the overlay is after initial concrete cracking, which should occur within the first two years of a new deck. Placement after this time allows the overlay to seal existing cracks and may reduce reflective cracking in the overlay. It should be noted that this application window is not ideal for projects, since it will usually require an additional contract for the overlay application. As a result, it is recommended that decks be sealed for the first several years and then receive a thin polymer overlay.

Sufficient bond strength is critical in maximizing the overlay's service life. The bond strength can be reduced by poor surface preparations, traffic conditions, moisture, and distressed concrete. As a result, TPO's should be used based on the following restrictions:

- Recommended on decks with a NBI rating greater than 7 to help mitigate chloride infiltration. The deck should be in good condition with wearing surface distressed areas not exceeding 2% of the total deck area.

- Not recommended on decks that have been exposed to chlorides for more than 10 years old or with a NBI rating less than 7. These restrictions assume that significant chloride infiltration has already occurred. When a robust deck washing and sealing program has been used, TPO's may be placed on decks 10-15 years old with above average deck condition. Roadway traffic volume should also be a consideration for determining when to apply a TPO. As roadway volumes increase, it is assumed that chloride infiltration occurs significantly faster due to the increased application of deicing salts.

- TPO's should not be placed on concrete decks or Portland cement concrete patches less than 28 days, unless approved otherwise. Patch and crack repairs shall be compatible with the overlay material.

- Use of TPO's on the concrete approaches should be avoided. Slab-on-grade conditions may cause the overlay to fail prematurely due to moisture issues.

- Not recommended on decks with widespread cracking, large cracks (>0.04 in), or active cracks (e.g. longitudinal reflective cracks between PS box girders). These cracks are likely to reflect through the overlay, even when fully repaired.

- Decks with an existing TPO may be considered for a TPO re-application provided that the previously discussed restrictions can be assumed to be satisfied. Generally, this assumes the existing overlay performed well over its expected service life and the effective deck exposure did not exceed 15 years, such that significant chloride infiltration has not occurred. If signification chloride infiltration is expected, a re-application would not be recommended.

Thin polymer overlays may be considered where friction needs to be restored or improved. In most cases, the two-layer polymer overlay system should be used as it will improve surface friction and protect the deck against future chloride infiltration. For situations requiring a high skid resistance, calcined bauxite or other alternative aggerates may be considered in lieu of natural or synthetic aggregates.

40.5.1.2 Low Slump Concrete Overlay

A low slump concrete overlay, also referred to as a concrete overlay, is expected to extend the service life of a bridge deck for 15 to 20 years. This system is comprised of low slump Grade E concrete and has a 1-1/2 inch minimum thickness. The overlay thickness can accommodate profile and cross-slope differences, but typically does not exceed 4-1/2 inches. Thicker overlays become increasingly unpractical due to load and cost implications.

Low slump Grade E concrete requires close adherence to the specification, including equipment, consolidation, and curing requirements. A properly cured concrete overlay will help limit cracks, but inevitably the concrete overlay will crack. After the concrete overlay has been placed, it is beneficial to seal cracks in the overlay to minimize deterioration of the underlying deck. The overlay may require crack sealing the following year and periodically thereafter.

On delaminated but structurally sound decks, a rehabilitation concrete overlay is often the only alternative to deck replacement. Typically, prior to placing the concrete overlay a minimum of 1” of existing deck surface is removed along with any unsound material and asphaltic patches.

> Rehabilitation concrete overlays are performed when significant distress of the wearing surface has occurred. If more than 25% of the wearing surface is distressed, an in-depth cost analysis should be performed to determine if a concrete overlay is cost effective verses a deck replacement.
>
> The quantity of distress on the underside of deck or slab should be negligible, less than 5%, indicating that the bottom mat of reinforcement steel is not significantly deteriorated. If significant quantities of distress are present under the deck, a deck replacement may be required in the future; an overlay at this time might not achieve full service life, but may be placed to provide a good riding surface until replacement.

If the structure has an existing overlay, the overlay condition should be evaluated in addition to the other previously discussed considerations. If the concrete deck remains structurally sound, it may be practical to remove an existing overlay and place a new overlay before replacing the entire deck. Prior to placing the concrete overlay, the existing overlay should be removed to at least the original deck surface. Additional surface milling may not be practical if the previous overlay included a milling operation.

40.5.1.3 Polyester Polymer Concrete Overlay

A polyester polymer concrete (PPC) is expected to extend the service life of a bridge deck for 20 to 30 years. This system is a mixture of aggregate, polyester polymer resin, and initiator; which can be placed as a deck overlay using conventional concrete mixing and placement equipment, albeit most likely dedicated to PPC usage. The main advantages of a PPC overlay is that it is impermeable and causes minimal traffic disruptions due to its quick cure time. High costs and lack of performance data are the main disadvantages.

Prior to the placement of the PPC overlay, a high molecular weight methacrylate (HMWM) binder is placed on the prepared deck. This bonds the overlay to the deck, and it also serves to seal existing cracks in the deck. When the existing concrete is in good condition, PPC is effective at mitigating chloride penetration due to its impermeability.

The total thickness of a PPC overlay is typically 3/4” to 1”. While thicker overlays are possible, they are usually cost prohibitive. PPC can be placed at 3/4” thick as opposed to a typical 1 1/2” thick concrete overlay. This may help in situations where bridge ratings and/or profile adjustments are of concern.

Since most applications recommend a 1-inch or less overlay, PPC overlays are considered a thin polymer overlay and have similar requirements and restrictions. PPC overlays should be limited to decks in good condition that require shorter traffic disruptions for sites with high traffic volumes and lane closure restrictions. PPC is a durable product and has a relatively fast curing time (2 to 4 hours), but also has a higher cost as compared to a concrete overlay. PPC overlays should be used based on the following restrictions:

- Deck wearing surface distress should not exceed 5% of the total deck area.
- Decks should have a NBI rating of 7 or greater and be less than 15 years old. Older decks may be considered when the existing deck has been protected by a thin polymer overlay or when chloride testing indicates acceptable chloride levels at the reinforcement. Chloride contents at the reinforcement should not exceed 5 lbs/CY for decks with epoxy coated reinforcement. PPC overlays are not recommended on decks with uncoated top mat reinforcement. Decks exposed to chlorides, exceeding 10 years, should consider a ¾-inch minimum scarification to remove chlorides.
- PPC overlays should not be placed on concrete decks or Portland cement concrete patches less than 28 days, unless approved otherwise. Patch and crack repairs shall be compatible with the overlay material.
- PPC shall not be used for structural repairs due to costs and performance concerns.

Note: PPC overlays are expensive and new to WisDOT. As a result, use of PPC overlays should be limited to preservation projects that meet the requirements outlined in Figure 40.5-2 **or as approved by the Bureau of Structures.**

40.5.1.4 Polymer Modified Asphaltic Overlay

A polymer modified asphaltic (PMA) overlay is expected to extend the service life of a bridge deck for 10 to 15 years. This system is a mixture of aggregate, asphalt content, and a thermoplastic polymer modifier additive, which can easily be placed as a deck overlay using conventional asphalt paving equipment. The thickness of the overlay is 2-inches minimum and can accommodate profile and cross-slope differences.

The added polymer allows for the overlay to resist water and chloride infiltration. Proper mix control and placement procedures are critical in achieving this protection. Core tests have shown the permeability of this product is dependent on the aggregate. As a result, limestone aggregates should not be used.

PMA overlays can be used on more flexible structures (e.g. timber decks or timber slabs) and to minimize traffic disruptions.

Designers should contact the region to determine if a PMA overlay is a viable solution for the project. In some areas, product availability or maintaining an acceptable temperature may be problematic.

Note: PMA overlays are expensive, have a limited service life relative other overlay types, and product availability may be problematic. As a result, PMA overlays usage should be limited.

40.5.1.5 Asphaltic Overlay

An asphaltic overlay, without a waterproofing membrane, is expected to extend the service life of a bridge deck for 3 to 7 years. This system may be a viable treatment if the deck or bridge is programmed for replacement within 4 years on lightly traveled roadways and is able to provide a smooth riding surface. Without a waterproofing material, the overlay may trap moisture at the existing deck surface, which may accelerate deck deterioration.

These overlays must be watched closely for distress as the existing deck surface problems are concealed. This system is typically an asphaltic pavement with a mixture of aggregates and asphaltic materials, which can easily be placed as a deck overlay using conventional asphaltic mixing and placement equipment. The thickness of the overlay is 2-inches minimum and can accommodate profile and cross-slope differences.

Note: Asphaltic overlays, without a waterproofing membrane, are not eligible for federal funds.

40.5.1.6 Asphaltic Overlay with Waterproofing Membrane

An asphaltic overlay, with a waterproofing membrane, is currently being used on a very limited basis. This system is expected to extend the service life of a bridge deck for 5 to 15 years. Experience indicates that waterproofing membranes decrease the rate of deck deterioration by preventing or slowing the migration of water and chloride ions into the concrete.

In the 1990's, waterproofing membranes were actively used with asphaltic overlays for protecting existing decks, but were phased out by 2009 when they were restricted due to performance concerns and the inability to inspect the deck. As a result, low slump concrete or PMA overlays are currently recommended when deck or bridge replacements are programed beyond 4 years, unless approved otherwise.

Note: Asphaltic overlays, with a waterproofing membrane, requires prior-approval by the Bureau of Structures. This system is currently under review for possible improvements.

40.13 Survey Report and Miscellaneous Items

Prior to scheduling bridge plans preparation, a Rehabilitation Structure Survey Report Form is to be completed for the proposed work with field and historical information. The Rehabilitation Structure Survey Report provides the Bridge Engineer pertinent information such as location, recommended work to be performed, and field information needed to accomplish the rehabilitation plans. A brief history of bridge construction date, type of structure, repair description and dates are also useful in making decisions as to the most cost effective rehabilitation. Along with this information, it is necessary to know the extent of deck delamination and current deck, super and substructure condition ratings. A thorough report recommending structural repairs and rehabilitation with corresponding notes is very useful for the designer including information from special inspections.

Because new work is tied to existing work, it is very important that accurate elevations be used on structure rehabilitation plans involving new decks and widenings. Survey information of existing beam seats is essential. Do not rely on original plans or a plan note indicating that it is the responsibility of the contractor to verify elevations.

Experience indicates that new decks open to traffic and subjected to application of de-icing salts within the first year show signs of early deck deterioration. Therefore, a protective surface treatment is applied to all new bridge deck concrete as given in the standard specifications.

For existing sloped faced parapets on decks under rehabilitation, the metal railings may be removed. On deck replacements and widenings, all existing railings are replaced. Refer to Bridge Manual Chapter 30-Railings for recommended railings.

On rehabilitation plans requiring removal of the existing bridge name plates; provide details on the plans for new replacement name plates if the existing name plate cannot be reused. The existing bridge number shall be used on the name plates for bridge rehabilitation projects.

If floor drain removal is recommended, review these recommendations for conformance to current design standards and remove any floor drains not required. Review each structure site for length of structure, grade, and water erosion at the abutments.

Bridge rehabilitation plans for steel structures are to provide existing flange and web sizes to facilitate selecting the proper length bolts for connections. If shear connectors are on the existing top flanges, additional shear connectors are not to be added as welding to the top flange may be detrimental. Do not specify any aluminized paint that will come in contact with fresh concrete. The aluminum reacts severely with the fresh concrete producing concrete volcanoes.

Recommended paint maintenance is determined with assistance from the Wisconsin Structures Asset Management System (WiSAMS), which utilizes information provided by the routine bridge inspections.

Structure plans (using a sheet border with a #8 tab) are required for all structure rehabilitation projects. This includes work such as superstructure painting projects and all overlay projects, including polymer overlay projects. See Chapter 6-Plan Preparation guidance for plan minimum requirements.

Existing steel expansion devices shall be modified or replaced with watertight expansion devices as shown in Bridge Manual Chapter 28-Expansion Devices. If the hinge is repaired, consideration should be given to replacing the pin plates with the pins. Replace all pins with stainless steel pins conforming to ASTM 276, Type S20161 or equal. On unpainted steel bridges, the end 6' or girder depth, whichever is greater, of the steel members adjacent to an expansion joint and/or hinge are required to have two shop coats of paint. The second coat is to be a brown color similar to rusted steel. Exterior girder faces are not painted for aesthetic reasons, but paint the hanger on the side next to the web.

A bridge on a steep vertical grade may slide downhill closing any expansion joint on this end and moving the girders off center from the bearings. One possible corrective action is to block the ends of the girder during the winter when the girders have shortened due to cold temperatures. Continued blocking over a few seasons should return the girders to the correct position. The expansion device at the upslope end may have to be increased in size.

Raised pavement markers require embedment in the travel way surface. As a general rule, they are not to be installed on bridge decks due to future leakage problems if the marker comes out. They shall not be installed thru the membranes on asphalt overlays and not at all on asphaltic decks where the waterproofing is integral with the surface.

ii. Use steel plates and post-tensioning bars to place compression loads on both ends of cap. Cover exposed bars with concrete.

iii. Pour wing extension under pier caps beginning at base to take all loads in compression. This would alter pier shape.

d. Consider sloping top of pier to get better drainage.

e. Consider placing coating on pier top to resist water intrusion.

If the shaft is severely spalled, it will require a layer of concrete to be placed around it. Otherwise, patching is adequate. To add a layer of concrete:

1. Place anchor bars in existing solid concrete. Use expansion anchors or epoxy anchors as available.

2. Place wire mesh around shaft.

3. Place forms and pour concrete. 6” is minimum thickness.

40.15.2 Bearings

Bearings being replaced should follow the Chapter 27 Standard Details, as well as the Chapter 40 Standard for Expansion Bearing Replacement Details. Replace lubricated bronze bearings with either laminated elastomeric bearings (preferred, if feasible) or Stainless Steel TFE bearings. If only outside bearings are replaced, the difference in friction/resistance values between adjacent girders can be ignored. In addition to the bid item for the new bearing, the STSP Removing Bearings is required.

For bearings requiring maintenance, consider the SPV Cleaning and Painting Bearings. Special Provisions Bearing Maintenance and Bearing Repair may also be worthy of consideration.

40.16 Concrete Anchors for Rehabilitation

Concrete anchors are used to connect concrete elements with other structural or non-structural elements and can either be cast into concrete (cast-in-place anchors) or installed after concrete has hardened (post-installed anchors). This section discusses post installed anchors used on bridge rehabilitation projects. Note: this section is also applicable for several cases where post installed anchors may be allowed in new construction.

This section includes guidance based on the ACI 318-14 manual, hereafter referred to as ACI. (AASHTO currently does not have guidance for anchors.)

40.16.1 Concrete Anchor Type and Usage

Concrete anchors installed in hardened concrete, post-installed anchors, typically fall into two main groups – adhesive anchors and mechanical anchors. For mechanical anchors, subgroups include undercut anchors, expansion (torque-controlled or displacement controlled) anchors, and screw anchors.

Mechanical anchors are seldom used for bridge rehabilitations and current usage has been restricted due to the following concerns: anchor installation (hitting rebar, abandoning holes, and testing), the number of different anchor types, design requirements that are more restrictive than adhesive anchors, the ability to remove and reuse railings/fences, and the collection of salt water within the hole. Note: mechanical anchors may be considered when it has been determined cast-in-place anchors or through bolts are cost prohibitive, adhesive anchors are not recommended, and the above concerns for mechanical anchors have been addressed. See post-installed anchor usage restrictions for additional information.

An Approved Products List addresses some of the concerns for creep, shrinkage, and deterioration under load and freeze-thaw cycles for adhesives anchors. Bridge rehabilitations projects typically use adhesive anchors for abutment and pier widenings. Other bridge rehabilitation applications may also warrant the use of adhesive anchors when required to anchor into existing concrete. Refer to the Standards for several examples of anchoring into existing concrete.

In limited cases, post installed concrete anchors may be allowed for new construction. One application is the allowance for the contractor to use adhesive anchors in lieu of cast-in-place concrete anchors for attaching pedestrian railings/fencing. Refer to Chapter 30 Standards for pedestrian railings/fencing connections.

The following is a list of current usage restrictions for post installed anchors:

<u>Usage Restrictions</u>:

- Pier cap extensions for multi-columned piers require additional column(s) to be utilized. See Chapter 13 – Piers for structural modeling concepts regarding multi-columned piers.

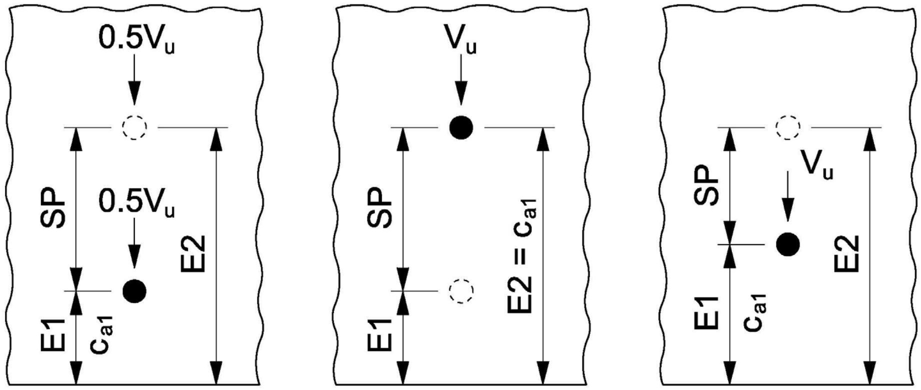

Figure 40.16-4
Concrete Anchor Shear Force Cases

The factored shear force on each anchor, V_u, must be less than or equal to the factored shear resistance, V_r. For mechanical and adhesive anchors:

$$V_r = \phi_{vs} V_{sa} \le \phi_{vc} V_{cb} \le \phi_{vp} V_{cp}$$

In which:

ϕ_{vs} = Strength reduction factor for anchors in concrete, **ACI [17.3.3]**
= 0.60 for brittle steel as defined in 40.16.1.1
= 0.65 for ductile steel as defined in 40.16.1.1

V_{sa} = Nominal steel strength of anchor in shear, **ACI [17.5.1.2]**
= $0.6 A_{se,V} f_{uta}$

$A_{se,V}$ = Effective cross-sectional area of anchor in shear (in^2)

ϕ_{vc} = Strength reduction factor for anchors in concrete, **ACI [17.3.3]**
= 0.70 for anchors without supplementary reinforcement per 40.16.2
= 0.75 for anchors with supplementary reinforcement per 40.16.2

V_{cb} = Nominal concrete breakout strength in shear, **ACI [17.5.2.1]**
= $\dfrac{A_{Vc}}{4.5(c_{a1})^2} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{p,V} V_b$

A_{Vc} = Projected area of the concrete failure surface on the side of the concrete member at its edge for a single anchor, see Figure 40.16-3
= $H(S_1 + S_2)$

c_{a1} = Distance from the center of anchor shaft to the edge of concrete in the direction of the applied shear, see Figure 40.16-3 and Figure 40.16-4 (in)

$\psi_{ed,V}$ = Modification factor for shear strength of anchors based on proximity to edges of concrete member, **ACI [17.5.2.6]**
= 1.0 if $c_{a2} \geq 1.5c_{a1}$ (perpendicular shear)
= $0.7 + 0.3\dfrac{c_{a2}}{1.5c_{a1}}$ if $c_{a2} < 1.5c_{a1}$ (perpendicular shear)
= 1.0 (parallel shear)

c_{a2} = Distance from the center of anchor shaft to the edge of concrete in the direction perpendicular to c_{a1}, see Figure 40.16-3 (in)

$\psi_{c,V}$ = Modification factor for shear strength of anchors based on the presence or absence of cracks in concrete and the presence or absence of supplementary reinforcement, **ACI [17.5.2.7]**
= 1.4 for anchors located in a region of a concrete member where analysis indicates no cracking at service load levels
= 1.0 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels without supplementary reinforcement per 40.16.2 or with edge reinforcement smaller than a No. 4 bar
= 1.2 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels with reinforcement of a No. 4 bar or greater between the anchor and the edge
= 1.4 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels with reinforcement of a No. 4 bar or greater between the anchor and the edge, and with the reinforcement enclosed within stirrups spaced at no more than 4 inches

$\psi_{h,V}$ = Modification factor for shear strength of anchors located in concrete members with $h_a < 1.5_{ca1}$, **ACI [17.5.2.8]**
= $\sqrt{\dfrac{1.5c_{a1}}{h_a}} \geq 1.0$

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41.1 Introduction

The Federal Highway Administration (FHWA) Moving Ahead for Progress in the 21st Century (MAP-21) legislation contains the following definition for asset management:

Asset management is a strategic and systematic process of operating, maintaining, and improving physical assets, with a focus on both engineering and economic analysis based upon quality information, to identify a structured sequence of maintenance, preservation, repair, rehabilitation, and replacement actions that will achieve and sustain a desired state of good repair over the life-cycle of the assets at minimum practicable cost.

The Wisconsin Department of Transportation (WisDOT) has developed and is implementing a structures asset management program that meets FHWA's definition. At a basic level, WisDOT structures asset management is practiced as shown in Figure 41.1-1.

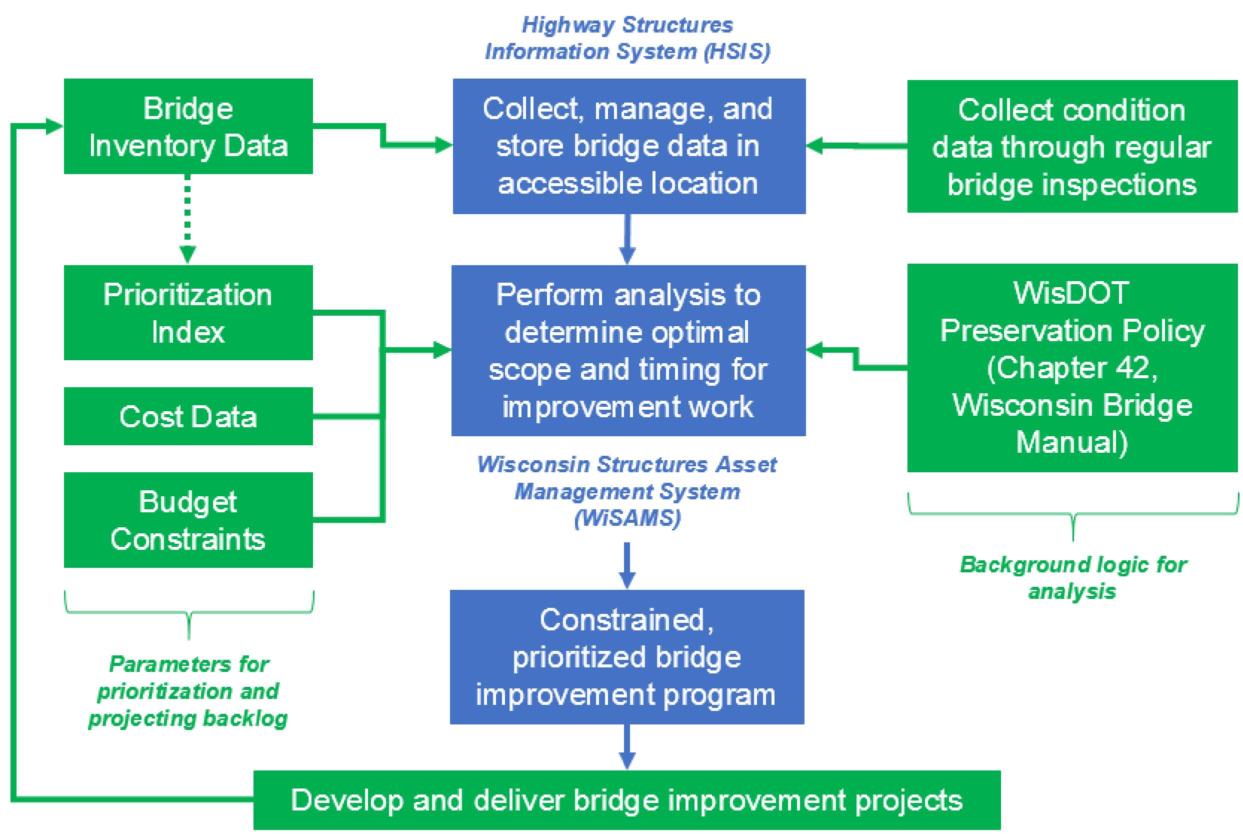

Figure 41.1-1
WisDOT Structures Asset Management

This chapter provides an outline of the WisDOT structures asset management process, including roles and responsibilities and policy items to be considered during the selection of structures improvement projects.

41.1.1 Definitions

Primary structure work concept: The primary work being performed on a given structure.*** Primary structure work concepts are currently defined as any of the following:

- New structure (new alignment, etc.)
- Structure replacement
- Superstructure replacement
- Deck replacement
- Structure widening
- Overlay (any type)
- Painting (full)

****Note that a given bridge may not have a primary structure work concept, but only a secondary structure work concept. One example of this would be a bridge requiring a joint replacement and concrete surface repairs to the substructure elements, with no "major rehabilitation" (deck replacement, overlay, etc.) required.*

Secondary structure work concept: Work performed on a given structure that is not designated as primary. Examples include, but are not limited to:

- Joint replacement or rehabilitation
- Bearing replacement or rehabilitation
- Parapet or railing repairs

Structures improvement project: An improvement project funded through WisDOT's let program that includes primary or secondary work to one or more structures. Other work, such as pavement or safety, may or may not be included.

41.1.2 WisDOT Asset Management Themes

The WisDOT Bureau of Structures (BOS) work in asset management is enveloped by the broader asset management philosophies of the Department. Current emphasis areas include:

- Ensuring that all in-service structures are safe for the travelling public. This is the top priority.
- Making decisions that are supported by data and policy and applied consistently across the state.
- Seeking to extend the usable life of a structure (versus replacement) when feasible, practical, and cost-effective by using identified preservation techniques.
- Considering the whole-life-cycle costs when selecting treatments.
- When structure replacement is unavoidable, replacing the existing structure following the current Department replace-in-kind policy and design standards.

41.2 Identifying Theme-Compliant Structure Work

Themes for structures asset management are noted in 41.1.2 and represented in the policy documented in Chapter 42 – Bridge Preservation. This section details how BOS arrives at recommended bridge improvement work actions that comply with Department asset management principles.

41.2.1 Wisconsin Structures Asset Management System (WiSAMS)

To accurately and consistently apply structures asset management strategies, BOS developed a software application; the Wisconsin Structures Asset Management System, or WiSAMS. WiSAMS was developed and is maintained within BOS. Its core function is to produce recommendations for structures improvement work using a consistent, objective, data-driven, logic-based process.

The success of WiSAMS is heavily dependent on the quality of the data it uses. The primary data consists of the following:

- Inventory data: Information that defines the bridge type, location, use, and history. This includes items such as number of spans, superstructure type, construction history, Average Daily Traffic (ADT).
- Condition data: Collected and recorded during bridge inspections, condition data reflects the current state of deterioration of the bridge. WiSAMS currently uses both NBI and AASHTO element condition data.

The background logic for WiSAMS consists of a series of “if-then” statements and a corresponding structure improvement work action. These if-then statements are referred to as “rules”. The WiSAMS rules are based on the asset management and bridge preservation policy documented in Chapter 42 – Bridge Preservation. WiSAMS evaluates each rule in sequence. When a rule evaluates as “true”, the corresponding work action for that rule is logged as the recommended structure improvement work. If no rules evaluate as “true”, then the WiSAMS recommendation is “no action.” For illustration purposes, a very simple WiSAMS rule is shown below.

- If all the following criteria are met…
    - The current NBI rating for substructure is less than or equal to 3, and
    - The structure is scour critical;
- …then the recommended work action is “REPLACE STRUCTURE.”

WiSAMS performs the analysis described above for the current year based on the most recent condition data (inspection report). To project future needs, WiSAMS uses deterioration curves to model the future condition of the structure. For each future year, WiSAMS again performs the rule analysis using the projected future condition data and provides recommendations for structure work concepts in these future years.

41.2.2 Eligibility

WiSAMS is the primary asset management tool for BOS. It is a tool that aims to meet WisDOT's need for data-driven, consistent, cost-effective structures work recommendations. The general accuracy of WiSAMS recommendations is heavily dependent on the available condition data and the ability to accurately project future deterioration. Final recommendations on structures improvement actions are subject to a manual review by BOS asset management engineers, as necessary. This combination of WiSAMS output and BOS review results in a recommendation for improvement work scope and timing.

A proposed structures work concept that matches this BOS recommended scope and is within an acceptable range for timing is considered an eligible structures work concept. Effort should be made to program structures improvement work to match the BOS-recommended work concept and optimal year to the extent possible.

WisDOT policy item:

BOS currently considers +/- 3 years as acceptable deviation from the BOS-recommended year for programming structures work concepts.

41.3 Structures Programming Process (State-System)

The process for developing projects with structures improvement work is shown in Figure 41.3-1 below. The process is primarily a collaboration between BOS and Regional personnel. The Division of Transportation Investment Management (DTIM) has a role in funding, which varies based on the funding source. Roles and responsibilities are discussed further in 41.5.

It is important to note that structures work concepts can be included in a let improvement project via several different methods. They include:

- Stand-alone structures improvement project: A let improvement project developed based on structure needs and including only structures improvement work.

- Combined improvement project: A let improvement project developed based in part on structure needs, but also including other improvement needs, such as pavement, safety, etc.

- Improvement project with only secondary structures improvement work: A let improvement project developed based on “other” needs (pavement, safety, etc.), but includes structures within the project limits. Structures within the project limits should always be evaluated for secondary work concepts.

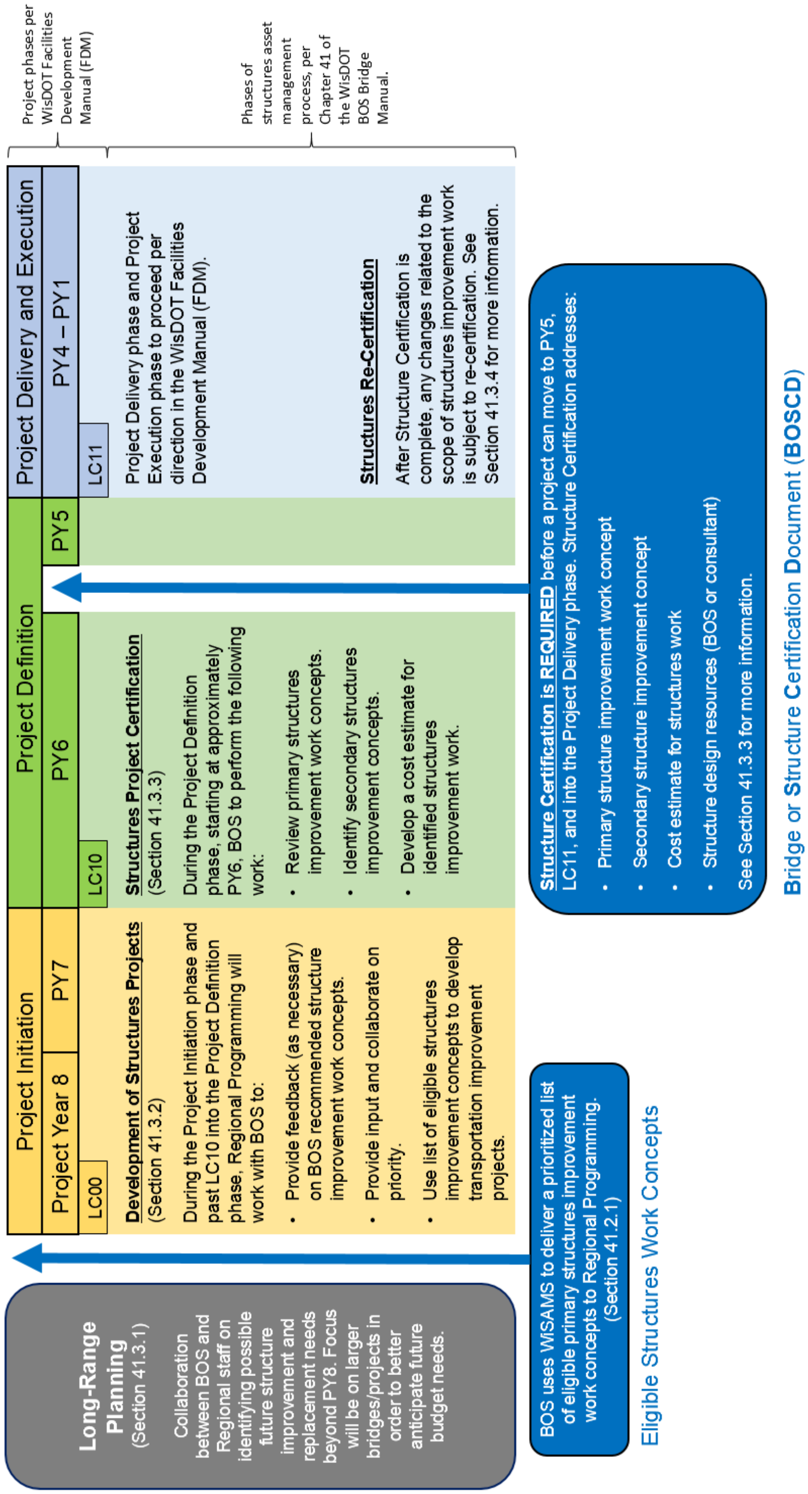

Figure 41.3-1
WisDOT Structures Asset Management – Structures Project Development

41.3.1 Long-Range Planning

Long-range planning refers to planning work done for projects with a target year beyond Program Year 8. Long-range planning serves several purposes, including examples such as:

- Coordinates improvement projects that are close in proximity to each other to minimize inconvenience for the travelling public.
- Project future improvement needs to large and/or complex bridges. Work of this nature may have a large impact in terms of budget and required design time.
- Provide information on future structure needs to coordinate with the long-term Division and Department vision for targeted corridors or areas.
- Provide a network-wide projection of future needs to be used when considering future transportation funding levels.

Projection of long-range structure improvement needs are based on WiSAMS output. BOS and Region collaboration on long-range planning occurs on an as-needed basis.

41.3.2 Development of Projects with Structures Work (PY8-PY7, Life Cycle 00-10)

The process of developing structure projects initiates with the BOS. Using WiSAMS (described above in 41.2.1) and review by BOS asset management engineers, BOS develops a list of eligible structures work concepts for the target year – Program Year 8 (PY8). The work is based on established BOS and Department policies for structures asset management, as described in this chapter and Chapter 42 – Bridge Preservation. The list of eligible structures work concepts is also prioritized. BOS will deliver these concepts to the Regions twice annually; in February and August.

41.3.2.1 Work Concept Review

At this stage in the process, Regional personnel has the opportunity to review the draft structures improvement program. The focus of this review is the primary work concepts, though some secondary work concepts may also be identified at this stage. BOS recommendations for structures work concepts are based on WiSAMS, supplemented as necessary by BOS asset management engineers.

Regional review should be focused on identifying perceived gross mismatches in scope and/or timing, and highlighting structure work concepts not identified by WiSAMS or BOS. Final decisions on scope and timing must be based on data and/or documentation. A majority of the time, this will be WiSAMS, but it can also be supplemented by other information, such as construction history, supplemental inspection data, IR data, or any other information pertinent to the programming decision. BOS will collaborate with Regional personnel to review and discuss any additional information that is brought forth. Final scope and timing decisions for structures work will be made by BOS, with strong consideration of Regional input.

41.3.2.2 Priority Review

BOS provides a prioritized list of eligible structures work. Priority is determined using a priority index (PI); an algorithm developed by BOS. The algorithm considers data such as ADT, functional class, etc. This is intended to assist the regions as they program projects.

The Region may see fit to adjust the prioritized list based on regional system and operational factors.

41.3.2.3 Creating Improvement Projects with Structures Work Concepts

The next step in the programming process is for Regional Programming to develop structures improvement projects based on the list of individual structures work concepts. Projects may combine structures work as appropriate, but also consider pavement needs, safety needs, operational needs, etc.

There may be non-structural rationale for deviations from BOS-recommended scope and/or timing. Common reasons include, but are not limited to:

- Coordination with other improvement work (pavements, safety, operations, etc.)
- Traffic control costs
- User delay

If reasons such as those noted above are used to justify deviations from BOS-recommended scope and/or timing, a cost-benefit analysis should be performed to support the decision. More information on cost-benefit analysis and structures programming policy can be found in 41.6.6.

During this phase as projects are developed and up until the Structures Project Certification Phase (See 41.3.3), BOS asset management engineers will evaluate proposed projects on a regular basis to ensure that programmed structures work is eligible in terms of both scope and timing. Projects that contain only eligible structures work concepts or have appropriate justification for any deviations are considered *pre-certified*.

<u>Only eligible projects or projects with appropriate justification will be considered for funding.</u>

41.3.3 Structures Project Certification Phase (PY6-PY5, Life Cycle 10/11)

Structures project certification refers to the work required to produce the Bridge or Structure Certification Document (BOSCD). The components of the BOSCD are outlined in 41.3.3.6 below.

> **<u>WisDOT policy item:</u>**
>
> Any improvement project with structures work (primary or secondary work concepts) requires certification.

41.3.3.1 BOS Structures Certification Liaison

BOS will designate a certification liaison for every structures improvement project, regardless of whether the project is designed by BOS or a consultant. The certification liaison will perform all of the work necessary for structures certification. A certification liaison will remain with each structures project (BOS-designed or consultant-designed) through the letting of that project, though the actual person assigned to a project may change over the lifecycle of that project.

41.3.3.2 Review of Primary Structures Work Concepts

Structures certification serves as the final review and approval for the scope and timing of the primary structures work concept. Regional planning engineers should only be selecting eligible structures work (scope and timing) for inclusion in transportation improvement projects. Additionally, BOS asset management engineers will evaluate projects on a regular basis (see 41.3.2) to ensure eligibility. With this process in place, the certification liaison will collaborate with BOS asset management engineers and Regional programming engineers (as necessary) to confirm scope and timing for primary structures work concepts.

41.3.3.3 Development of Secondary Structures Work Concepts

A key portion of the BOSCD is the early identification of secondary structures improvement work. Some examples of secondary work include, but are not limited to:

- Bearing rehabilitation or replacement
- Parapet or railing repairs
- Backwall or wingwall repairs
- Identification of specific substructure repairs
- Scour mitigation

Some items such as those above may have already been identified during the scoping of the primary structures work concepts. The certification liaison will review the existing inspection reports on file and consult the appropriate Regional structures maintenance engineer(s) to identify any and all eligible secondary structures work concepts.

41.3.3.4 Development of the Structures Cost Estimate

A high-level cost estimate will have been developed as a part of the primary structures work concept. This estimate is for structures work only; costs for traffic control and mobilization are not included. The certification liaison will refine that estimate, taking into account the identified secondary structures improvement work. This estimate is not intended to be a final structures construction cost estimate, but is a refinement of the unit cost estimate previously developed.

41.3.3.5 Determination of Design Resourcing

BOS will determine design resourcing as a part of the structures certification process. If BOS chooses to decline structures design for a given project, the certification liaison will provide an estimated level of effort (in man-hours) for the structures work. The certification liaison and the BOS Consultant Review Supervisor will coordinate with Regional PDS staff to ensure selection of an appropriate consultant engineer for the project.

41.3.3.6 Bridge or Structure Certification Document (BOSCD)

The BOSCD is under development at the time of publishing this chapter. The certification document will include information on all the items noted above, in addition to other key information identified by Region personnel.

41.3.4 Project Delivery and Execution Phase (PY4-Construction, Life Cycle 12+)

41.3.4.1 Structures Re-Certification

Any and all changes related to structures improvement work affecting items approved as part of the structures project certification shall be reviewed and approved by the certification liaison. This includes, but is not limited to, any of the following items:

- Scope (primary or secondary)
- Structures construction cost estimate
- PS&E or let date
- Advanceable date
- Structures design resourcing

The certification liaison for the project should be notified of any changes as soon as reasonably possible to approve/re-certify the project in a timely manner and not delay project schedule.

41.4 Structures Programming Process (Local System)

Local structure improvement work is funded through a different mechanism than state structure improvement work. Based on Wisconsin State Statutes, local structure eligibility is largely based on the sufficiency rating, a measure detailed in the Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges, published by FHWA. BOS provides a prioritized eligibility list of both primary and secondary work concepts for the Division of Transportation Investment Management (DTIM), which is then posted publicly for the local owners. Local owners use this eligibility list to select projects for submission to the local program, and DTIM programs structure work on a biannual basis. At this point, BOS has not pursued additional involvement in structures asset management for the local inventory.

More information about the Local Bridge Program can be found at the following link:

https://wisconsindot.gov/Pages/doing-bus/local-gov/astnce-pgms/highway/localbridge.aspx

41.5 Structures Asset Management Roles and Responsibilities

41.5.1 Bureau of Structures (BOS)

BOS has three sections, each of which contribute to the structures asset management process, either directly or indirectly.

BOS Design Section

- Resource the design (including hydraulic considerations) of structures improvement projects or providing oversight for consultant-designed projects.
- Provide resources (certification liaison) for the structures project certification (See 41.3.3).

BOS Maintenance Section

- Provide oversight for the WisDOT structures inspection program, working to ensure and improve the quality and accuracy of condition data.

BOS Development Section

- Manage and maintain the Highway Structures Information System (HSIS), an on-line database for collecting structures inventory and condition data.
- Develop, maintain, and refine Chapter 42 – Bridge Preservation. Policy documented in this chapter is the basis for WisDOT structures asset management.
- Develop and maintain WiSAMS, the software application that uses inspection and inventory data to produce recommendations for future structure improvement projects.
- Using WiSAMS (including priority and budget features), develop draft recommendations for the program-level scope of recommended structures work for the 8-year structures improvement program.
- Collaborate with Regional personnel to develop structures projects for the 8-year structures improvement program.
- Review and pre-certify structures projects that are introduced to the 8-year structures improvement program. See 41.3.2.3.
- Develop and maintain a program effectiveness measure to assess progress toward achieving program goals.

41.5.2 WisDOT Regions

WisDOT divides the state into five regions; Northwest, North Central, Northeast, Southeast, and Southwest. See Figure 2.1-3. Each Region has the responsibilities outlined below for the structures in their designated territory.

Regional Planning and Scoping Units

- Review structures work concepts provided by BOS and coordinate with other stakeholders (pavements, operations, safety, etc.) to recommend adjustments as deemed necessary.
- Collaborate with BOS to develop structures improvement projects that incorporate identified structure needs, coordinating as appropriate to address other need areas (pavement, safety, etc.).

- Collaborate with BOS in the structures certification process (See 41.3.3).

Regional Project Development Sections (PDS)

- Participate in the structures certification process, as necessary (See 41.3.3).
- Coordinate with BOS on structures project re-certification, as necessary. (See 41.3.4.1.)
- Guide structures improvement projects from project certification through construction, working to ensure that the project is constructed per plans and specifications.

Regional Structures Maintenance Units

- Provide detailed structures condition data (via inspection reports) that fully and accurately depict the current state of each individual structure.
- Collaborate with BOS certification liaison in the structures certification process, specifically in the scoping of primary and secondary structures work concepts (See 41.3.3.3).
- Perform or coordinate some preventative maintenance work; deck washing, deck sealing, crack sealing, etc. See Chapter 42 – Bridge Preservation for more information.

41.5.3 Division of Transportation Investment Management (DTIM)

DTIM is responsible for the financial component of structures asset management, determining the allocation of funds for structures improvement projects.

Bureau of State Highway Programs (BSHP)

- Collaborate with BOS to assess structures needs as they relate to the allocation of available funds to the various WisDOT funding programs.
- Determine the specific allocation of available funding for each of the WisDOT funding programs.
- Provide direct oversight and prioritization for the state-wide Backbone funding program.
- Provide financial analysis expertise and tools, such as Life Cycle Cost Analysis (LCCA) guidance.

Bureau of Transit, Local Roads, Railroads & Harbors (BTLRRH)

- Provide direct oversight and programming for the Local Bridge program, utilizing the list of eligible structure work concepts provided by BOS.

41.6 Programming Policy for Structures Improvement Projects

Structures improvement needs are identified by BOS as detailed 41.2 above. As Regional personnel work to develop projects to address these structures needs, other factors may contribute to the final project scope and timing. The policy items noted below provide direction on how some of these project factors shall be considered as they relate to the scope of structures improvement work.

41.6.1 Bridge Age

WisDOT policy item:

Bridge age shall not be a primary driver for the initiation of structures improvement work.

For a given bridge, there is correlation between the condition of the bridge and its age. However, condition (not age) shall be the primary driver for structures improvement work. The focus of evaluation should be on how the structure is currently performing, regardless of structure age.

41.6.2 Bridge Ratings

WisDOT policy item:

Unless specifically approved by BOS, inventory rating, operating rating, or the presence of a load posting shall not be the primary driver for the initiation of structures improvement work.

If a structures improvement project has been reviewed and approved by BOS (see 41.3.3), it may be appropriate to include work to improve load ratings or remove a load posting. Consult with the BOS Rating Unit before expanding structures scope to include strengthening.

41.6.3 Vertical Clearance

WisDOT policy item:

Vertical clearance shall not be the primary driver for the initiation of structures improvement work.

Various impact mitigation techniques shall be evaluated for bridges with a history of impacts before scoping an improvement project to include addressing substandard vertical clearance.

If deck replacement, superstructure replacement, or structure replacement are identified as the appropriate treatment and vertical clearance is substandard, the project team should investigate the additional cost of creating more vertical clearance.

Region and BOS concurrence is required to up-scope a project for vertical clearance issues.

41.6.4 Hydraulics

WisDOT policy item:
In the case of structures with flooding history or concerns, improvement work shall not be initiated unless mitigation (detours) are not possible. If mitigation is not possible, consult BOS Hydraulics Unit for direction.

In most cases, traffic can be adequately detoured around flooded structures until such time as waters recede.

41.6.5 Freight Considerations

WisDOT policy item:
Freight needs shall not drive the initiation of a structures improvement project.

As related to structures, freight needs are primarily capacity (load ratings and/or load postings) and clearance (vertical and horizontal).

41.6.6 Cost Benefit Analysis

When considering different options for structures improvement work, a cost-benefit analysis should be performed. The analysis should be performed by Regional programming staff using analysis tools approved by the DTSD Administrator’s Office. Direction on select input data to be used for cost-benefit analysis is detailed below.

41.6.6.1 Treatment Schedule

When performing cost-benefit analysis, the following shall be used as the idealized treatment schedule for a new bridge. **The treatment schedules below are only for use in cost-benefit analysis and are not intended to be used for programming purposes.**

Prestressed Girder Superstructure

Primary Work Concept	Secondary Work Concept	Structure Year
New Construction	---	Year 0
Thin Polymer Overlay	---	Year 0
Thin Polymer Overlay	---	Year 10
Concrete Overlay and New Joints	• Substructure repair • Superstructure repair	Year 35
Deck Replacement	• Substructure repair • Superstructure repair	Year 55
Thin Polymer Overlay	---	Year 55
Thin Polymer Overlay	• Substructure repair • Superstructure repair	Year 65
Concrete Overlay and New Joints	---	Year 90
Bridge Replacement	---	Year 100

Steel Girder Superstructure

Primary Work Concept	Secondary Work Concept	Structure Year
New Construction	---	Year 0
Thin Polymer Overlay	---	Year 0
Thin Polymer Overlay	---	Year 10
Concrete Overlay and New Joints	• Spot/zone painting • Substructure repair • Superstructure repair	Year 35
Deck Replacement	• Complete painting • Substructure repair • Superstructure repair	Year 55
Thin Polymer Overlay	---	Year 55
Thin Polymer Overlay	• Spot/zone painting • Substructure repair • Superstructure repair	Year 65
Concrete Overlay and New Joints	---	Year 90
Bridge Replacement	---	Year 100

Concrete Slab Superstructure

Primary Work Concept	Secondary Work Concept	Structure Year
New Construction	---	Year 0
Thin Polymer Overlay	---	Year 0
Thin Polymer Overlay	---	Year 10
Concrete Overlay and New Joints	• Substructure repair • Superstructure repair	Year 35
Concrete Overlay and New Joints	• Substructure repair • Superstructure repair	Year 55
Bridge Replacement	---	Year 75

For all other superstructure types or in-service structures, consult BOS Bridge Management Unit for direction.

41.6.6.2 Discount Rate

WisDOT policy item:

A discount rate of 5% shall be used for cost-benefit analysis.

This value was determined based on analysis conducted by DTIM and is Department policy.

41.6.7 User Delay

WisDOT policy item:

For the purposes of cost-benefit analysis, user delay shall be addressed per direction in the WisDOT Facilities Development Manual (FDM).

User delay can have a dramatic impact on the results of a cost-benefit analysis and must be considered based on Department policy.

41.7 References

1. *Specification for the National Bridge Inventory Bridge Elements Bridges* by Federal Highway Association, 2014

2. *Manual for Bridge Element Inspection, First Edition* by American Association of State Highway Transportation Officials, 2010

3. *Structure Inspection Manual* by Wisconsin Department of Transportation, 2003.

4. *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* by Federal Highway Association, 1995

5. *Facilities Development Manual (FDM)* by Wisconsin Department of Transportation, 2018

6. *Program Management Manual (PMM)* by Wisconsin Department of Transportation, Division of Transportation Investment Management, 2018

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42.1 Overview

This chapter provides goals, objectives, measures, and strategies for the preservation of bridges. This chapter contains criteria that is used to identify condition based and cyclical preservation, maintenance, and improvement work actions for bridges. Bridge preservation is defined as actions or strategies that prevent, delay, or reduce deterioration of bridges or bridge elements; restore the function of existing bridges; keep bridges in good or fair condition; and extend their service life. Preservation actions may be cyclic or condition-driven. (1)

A successful bridge program will seek a balanced approach to preservation, rehabilitation, and replacement. One measure of success is to maximize the life of structures while minimizing the life cycle cost. Preservation of structures is one of the strategies in maximizing the effectiveness of the overall bridge program by retarding the rate of overall deterioration of the bridges.

Bridges are key components of our highway infrastructure. Wisconsin has over 14,000 bridges, of which about 37% are owned by WisDOT. The average age of these bridges in 2019 is 38 years. The aging infrastructure is expected to deteriorate faster in the coming decades with increased operational demand unless concerted efforts are taken to preserve and extend their life. In addition, the state bridge infrastructure is also likely to see an increased funding competition among various highway assets. As a result, WisDOT must emphasize a concerted effort to preserve and extend the life of bridge infrastructure while minimizing long-term maintenance costs.

This chapter provides WisDOT personnel and partners with a framework for developing preservation programs and projects using a systematic and consistent process that reflects the environment and conditions of bridges and reflects the priorities and strategies of the Department.

A well-defined bridge preservation program will also help WisDOT use federal funding (2) for Preventative Maintenance (PM) activities by using a systematic process of identifying bridge preservation needs and its qualifying parameters as identified in FHWA's Bridge Preservation Guide (1). This chapter will promote timely preservation actions to extend and optimize the life of bridges in the state.

42.2 WisDOT Goals and Strategies for Bridge Preservation

The main goal of a bridge preservation program is to maximize the useful life of bridges in a cost-effective way. To meet this goal, many of the strategies are aimed at applying the appropriate bridge preservation treatments and activities at the proper time resulting in longer service life at an optimal life cycle cost. Federal transportation legislation (MAP-21) promotes the goal of maintaining or preserving infrastructure assets "in a state of good repair". Preservation of assets is one of the tools that will be used to achieve an overall transportation investment strategy.

42.2.1 Goals for WisDOT Bridge Preservation Program

The bridge preservation goals address the priorities of the department and our stakeholders and include:

- Maintain bridges in a "state of good repair" using low-cost effective strategies.
- Implement timely preservation treatments on structurally sound bridges to promote optimal life cycle cost and extend service life. This will reduce the need for major rehabilitation and replacement – "right treatment at the right time".
- Promote and support budgeting of preventive maintenance activities.
- Establish performance goals and monitor progress related to preservation of bridges.
- Optimize the benefits and effectiveness of long-term maintenance investment in achieving bridges in good condition.

42.2.2 Strategies to Achieve WisDOT Bridge Preservation Goals

To achieve the goals of the bridge preservation program, WisDOT will use data-driven strategies. This approach is aimed at applying the appropriate bridge preservation treatments and activities at the proper time. These strategies are also aimed at maximizing efficiency and effectiveness of the program. The strategies of the WisDOT Bridge Preservation Program include:

- Regular analysis of the bridge inventory data to establish conditions and trends related to performance.
- Develop and maintain criteria for eligible preservation activities.
- Define preservation program and project needs (using HSIS and WISAMS).
- Develop estimates of needed financial resources at the project/program level.
- Prioritize, plan, and perform preservation treatments.

- As appropriate, group preservation maintenance projects to promote economy of scale.
- Identify preservation needs that complement maintenance, repair, and rehabilitation actions and timelines.
- Securing approval and support from key stakeholders in the use of Federal and State funding for systematic preventive maintenance and preservation activities.
- Utilize multiple programs to implement preservation work activities Improvement (Let) program, DMA, PBM & RMA maintenance programs.
- Develop and maintain records of preservation applications to analyze for cost and effectiveness of treatments.
- Consider preservation at the bridge design stage.

42.3 Bridge Preservation Actions

This chapter focuses on bridge preservation actions that relate to preventive maintenance and element rehabilitation. Cyclical and condition-based activities are subsets of preventative maintenance as shown below in Figure 42.3-1. Descriptions of these preservation actions can be found in 42.7.

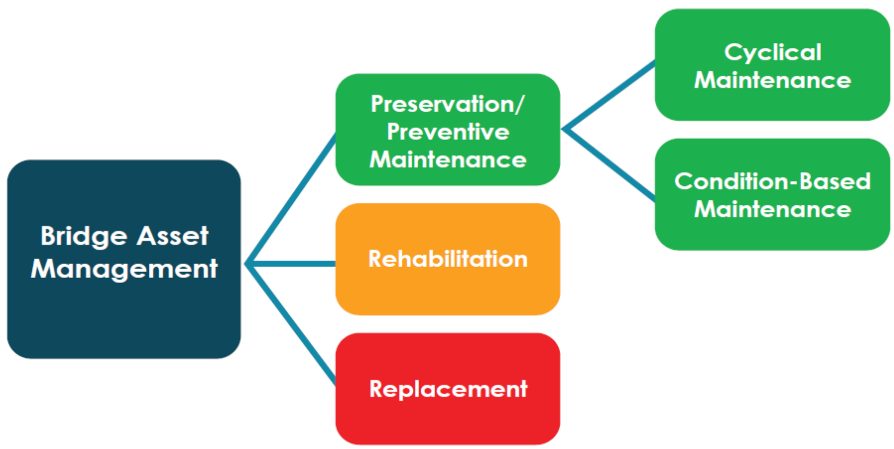

Figure 42.3-1
Asset Management and Preservation Actions

Major rehabilitation, bridge replacement, improvement, and new bridge construction projects are addressed by other WisDOT Bridge Programs.

42.4 Bridge Preservation Goals, Strategies and Performance Measures

This chapter outlines preservation goals, strategies and performance measures to track progress. Maintaining safe and dependable operations is a high priority for the department. **The Department has the goal to maintain 95% of the state-owned bridges in fair or better condition** (NBI ratings 5 or higher). To achieve this goal, the department employs strategies that include condition and cyclical treatments.

42.4.1 Condition Based Strategies

Condition based preventive maintenance activities are performed on bridge elements as needed and identified through the bridge inspection process. To achieve the goal of maintaining 95% of the state system bridge inventory in fair or better condition, maintaining key bridge elements or components that will promote this goal. These include:

- Bridge decks in good or fair condition (per NBI condition rating).
- Strip seal joints effective in stopping leakage. (effective joint)).
- Coated steel surfaces for superstructures in condition state 2 or better.
- Bearing elements in condition state 2 or better.

42.4.2 Cyclical Based Strategies

Cyclical based activities are performed on a pre-determined interval and aimed to preserve existing bridge element or component conditions. These types of activities may not improve the condition of the bridge element or component directly, but will delay their deterioration. Examples of cyclical activities include:

- Deck sweeping
- Deck and Superstructure washing
- Deck sealing.

42.4.3 Performance Measures and Objectives

Performance measures in this chapter are consistent with the objectives of the program and reflect the experience and input of the WisDOT Regional Bridge Maintenance Staff as well as consideration of other DOT’s insight and experience.

Table 42.4-1 lists the measures and objectives for preservation program performance:

Objective	Target/Goals	Performance Measure
Maintain bridges in good or fair condition	95% of bridges	Percentage of bridge in good or fair condition (NBI rating 5 or higher)
Maintain bridge decks in good or fair condition	95% of bridge decks	Percentage of bridge decks in good or fair condition (NBI Rating 5 or higher)
Maintain effective expansion joints that do not leak	85% of bridges with strip seal joints that are effective in stopping leakage	Percent of a bridges with 90% of their strip seal expansion joints in condition state 2 or better (effective joint)
Maintain coated steel surfaces in condition state 2 or better	90% of coated steel surfaces	Percentage of coated steel surfaces in condition state 2 or better (effective)
Maintain bearings in condition state 2 or better	95 % of bearings in condition state 2 or better	Percentage of bearings in condition state 2 or better
Seal eligible concrete decks (NBI rating 6 or higher) with sealant every 4 years	Seal 25% eligible concrete decks	Number of decks sealed (sq. ft of deck area) each year during a 4-year period

Table 42.4-1
Objectives and Performance Measures

42.4.4 Preservation Program Benefits

Each objective and measure proposed in Table 42.4-1 is aimed at extending the life of the main bridge components by performing timely cyclical or condition-based (corrective) preservation actions. The cost of performing preservation actions is minor when compared to premature replacement or rehabilitation of bridge components. The benefits of each objective are discussed below:

- Maintaining 95% of bridge decks in good or fair condition is an asset management approach that should extend the service life of bridges and promote the MAP21 objectives. Experience has shown that bridges designed for a 100-year life expectancy should have decks that last 55 with progressive preservation activities though the life of the bridge deck. Appropriate corrective actions taken as part of deck preservation extends the bridge deck life significantly. The costs of such corrective actions are substantially less than the costs of prematurely replacing the decks.

- The objective of maintaining 85% of strip seals in good or fair condition will focus on a program that will help in minimizing the damage on bridge superstructure and substructure components. Leaking joints cause significant deterioration and damage to bridge components that include girders, bearings, and substructures. There is significant cost each year in repairing structural elements that have deteriorated prematurely as a result of leaking joints. Maintaining effective (non-leaking) strip seals can delay superstructure and substructure deterioration.

- Maintaining protective paint systems is important. The structural components of the steel bridges will corrode and lose load carrying capacity if left unprotected or partially-protected. Protective paint coatings systems should have a service life of 25-40 years for the protection of structural steel. The objective of maintaining 90% of coated steel surfaces in good or fair condition will aim at creating a paint program for extending the life of steel components up to 100 years.

- Bridge bearings are a key component. Bearings support bridge super structures and allow for expansion of the superstructure. Experience has shown that loss of lubrication, tipping, or corrosion of bearings can cause harm to the deck and superstructure. The proposed measure of keeping 95% of bearings in good or fair condition will help WisDOT maintain bridges in a state of good repair.

- Objective of sealing 25% of all eligible concrete decks at 4 year intervals will help delay deck deterioration and prolong deck life. Sealing decks every 4 years at a minor cost can delay deck deterioration by 10-12 years that will promote increased deck life.

42.5 Bridge Preservation Activities, Eligibility and Need Assessment Criteria

The bridge preservation activities shown below relate to deck, superstructure and substructure elements. Table 42.5-1 shows the most common bridge preservation activities that are considered cost effective when applied to the appropriate bridge at the appropriate time, as well as considered eligible for bridge preservation funding. Additionally, these activities together with the eligibility and prioritization criteria discussed in this section will form a basis to generate an eligibility list of bridges that are candidates for cyclical and condition based PM actions.

<table>
<tr><th>Bridge Component</th><th>Bridge Preservation Type</th><th>Activity Description</th><th>Preventive Maintenance Type</th><th>Action Frequency (years)</th></tr>
<tr><td>All</td><td>Preventive Maintenance</td><td>Sweeping, power washing, cleaning</td><td>Cyclical</td><td>1-2</td></tr>
<tr><td rowspan="16">Deck</td><td rowspan="12">Preventive Maintenance</td><td>Deck washing</td><td rowspan="4">Cyclical</td><td>1</td></tr>
<tr><td>Deck sweeping</td><td>1</td></tr>
<tr><td>Deck sealing/crack sealing</td><td>4-5</td></tr>
<tr><td>Thin polymer (epoxy) overlays</td><td>7-15</td></tr>
<tr><td>Drainage cleaning/repair</td><td rowspan="8">Condition Based</td><td>As needed</td></tr>
<tr><td>Joint cleaning</td><td></td></tr>
<tr><td>Deck patching</td><td>1- 2</td></tr>
<tr><td>Chloride extraction</td><td>1 -2</td></tr>
<tr><td>Asphalt overlay with membrane</td><td>5-15</td></tr>
<tr><td>Polymer modified asphalt overlay</td><td>10-15</td></tr>
<tr><td>Joint seal replacement</td><td>10</td></tr>
<tr><td>Drainage cleaning/repair</td><td>1</td></tr>
<tr><td rowspan="4">Repair or Rehab Element</td><td>Rigid concrete overlays</td><td rowspan="4">Condition Based</td><td rowspan="4">As needed</td></tr>
<tr><td>Structural reinforced concrete overlay</td></tr>
<tr><td>Deck joint replacement</td></tr>
<tr><td>Eliminate joints</td></tr>
<tr><td rowspan="9">Super</td><td rowspan="2">Preventive Maintenance</td><td>Bridge approach restoration</td><td rowspan="2">Cyclical</td><td>2</td></tr>
<tr><td>Seat and beam ends washing</td><td>2</td></tr>
<tr><td rowspan="7">Repair or Rehab Element</td><td>Bridge rail restoration</td><td rowspan="7">Condition Based</td><td rowspan="7">As needed</td></tr>
<tr><td>Retrofit rail</td></tr>
<tr><td>Painting</td></tr>
<tr><td>Bearing restoration (replacement, cleaning, resetting)</td></tr>
<tr><td>Superstructure restoration</td></tr>
<tr><td>Pin and hanger replacement</td></tr>
<tr><td>Retrofit fracture critical members</td></tr>
<tr><td rowspan="3">Sub</td><td rowspan="3">Preventive Maintenance</td><td>Substructure restoration</td><td rowspan="3">Condition Based</td><td rowspan="3">As needed</td></tr>
<tr><td>Scour counter measure</td></tr>
<tr><td>Channel restoration</td></tr>
</table>

Table 42.5-1
Bridge Preservation Activities

42.5.1 Eligibility Criteria

This chapter includes two distinct matrices outlining eligibility criteria for preservation activities shown in Table 42.5-2 and Table 42.5-3. The first matrix relates to concrete deck/slab activities and the second matrix covers other bridge component activities. Bridge inspection information and data that is managed in HSIS and the WISAMS (Chapter 41.2.1) will be used to develop reports that quantify needs at the program and project level. This method will also serve to develop reports to monitor progress related to performance goals.

The deck/slab matrix shown in Table 42.5-2 is based on the NBI Item 58 - Condition Rating for decks and total deck/slab distress area. The distress area on a deck is quantified using inspection defects including delaminations, spalls, cracking, and scaling. Other deck inspection methods such as chain drag sounding, ground penetrating radar (GPR) surveys, infrared (IR) surveys, and chloride potentials may also be used in quantifying deck defects.

The matrix shown in Table 42.5-3 is based on listed NBI condition ratings and specific inspection element condition states. As with decks, information and data from HSIS will be used with this matrix as well.

Table 42.5-3 lso makes reference to “defects”. For a better understanding of this concept, the reader is referred to Appendix D of the AASHTO Manual for Bridge Element inspection. This appendix describes the element materials defined for this guide and the defects that may be observed for each condition state. Included are individual materials, such as reinforced and prestressed concrete, steel, timber, masonry, and other materials.

These matrices guide the user to select a preservation activity and also show the potential enhancement to the NBI values and anticipated service life increase as a result of that activity. Note that even though some preservation activities list no change to the potential result to the condition rating of NBI items, there is an inherent benefit both in the short and long term of these preservation activities to extend the current condition and ultimately extend the life of the bridge.

Sound engineering judgment is needed to decide if the recommended action is best suited for extending the life of the bridge.

	NBI Item 58	Top Deck Element Distress Area (%)	Bottom Deck Element Distress Area (%)	Preservation Activity	Benefit to Deck from Action	Application Frequency (in years)
Concrete Deck/Slab	≥7	-	-	Deck Sweeping/Washing	Extend Service Life	1 to 2
		5% < 3220 < 25%	-	Crack Sealing	Extend Service Life	3 to 5
		3220 CS3 + CS4 > 0%	-	Deck Sealing	Service life extended	3 to 5
		-	1080 < 5%	Full Depth Deck Patching	Service life maintained	As needed
		3210 CS3 + CS4 < 5%	1080 < 5%	Wearing Surface Patching	Service life maintained	As needed
		>20% (3220 OR 8911 CS3 + CS4) OR	(1140 OR 1150) < 20% for timber deck	Polymer Modified Asphalt Overlay	Service life extended	10 to 15
		>15% 3210 (applied to bare deck)				
		>20% (3210 OR 8911 CS3 + CS4) OR	1080 < 5% for concrete deck			
		>50% 3220 (reapplication)				
		>20% (3220 OR 8911 CS3 + CS4) OR	(1140 OR 1150) < 20% for timber deck	HMA w/ membrane	Service life extended	5 to 15
		>15% 3210 (applied to bare deck)				
		>20% (3210 OR 8911 CS3 + CS4) OR	1080 < 5% for concrete deck			
		>50% 3220 (reapplication)				
		3210 < 5%	1080 < 1%	Polyester Polymer Concrete	Service life extended	20 to 30
		3210 < 2% (applied to bare deck)	1080 < 1%	Thin Polymer Overlay	Service life extended	7 to 15
		8513 CS3 + CS4 > 15% (reapplication)				
	6	-	-	Deck Sweeping/Washing	Extend Service Life	1 to 2
		5% < 3220 < 25%	-	Crack Sealing	Extend Service Life	3 to 5
		3220 CS3 + CS4 > 0%	-	Deck Sealing	Service life extended	3 to 5
		-	1080 < 5%	Full Depth Deck Patching	Service life maintained	As needed
		3210 CS3 + CS4 < 5%	1080 < 5%	Wearing Surface Patching	Service life maintained	As needed
		>20% (3220 OR 8911 CS3 + CS4) OR	(1140 OR 1150) < 20% for timber deck	Polymer Modified Asphalt Overlay	Improve NBI (58) ≥ 7	10 to 15
		>15% 3210 (applied to bare deck)				
		>20% (3210 OR 8911 CS3 + CS4) OR	1080 < 5% for concrete deck			
		>50% 3220 (reapplication)				
		>20% (3220 OR 8911 CS3 + CS4) OR	(1140 OR 1150) < 20% for timber deck	HMA w/ membrane	Improve NBI (58) ≥ 7	5 to 15
		>15% 3210 (applied to bare deck)				
		>20% (3210 OR 8911 CS3 + CS4) OR	1080 < 5% for concrete deck			
		>50% 3220 (reapplication)				
		8513 CS3 + CS4 > 15% (reapplication)	1080 < 1%	Thin Polymer Overlay	Service life extended	7 to 15
		>20% (3220 OR 8911 CS3 + CS4) OR	1080 < 5% OR 1130 CS3 + CS4 < 25%	Concrete Overlay	Improve NBI (58) ≥ 7	12 to 20
		>15% 3210 (applied to bare deck)				
		>20% (3210 OR 8911 CS3 + CS4) OR				
		>50% 3220 (reapplication)				
	5	5% < 3220 < 25%	-	Crack Sealing	Extend Service Life	3 to 5
		3220 CS3 + CS4 > 0%	-	Deck Sealing	Service life extended	3 to 5
		-	1080 < 5%	Full Depth Deck Patching	Service life maintained	As needed
		3210 CS3 + CS4 < 5%	1080 < 5%	Wearing Surface Patching	Service life maintained	As needed
		>20% (3220 OR 8911 CS3 + CS4) OR	1080 < 5% OR 1130 CS3 + CS4 < 25%	Concrete Overlay	Improve NBI (58) ≥ 7	12 to 20
		>15% 3210 (applied to bare deck)				
		>20% (3210 OR 8911 CS3 + CS4) OR				
		>50% 3220 (reapplication)				
	4	>20% (3220 OR 8911 CS3 + CS4) OR	1080 < 5% OR 1130 CS3 + CS4 < 25%	Concrete Overlay	Improve NBI (58) ≥ 7	12 to 20
		>15% 3210 (applied to bare deck)				
		>20% (3210 OR 8911 CS3 + CS4) OR				
		>50% 3220 (reapplication)				
	≤ 4	-	1080 > 15% OR 1130 CS3 + CS4 > 50%	Deck Replacement	Improve NBI (58) = 9	25 to 45

Table 42.5-2
Concrete Deck/Slab Eligibility Matrix

NBI Item	Element	NBI Criteria	Defect	Element Defect Condition State Criteria	Repair Action	Potential Benefits to NBI or CS	Anticipated Service Life Years
Deck	Joints	Item 58 ≥ 5	2350	CS2, CS3, or CS4	Joint Cleaning	CS1or CS2	
			2310	CS3 + CS4 ≥ 10%	Joint Seal Replacement/Restoration	CS1	5 to 8
			2360	CS3 + CS4 ≥ 25%	Joint Replacement ④ ⑦	CS1	10 to 20
				All Condition State	Joint Elimination ④	Elimination	15 to 25
	Railing	Item 58 ≥ 5		CS3 or CS4	Railing Restoration	CS1 or CS2	3 to 10
				CS3 or CS4	Railing Replacement/Retrofit ⑧	CS1	10 to 20
Super	Steel Elements	Item 59 ≥ 5		N/A	Superstructure Washing/Cleaning	NA	1 to 2
			3440	CS2 + CS3 Area> 5% ⑥	Painting - Spot	CS1	1 to 5
				CS3 Area ≤ 25% ⑥	Painting - Zone	CS1 ①	5 to 7
				CS3 Area ≥ 25% ⑥	Painting - Complete	CS1 ②	15 to 20
		Item 59 ≥ 4		CS2, CS3, or CS4	Superstructure Restoration ③	NBI ≥ 7	5 to 20
	Bearings	Item 59 ≥ 5		CS3 or CS4	Bearing Reset/Repair	CS1 or CS2	1 to 5
				CS2 or CS3	Bearing Cleaning/Painting	CS1 or CS2	5 to 7
				CS3 + CS4 ≥ 25% or CS4 > 5%	Bearing Replacement	CS1	10 to 15
Sub	Miscellaneous	Item 60 ≥ 5		N/A	Substructure Washing/Cleaning	NA	1 to 2
			3440	CS2+CS3+CS4 Area > 5% ⑥	Painting - Spot	CS1	1 to 5
			3440	CS3 Area > 25% ⑥	Painting - Complete	CS1 ②	10 to 20
				CS2 or CS3 or CS4	Substructure Restoration ⑤	NBI ≥ 7	5 to 20
			9290	CS1 or CS2	Pier Protection ⑨	NBI ≥ 7	5 to 20
				CS3 or CS4	Scour Counter Measure ⑩	NBI ≥ 7	5 to 20

Table 42.5-3
Other Bridge Elements Eligibility Matrix

① Increase NBI only if combine with structural steel repairs.

② Complete painting only if combined with structural steel repairs to improve the component NBI ≥ 7.

③ Superstructure restoration includes all work related to the superstructure including but not limited to strengthening, pin and hanger replacement, retrofit FC member, etc.

④ Combined with deck overlay or replacement project.

⑤ Substructure restoration includes all work related to the substructure including but not limited to fiber wrapping, strengthening, crack injection, encapsulation, etc.—regardless of material type.

⑥ Element condition state for steel protective coating.

⑦ Includes but is not limited to end block/paving block replacement.

⑧ Must bring railing to current standards or have an approved exception to standards.

⑨ Examples are pier protection dolphins and fender systems.

⑩ Provide scour countermeasures after repairing any other substructure defects.

42.5.2 Identification of Preservation Needs

The identification of preservation needs will start with inventory and inspection information collected as part of the ongoing inspection program. The inspection information is analyzed by BOS asset management engineers with the Wisconsin Structures Asset Management System (WISAMS - 41.2.1). The analysis will include inspection reports, past work actions, and preservation policy logic as shown in Table 42.5-2 and Table 42.5-3. BOS will develop bridge work eligibility reports.

The programming of projects will start with the development of eligibility reports as defined in Chapter 41 – Structures Asset Management. Eligible work could be standalone projects or combined into roadway projects, or combined into a group that may include cyclical preventive maintenance activities. Programming of work will be through the Improvement (Let) program and various Maintenance programs (DMA, RMA, and PBM)

42.6 Funding Resources and Budgeting

The experiences of several states have shown that having commitments for funding preservation programs extends the life of bridges and defers untimely replacement. Having a commitment for funding of bridge preservation will help WisDOT optimize the overall bridge program. We promote the idea of recognizing and prioritizing preservation opportunities as part of the planning and programming functions of the department at the Division and Regional level. Through this organizational approach to implementation, preservation will yield the greatest system wide benefits. We recognize the ability to implement policy-driven bridge preservation work actions through a number of our department bridge improvement and maintenance programs. Some work actions are more appropriate for various programs depending on the scale, complexity, or resourcing of the work.

Implementing bridge improvements, rehabilitation, maintenance, and preservation activities occurs through a number of programs. These programs include:

- **Let Improvement Projects (Let).** The let improvement program is identified and programmed in Regional Planning and developed in Regional PDS. The projects are let to competitive bid on a regular schedule.

- **Discretionary Maintenance Agreement (DMA).** This is a contracting mechanism initiated by the Department with a county highway department for specific projects and locations. DMAs are typically used in response to highway or services maintenance research opportunities, or awarded as part of a targeted maintenance initiative.

- **Performance-based Maintenance (PbM).** Performance-based highway maintenance is based on the authority to contract with counties to perform specific highway maintenance tasks. Unlike Discretionary Maintenance Agreements which are paid based on actual cost reimbursement basis, PbM contracts are paid based on a negotiated contract price

- **Routine Maintenance Agreement (RMA).** Maintenance of state highways is performed by county highway departments under annual calendar year contracts called the Routine Maintenance Agreement (RMA) document. The RMA document provides each county with a state highway maintenance budget and the approval for expenditure within that budget.

Given our ability to use the structure inventory data and preservation policy to identify work actions from minor preservation activities through major reconstruction activities, we direct a range of work activates to various maintenance and improvement programs to promote appropriate actions throughout the complete lifecycle of the structures. This is shown in Figure 42.6-1 as the Overall Structures Program.

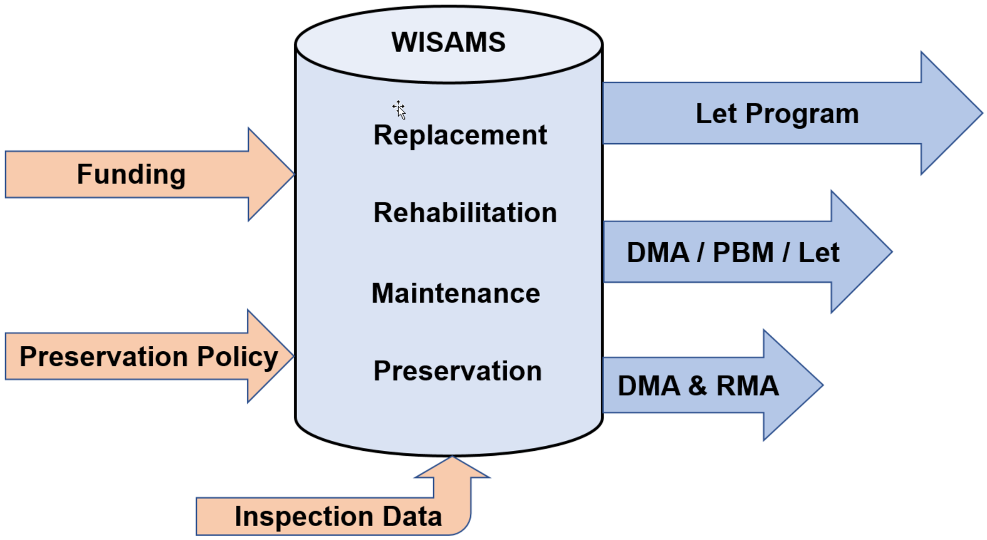

Figure 42.6-1
Overall Structures Program Diagram

Federal Funding and Preventive Maintenance

The May 2016 Agreement for the Use of Federal Funds for Preventive Maintenance of Structures [2], Section 5 (Special Limitations) outlines areas where certain work would not be eligible for federal funding in our improvement program, but could be included in our maintenance program with state funding. The following actions are usually considered ***routine maintenance*** and are not eligible for federal funding in the Let program under the WisDOT/FHWA agreement:

- Vehicle damage repair
- Asphalt deck patching
- Asphalt Overlay *without* Membrane
- Graffiti Removal
- Flood damage & minor channel debris removal

42.7 Definitions

Bridge Program: The WisDOT Bridge Program includes preservation, rehabilitation, improvement or major rehabilitation, replacement and new bridge construction actions.

Bridge Preservation: Bridge Preservation is defined as actions or strategies that prevent, delay, or reduce deterioration of bridges or bridge elements, restore the function of existing bridges, keep bridges in good or fair condition and extend their service life. Preservation actions may be cyclic or condition-driven.

Highway Structures Information System: Highway Structures Information System (HSIS) is the system developed by WisDOT for managing the inventory and inspection data of all highway structures. The inspection data is collected in accordance with the NBIS and *2013 AASHTO Manual for Bridge Element Inspection.*

Wisconsin Structures Asset Management System (WiSAMS): Automated application to determine optimal work candidates for improving the condition of structures. This application serves as a programming and planning tool for structures improvements, rehabilitations, maintenance, and preservation. This application coupled with the Highways Structures Information System (HSIS) serves as a comprehensive Structures (Bridge) Management system.

State of Good Repair (SGR): State of Good Repair (SGR) is a condition in which the existing physical assets, both individually and as a system (a) are functioning as designed within their useful service life, and (b) are sustained through regular maintenance and replacement programs. SGR represents just one element of a comprehensive capital investment program that also addresses system capacity and performance.[3]

Systematic Preventive Maintenance Program (SPM): Systematic Preventive Maintenance (SPM) is a planned strategy of cost-effective treatments to highway bridges that are intended to maintain or preserve the structural integrity and functionality of bridge elements and/or components, and retard future deterioration, thus maintaining or extending the useful life of bridges. An SPM program is based on a planned strategy that is equivalent to having a systematic process that defines the strategy, how it is planned, and how activities are determined to be cost effective. An SPM program may be applied to bridges at the network, highway system, or region-wide basis and have acceptable qualifying program parameters. The details on an SPM program and qualifying parameters are found in FHWA's *Bridge Preservation Guide.*

Preventive Maintenance(PM): Preventive maintenance is a cost-effective means to extend the service life of bridges. PM treatments retard future deterioration and avoid large expensive bridge rehabilitation or replacements. PM includes cyclic and condition based treatments.

Cyclical PM Activities: Cyclical PM activities are those activities performed on a pre-determined interval and aimed to preserve existing bridge element or component conditions. Bridge element or component conditions are not always directly improved as a result of these activities, however deterioration is expected to be delayed.

Condition Based PM Activities: Condition Based PM Activities are those activities that are performed on bridge elements in response to known defects as identified through the bridge inspection process.

Rehabilitation: Rehabilitation involves major work required to restore the structural integrity of a bridge as well as work necessary to correct major safety defects as defined in the Code of Federal Regulation (CFR) 23 clause 650.403.

Improvement or Major Rehab: Bridge improvement is a set of activities that fixes the deterioration found in a structure and improves the geometrics and load-carrying capacity beyond the original design standards, but may not provide improvement that meets new construction standards.

Replacement: Replacement of an existing bridge with a new facility constructed in the same general traffic corridor is considered total replacement. The replacement structure must meet the current geometric, construction, and structural standards as defined in the Code of Federal Regulation (CFR) 23 clause 650.403.

New Bridge Construction: The construction of a new bridge is defined as bridge construction that does not replace or relocate an existing bridge as described in FHWA's MAP-21 STP.

NBI Condition Rating: The FHWA coding guide describes the condition ratings used in evaluating four main components of a bridge as decks, superstructure, substructure, and culverts. The condition ratings are used to measure the deterioration level of bridges in a consistent and uniform manner to allow for comparison of the condition state of bridges on a national level. The condition ratings are also known as NBI ratings and are measured on a scale of 0 (worst) to 9 (excellent). For WisDOT bridges and culverts, an NBI rating of 4 is classified as poor, an NBI rating of 5 is classified as fair, and an NBI rating of 6 or higher is classified as 'good' (See Table 42.7-1).

Code	Description	Common Actions
9	**EXCELLENT CONDITION**	Preservation/Cyclic Maintenance
8	**VERY GOOD CONDITION**—No problems noted.	
7	**GOOD CONDITION**—Some minor problems.	
6	**SATISFACTORY CONDITION**—Structural elements show some minor deterioration.	Preservation/ Condition-Based Maintenance
5	**FAIR CONDITION**—All primary structural elements are sound but may have some minor section loss, cracking, spalling, or scour.	
4	**POOR CONDITION**—Advanced section loss, deterioration, spalling, or scour.	Rehabilitation or Replacement
3	**SERIOUS CONDITION**—Loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.	
2	**CRITICAL CONDITION**—Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present, or scour may have removed substructure support. Unless closely monitored, the bridge may have to be closed until corrective action is taken.	
1	**IMMINENT FAILURE CONDITION**—Major deterioration or section loss present in critical structural components, or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic, but corrective action may put it back in light service.	
0	**FAILED CONDITION**—Out of service. Bridge is beyond corrective action.	

Table 42.7-1
NBI General Condition Ratings & Common Actions

Element Condition State: A condition state categorizes the nature and extent of damage or deterioration of a bridge element. The 2013 *AASHTO Manual for Bridge Element Inspection* describes a comprehensive set of bridge elements mainly categorized as National Bridge Elements (NBE), Bridge Management Elements (BME) and Agency Develop Elements (ADE) and their corresponding four condition states. The element condition states1 to 4 are described as good (CS1), fair (CS2), poor (CS3), and severe (CS4).

Condition State	Description	Common Actions[10]
1	**Varies depending on element—Good**	Preservation/Cyclic Maintenance
2	**Varies depending on element—Fair**	Cyclic Maintenance or Condition-Based Maintenance when cost effective.
3	**Varies depending on element—Poor**	Condition-Based Maintenance, or Rehabilitation—when quantity of poor exceeds a limit that condition-based maintenance is not cost effective, or Replacement—when rehabilitation is not cost effective.
4	**Varies depending on element—Severe**	Rehabilitation or Replacement

Table 42.7-2
Element Condition States & Common Actions

42.8 References

1. *Bridge Preservation Guide*, *Maintaining a Resilient Infrastructure to Preserve Mobility* (FHWA) – Spring 2018, (https://www.fhwa.dot.gov/bridge/preservation/guide/guide.pdf)

2. FDM 3-1 Exhibit 5.2 Agreement for the Use of Federal Funds for Preventive Maintenance of Structures. (May 2016). (https://wisconsindot.gov/rdwy/fdm/fd-03-05-e0502.pdf#fd3-5e5.2)

3. Source: U.S. DOT Secretary Mary Peters July 25, 2008 letter to Congress

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E45-1 Reinforced Concrete Slab Rating Example - LRFR

The 3-span continuous haunched slab structure shown in the Design Example from Chapter 18 is rated below. This same basic procedure is applicable for flat slab structures. For LRFR, the Bureau of Structures rates concrete slab structures for the Design Load (HL-93) and for Permit Vehicle Loads on an Interior Strip. The Permit Vehicle may be the Wisconsin Standard Permit Vehicle (Wis-SPV) or an actual Single-Trip Permit Vehicle. This bridge was analyzed using a slab width equal to one foot.

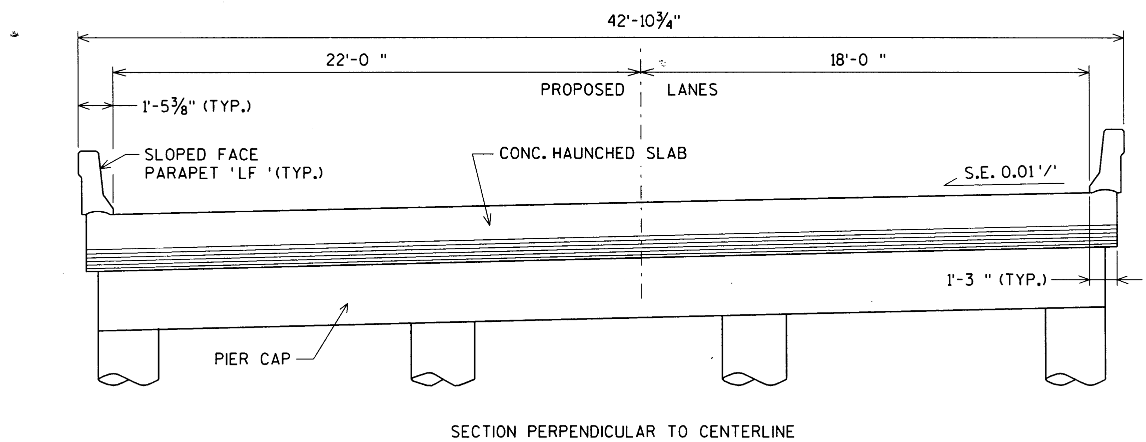


Figure E45-1.1

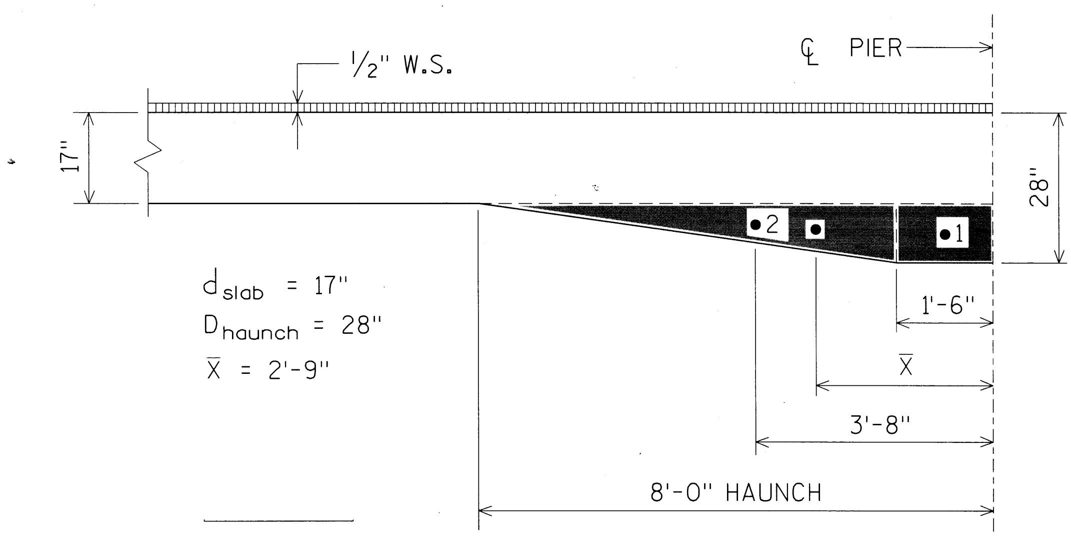


Figure E45-1.2

E45-1.1 Design Criteria

Geometry:

$L_1 := 38.0$	ft	Span 1 Length
$L_2 := 51.0$	ft	Span 2 Length
$L_3 := 38.0$	ft	Span 3 Length
$slab_{width} := 42.5$	ft	out to out width of slab
$skew := 6$	deg	skew angle (RHF)
$w_{roadway} := 40.0$	ft	clear roadway width
$cover_{top} := 2.5$	in	concrete cover on top bars (includes 1/2in wearing surface)
$cover_{bot} := 1.5$	in	concrete cover on bottom bars
$d_{slab} := 17$	in	slab depth (not including 1/2in wearing surface)
$D_{haunch} := 28$	in	haunch depth (not including 1/2in wearing surface)
$A_{st\_0.4L} := 1.71$	$\frac{in^2}{ft}$	Area of longitudinal bottom steel at 0.4L (# 9's at 7in centers)
$A_{st\_pier} := 1.88$	$\frac{in^2}{ft}$	Area of longitudinal top steel at Pier (# 8's at 5in centers)

Material Properties:

$f'_c := 4$	ksi	concrete compressive strength
$f_y := 60$	ksi	yield strength of reinforcement
$E_c := 3800$	ksi	modulus of elasticity of concrete
$E_s := 29000$	ksi	modulus of elasticity of reinforcement
$n := 8$		E_s / E_c (modular ratio)

Weights:

$w_c := 150$	pcf	concrete unit weight
$w_{LF} := 387$	plf	weight of Type LF parapet (each)

E45-1.2 Analysis of an Interior Strip - one foot width

Use Strength Limit States to rate the concrete slab bridge. **MBE [6A.4.2.2]**

The influence of ADTT and skew on force effects are ignored for slab bridges (See 18.3.2.2).

E45-1.2.1 Dead Loads (DC, DW)

The slab dead load, DC_{slab}, and the section properties of the slab, do not include the 1/2 inch wearing surface. But the 1/2 inch wearing surface load, DC_{WS}, of 6 psf must be included in the analysis of the slab. For a one foot slab width:

$DC_{ws} := 6$ 1/2 inch wearing surface load, plf

The parapet dead load is uniformly distributed over the full width of the slab when analyzing an Interior Strip. For a one foot slab width:

$$DC_{para} := 2 \cdot \frac{w_{LF}}{slab_{width}} \qquad \boxed{DC_{para} = 18} \quad \text{plf}$$

The unfactored dead load moments, M_{DC}, due to slab dead load (DC_{slab}), parapet dead load (DC_{para}), and the 1/2 inch wearing surface (DC_{ws}) are shown in Chapter 18 Example (Table E18.4).

The structure was designed for a possible future wearing surface, DW_{FWS}, of 20 psf.

$DW_{FWS} := 20$ Possible wearing surface, plf

E45-1.2.2 Live Load Distribution (Interior Strip)

Live loads are distributed over an equivalent width, E, as calculated below.
The live loads to be placed on these widths are axle loads (i.e., two lines of wheels) and the full lane load. The equivalent distribution width applies for both live load moment and shear.

Single - Lane Loading: $E = 10.0 + 5.0 \cdot (L_1 \cdot W_1)^{0.5}$ in

Multi - Lane Loading: $E = 84.0 + 1.44 \cdot (L_1 \cdot W_1)^{0.5} \leq 12.0 \cdot \frac{W}{N_L}$ in

Where:

L_1 = modified span length taken equal to the lesser of the actual span or 60ft (L_1 in ft)

W_1 = modified edge to edge width of bridge taken to be equal to the lesser of the actual width or 60ft for multi-lane loading, or 30ft for single-lane loading (W_1 in ft)

W = physical edge to edge width of bridge (W in ft)

N_L = number of design lanes as specified in **LRFD [3.6.1.1.1]**

For single-lane loading:

(Span 1, 3) $E := 10.0 + 5.0\cdot(38\cdot 30)^{0.5}$ $\boxed{E = 178.819}$ in

(Span 2) $E := 10.0 + 5.0\cdot(51\cdot 30)^{0.5}$ $\boxed{E = 205.576}$ in

For multi-lane loading:

$$12.0\cdot\frac{W}{N_L} = 12.0\cdot\frac{42.5}{3} = 170 \quad \text{in}$$

(Span 1, 3) $E := 84.0 + 1.44\cdot(38\cdot 42.5)^{0.5}$ $\boxed{E = 141.869}$ in <170" O.K.

(Span 2) $E := 84.0 + 1.44\cdot(51\cdot 42.5)^{0.5}$ $\boxed{E = 151.041}$ in <170" O.K.

E45-1.2.3 Nominal Flexural Resistance: (M_n)

The depth of the compressive stress block, (a) is (See 18.3.3.2.1):

$$a = \frac{A_s\cdot f_s}{\alpha_1\cdot f'_c\cdot b}$$

where:

A_s = area of developed reinforcement at section (in²)

f_s = stress in reinforcement (ksi)

$f'_c = 4$ ksi

$b := 12$ in

$\alpha_1 := 0.85$ (for $f'_c \leq 10.0$ ksi) **LRFD [5.6.2.2]**

As shown throughout the Chapter 18 Example, when f_s is assumed to be equal to f_y, and is used to calculate (a), the value of c/d_s will be < 0.6 (for f_y = 60 ksi) per **LRFD [5.6.2.1]** Therefore the assumption that the reinforcement will yield ($f_s = f_y$) is correct. The value for (c) and (d_s) are calculated as:

$$c = \frac{a}{\beta_1}$$

$\beta_1 := 0.85$

d_s = slab depth(excl. 1/2" wearing surface) - bar clearance - 1/2 bar diameter

For rectangular sections, the nominal moment resistance, M_n, (tension reinforcement only) equals:

$$M_n = A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$$

Minimum Reinforcement Check

All sections throughout the bridge meet minimum reinforcement requirements, because this was checked in the chapter 18 Design example. Therefore, no adjustment to nominal resistance (M_n) or moment capacity is required. **MBE [6A.5.6]**

E45-1.2.4 General Load - Rating Equation (for flexure)

$$RF = \frac{C - (\gamma_{DC}) \cdot (M_{DC}) - (\gamma_{DW}) \cdot (M_{DW})}{\gamma_L \cdot (M_{LL\_IM})}$$

MBE [6A.4.2.1]

For the Strength Limit State:

$$C = (\phi_c)(\phi_s)(\phi) \cdot R_n$$

where:

$R_n = M_n$ (for flexure)

$(\phi_c)(\phi_s) \geq 0.85$

Factors affecting Capacity (C):

Resistance Factor (ϕ), for Strength Limit State **MBE [6.5.3]**

$\phi := 0.9$ for flexure (all reinforced concrete section in the Chapter 18 Example were found to be tension-controlled sections as defined in **LRFD [5.6.2.1]**).

Condition Factor (ϕ_c) per Chapter 45.3.2.4

$\phi_c := 1.0$

System Factor (ϕ_s) Per Chapter 45.3.2.5

$\phi_s := 1.0$ for a slab bridge

E45-1.2.5 Design Load (HL-93) Rating

Use Strength I Limit State to find the Inventory and Operating Ratings **MBE [6A.4.2.2, 6A.5.4.1]**

Equivalent Strip Width (E) and Distribution Factor (DF):

Use the smaller equivalent width (single or multi-lane), when (HL-93) live load is to be distributed, for Strength I Limit State. Multi-lane loading values will control for this bridge.

The distribution factor, DF, is computed for a slab width equal to one foot.

$$DF = \frac{1}{E}$$ (where E is in feet)

The multiple presence factor, m, has been included in the equations for distribution width, E, and therefore is not used to adjust the distribution factor, DF, **LRFD [3.6.1.1.2]**.

Spans 1 & 3:

DF = 1/(141"/12) = 0.0851 lanes / ft-slab

Span 2:

DF = 1/(151"/12) = 0.0795 lanes / ft-slab

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.
Therefore use: $DF := 0.0851$s / ft-slab for all spans.

Dynamic Load Allowance (IM)

$IM := 33 \quad \%$ **MBE [6A.4.4.3]**

Live Loads (LL)

The live load combinations used for Strength I Limit State are shown in the Chapter 18 Example in Table E18.2 and E18.3. The unfactored moments due to Design Lane, Design Tandem, Design Truck and 90%{Double Design Truck + Design Lanes] are shown in Chapter 18 Example (Table E18.4).

Rating for Flexure

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

Load Factors

$\gamma_{DC} := 1.25$	Chapter 45 Table 45.3-1
$\gamma_{DW} := 1.50$	WisDOT policy is to always use 1.50; Chapter 45 Table 45.3-1
$\gamma_{Li} := 1.75$	(Inventory Rating) Chapter 45 Table 45.3-1
$\gamma_{Lo} := 1.35$	(Operating Rating) Chapter 45 Table 45.3-1

The Design Load Rating was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location, for this example, is in span 1 at the 0.4 pt.

Span 1 (0.4 pt.)

Inventory:

$$RF_i = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_{Li}\cdot(M_{LL\_IM})}$$

$A_{st\_0.4L} = 1.71 \dfrac{in^2}{ft}$ and $\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi) **LRFD [5.6.2.2]**

$d_s := 17.0 - cover_{bot} - \dfrac{1.128}{2}$ $\boxed{d_s = 14.94}$ in

$a := \dfrac{A_{st\_0.4L}\cdot f_y}{\alpha_1\cdot f'_c\cdot b}$ $\boxed{a = 2.51}$ in

$M_n := A_{st\_0.4L}\cdot f_y\cdot\left(d_s - \dfrac{a}{2}\right)$ $\boxed{M_n = 1403.4}$ kip – in

$\boxed{M_n = 117.0}$ kip – ft

$M_{DC} := 18.1$ kip – ft (from Chapter 18 Example, Table E18.4)

$M_{DW} := 0.0$ kip – ft (additional wearing surface not for HL-93 rating runs)

The positive live load moment shall be the largest caused by the following (from Chapter 18 Example, Table E18.4):

Design Tandem (+IM) + Design Lane: (37.5 kip-ft + 7.9 kip-ft) = 45.4 kip-ft
Design Truck (+IM) + Design Lane: (35.4 kip-ft + 7.9 kip-ft) = 43.3 kip-ft

Therefore:

$M_{LL\_IM} := 45.4$ kip – ft

Inventory:

$$RF_i := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_{Li}\cdot(M_{LL\_IM})}$$

$\boxed{RF_i = 1.04}$

Operating:

$$RF_o := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_{Lo}\cdot(M_{LL\_IM})}$$

$\boxed{RF_o = 1.35}$

Rating for Shear:

Slab bridge designed for dead load and (HL-93) live load moments in conformance with **LRFD [4.6.2.3]** may be considered satisfactory in shear **LRFD [5.12.2.1]**. This bridge was designed using this procedure, therefore a shear rating is not required.

The Rating Factors, RF, for Inventory and Operating Rating are shown on the plans and also on the load rating summary sheet.

E45-1.2.6 Permit Vehicle Load Ratings

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 load allowance is utilized. Future wearing surface will not be considered.

Since this example is rating a newly designed bridge, an additional check is required. The designer shall ensure that the results of a single-lane analysis utilizing the future wearing surface are greater than 190 kips MVW.

Use Strength II Limit State to find the Permit Vehicle Load Rating **MBE[6A.4.2.2, 6A.5.4.2.1]**.

E45-1.2.6.1 Wis-SPV Permit Rating with Single Lane Distribution w/ FWS

Equivalent Strip Width (E) and Distribution Factor (DF)

The equivalent width from single-lane loading is used, when Permit Vehicle live load is to be distributed, for Strength II Limit State **MBE [6A.4.5.4.2]**.

Calculate the distribution factor, DF, and divide it by (1.20) to remove the effects of the multiple presence factor (m), which are present in the equation for equivalent width (E) **MBE [6A.3.2, C6A.4.5.4.2b]**.

The distribution factor, DF, is computed for a slab width equal to one foot.

$$DF = \frac{1}{E \cdot (1.20)} \qquad \text{(where E is in feet)}$$

Spans 1 &3:

DF = 1/(178"/12)(1.20) = 0.0562 lanes / ft-slab

Span 2:

DF = 1/(205"/12)(1.20) = 0.0488 lanes / ft-slab

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Therefore use: $DF := 0.0562$s / ft-slab for all spans.

Dynamic Load Allowance (IM)

$IM = 33 \quad \%$ **MBE [6A.4.5.5]**

Rating for Flexure

$$RF = \frac{(\phi_c)(\phi_s)(\phi) \cdot M_n - (\gamma_{DC}) \cdot (M_{DC}) - (\gamma_{DW}) \cdot (M_{DW})}{\gamma_L \cdot (M_{LL\_IM})}$$

Load Factors

$\gamma_{DC} := 1.25$ Chapter 45 Table 45.3-1

$\gamma_{DW} := 1.50$ WisDOT policy is to always use 1.50; Chapter 45 Table 45.3-1

$\gamma_L := 1.20$ WisDOT Policy is to designate the (Wis_SPV) as a "Single-Trip" vehicle with no escorts. Current policy is to select the value for γ_L from Chapter 45 Table 45.3-3

The Maximum Permit Vehicle Load was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location is the C/L of Pier.

At C/L of Pier

Permit Vehicle:

$$RF = \frac{(\phi_c)(\phi_s)(\phi) \cdot M_n - (\gamma_{DC}) \cdot (M_{DC}) - (\gamma_{DW}) \cdot (M_{DW})}{\gamma_L \cdot (M_{LL\_IM})}$$

$A_{st\_pier} := 1.88 \dfrac{in^2}{ft}$ and $\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi) **LRFD [5.6.2.2]**

$d_s := 28.0 - (cover_{top} - 0.5) - \dfrac{1.00}{2}$ $\boxed{d_s = 25.5}$ in

$a := \dfrac{A_{st\_pier} \cdot f_y}{\alpha_1 \cdot f'_c \cdot b}$ $\boxed{a = 2.76}$ in

$M_n := A_{st\_pier} \cdot f_y \cdot \left(d_s - \dfrac{a}{2}\right)$ $\boxed{M_n = 2720.5}$ kip – in

$\boxed{M_n = 226.7}$ kip – ft

$M_{DC} := 59.2\ kip - ft$ (from Chapter 18 Example, Table E18.4)

$M_{DW} := 1.5\ kip - ft$

The live load moment at the C/L of Pier due to the Wisconsin Standard Permit Vehicle (Wis-SPV) having a gross vehicle load of 190 kips and utilizing single lane distribution is:

$$M_{LL\_IM} := 65.2 \quad kip - ft$$

Permit:

$$RF_{permit} := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

$$\boxed{RF_{permit} = 1.63}$$

The maximum Wisconsin Standard Permit Vehicle (Wis_SPV) load is:

$RF_{permit}\cdot(190) = 310$ kips which is > 190k, Check OK

This same procedure used for the (Wis-SPV) can also be used when evaluating the bridge for an actual "Single-Trip Permit" vehicle.

Rating for Shear:

WisDOT does not rate Permit Vehicles on slab bridges based on shear.

E45-1.2.6.2 Wis-SPV Permit Rating with Single Lane Distribution w/o FWS

Equivalent Strip Width (E) and Distribution Factor (DF)

The equivalent width from single-lane loading is used, when Permit Vehicle live load is to be distributed, for Strength II Limit State **MBE [6A.4.5.4.2]**.

Calculate the distribution factor, DF, and divide it by (1.20) to remove the effects of the multiple presence factor (m), which are present in the equation for equivalent width (E) **MBE [6A.3.2, C6A.4.5.4.2b]**.

The distribution factor, DF, is computed for a slab width equal to one foot.

$$DF = \frac{1}{E\cdot(1.20)} \qquad \text{(where E is in feet)}$$

Spans 1 &3:

DF = 1/(178"/12)(1.20) = 0.0562 lanes / ft-slab

Span 2:

DF = 1/(205"/12)(1.20) = 0.0488 lanes / ft-slab

Look at the distribution factor calculated for each span and select the largest value.
This single value is to be applied along the entire length of the bridge.

Therefore use: $DF := 0.0562$ / ft-slab for all spans.

Dynamic Load Allowance (IM)

$IM = 33 \quad \%$ **MBE [6A.4.5.5]**

Rating for Flexure

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

Load Factors

$\gamma_{DC} := 1.25$ Chapter 45 Table 45.3-1

$\gamma_L := 1.20$ WisDOT Policy is to designate the (Wis_SPV) as a "Single-Trip" vehicle with no escorts. Current policy is to select the value for γ_L from Chapter 45 Table 45.3-3

The Maximum Permit Vehicle Load was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location is the C/L of Pier.

At C/L of Pier

Permit Vehicle:

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

$A_{st\_pier} := 1.88 \frac{in^2}{ft}$ and $\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi) **LRFD [5.6.2.2]**

$d_s := 28.0 - (cover_{top} - 0.5) - \frac{1.00}{2}$ $\boxed{d_s = 25.5}$ in

$a := \frac{A_{st\_pier}\cdot f_y}{\alpha_1\cdot f'_c\cdot b}$ $\boxed{a = 2.76}$ in

$M_n := A_{st\_pier}\cdot f_y\cdot\left(d_s - \frac{a}{2}\right)$ $\boxed{M_n = 2720.5}$ kip – in

$\boxed{M_n = 226.7}$ kip – ft

$M_{DC} := 59.2\,kip - ft$ (from Chapter 18 Example, Table E18.4)

The live load moment at the C/L of Pier due to the Wisconsin Standard Permit Vehicle (Wis-SPV) having a gross vehicle load of 190 kips and utilizing single lane distribution is:

$M_{LL\_IM} := 65.2$ kip – ft

Permit:

$$RF_{permit} := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC})}{\gamma_L\cdot(M_{LL\_IM})}$$

$$\boxed{RF_{permit} = 1.66}$$

The maximum Wisconsin Standard Permit Vehicle (Wis_SPV) load is:

$RF_{permit}\cdot(190) = 316$ kips

This same procedure used for the (Wis-SPV) can also be used when evaluating the bridge for an actual "Single-Trip Permit" vehicle.

E45-1.2.6.3 Wis-SPV Permit Rating with Multi Lane Distribution w/o FWS

Rating for Flexure

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC})}{\gamma_L\cdot(M_{LL\_IM})}$$

The capacity of the bridge to carry the Permit Vehicle Load was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location is at the C/L of Pier.

Load Factors

$\gamma_{DC} := 1.25$ Chapter 45 Table 45.3-1

$\gamma_{DW} := 1.50$ WisDOT policy is to always use 1.50; Chapter 45 Table 45.3-1

$\gamma_L := 1.30$ WisDOT Policy when analyzing the Wis-SPV as an "Annual Permit" vehicle with no escorts

At C/L of Pier

Permit Vehicle:

$$RF_{permit} = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC})}{\gamma_L\cdot(M_{LL\_IM})}$$

$M_n = 226.7$ kip – ft (as shown previously)

$M_{DC} = 59.2$ kip – ft (as shown previously)

The live load moment at the C/L of Pier due to the Wisconsin Permit Vehicle (Wis_SPV) having a gross vehicle load of 190 kips and a DF of 0.0851 lanes/ft-slab:

$M_{LL\_IM} := 98.7$ kip – ft

$$RF_{permit} := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC})}{\gamma_L\cdot(M_{LL\_IM})}$$

$$\boxed{RF_{permit} = 1.01}$$

The Wisconsin Standard Permit Vehicle (Wis_SPV) load that can be carried by the bridge is:

$RF_{permit}\cdot(190) = 193$ kips

E45-1.3 Summary of Rating

Slab - Interior Strip							
Limit State		Design Load Rating		Legal Load Rating	Permit Load Rating (kips)		
		Inventory	Operating		Single DF w/ FWS	Single DF w/o FWS	Multi DF w/o FWS
Strength I	Flexure	1.04	1.34	N/A	310	316	193
Service I		N/A	N/A	N/A	Optional	Optional	Optional

E45-2.5.3 Live Load Moments

The unfactored live load load moments (per lane including impact) are listed below (values are in kip-ft). Note that the dynamic load allowance is applied only to the truck portion of the HL-93 loads.

Unfactored Live Load + Impact Moments per Lane (kip-ft)		
Tenth Point	Truck	Tandem
0	0	0
0.1	1783	1474
0.2	2710	2618
0.3	4100	3431
0.4	4665	3914
0.5	4828	4066

The unfactored live load moments per lane are calculated by applying the appropriate distribution factor to the controlling moment. For the interior girder:

$$g_i = 0.636$$

$$M_{LLIM} := g_i \cdot 4828 \qquad \boxed{M_{LLIM} = 3073} \text{ kip-ft}$$

E45-2.6 Compute Nominal Flexural Resistance at Midspan

At failure, we can assume that the tendon stress is:

$$f_{ps} = f_{pu}\left(1 - k \cdot \frac{c}{d_p}\right)$$

where:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right)$$

From **LRFD Table [C5.6.3.1.1-1]**, for low relaxation strands, $k := 0.28$.

"c" is defined as the distance between the neutral axis and the compression face (inches).

Assumed dimensions:

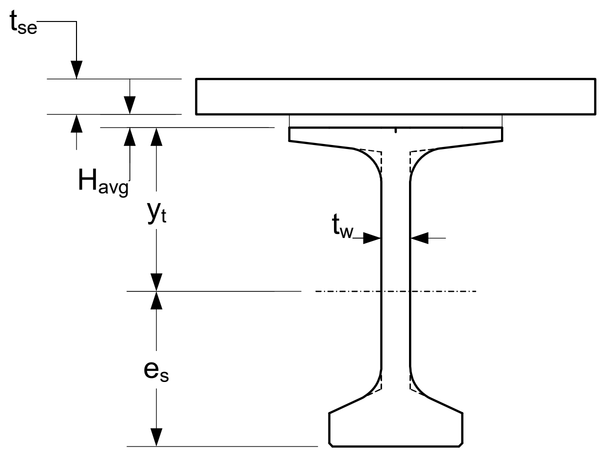


Figure E45-2.4

Assume that the compression block is in the deck. Calculate the capacity as if it is a rectangular section (with the compression block in the flange). The neutral axis location,calculated in accordance with **LRFD 5.6.3.1.1** for a rectangular section, is:

$$c = \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$

where:

$A_{ps} := ns \cdot A_s$	$A_{ps} = 9.98$	in^2
$b := b_{eff}$	$b = 90.00$	in
LRFD [5.6.2.2] $\alpha_1 := 0.85$ (for $f'_{cd} \leq 10.0$ ksi)		
$\beta_1 := \max\left[0.85 - \left(f'_{cd} - 4\right) \cdot 0.05, 0.65\right]$	$\beta_1 = 0.850$	
$d_p := y_t + H_{avg} + t_{se} - e_s$	$d_p = 77.15$	in
$c := \dfrac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$	$c = 9.99$	in
$a := \beta_1 \cdot c$	$a = 8.49$	in

The calculated value of "a" is greater than the deck thickness. Therefore, the rectangular assumption is incorrect and the compression block extends into the haunch. Calculate the neutral axis location and capacity for a flanged section:

$h_f := t_{se}$	depth of compression flange	$t_{se} = 7.500$	in
$b_{tf} = 48.00$	width of top flange, inches		

$$c := \frac{A_{ps} \cdot f_{pu} - \alpha_1 \cdot f'_{cd} \cdot (b - b_{tf}) \cdot h_f}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b_{tf} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}} \qquad \boxed{c = 10.937} \text{ in}$$

$$a := \beta_1 \cdot c \qquad \boxed{a = 9.30} \text{ in}$$

This is above the base of the haunch (9.5 inches) and nearly to the web of the girder. Assume OK.
Now calculate the effective tendon stress at ultimate:

$$f_{ps} := f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right) \qquad \boxed{f_{ps} = 259.283} \text{ ksi}$$

$$T_u := f_{ps} \cdot A_{ps} \qquad \boxed{T_u = 2588} \text{ kips}$$

Calculate the nominal moment capacity of the composite section in accordance with **LRFD [5.6.3.2], [5.6.3.2.2]**:

$$M_n := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + \alpha_1 \cdot f'_{cd} \cdot (b - b_{tf}) \cdot h_f \cdot \left(\frac{a}{2} - \frac{h_f}{2}\right)\right] \cdot \frac{1}{12}$$

$$\boxed{M_n = 15717} \text{ kip-ft}$$

For prestressed concrete, $\phi_f := 1.00$, **LRFD [5.5.4.2]**. Therefore the usable capacity is:

$$M_r := \phi_f \cdot M_n \qquad \boxed{M_r = 15717} \text{ kip-ft}$$

Check Minimum Reinforcement
The amount of reinforcement must be sufficient to develop M_r equal to the lesser of M_{cr} or 1.33 M_u per **LRFD [5.6.3.3]**

$$\gamma_{LL} := 1.75 \qquad \gamma_{DC} = 1.250 \qquad \eta := 1.0$$

$$Mu := \eta \cdot \left[\gamma_{DC} \cdot (M_{DC1} + M_{DC2}) + \gamma_{LL} \cdot M_{LLIM}\right] \qquad \boxed{Mu = 11832} \text{ kip-ft}$$

$$\boxed{1.33 \cdot Mu = 15737} \text{ kip-ft}$$

Calculate M_{cr} next and compare its value with 1.33 Mu

M_{cr} is calculated as follows:

$f_r = 0.24 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := 0.24 \cdot \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.679}$ ksi

$$f_{cpe} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_b}$$ $\boxed{f_{cpe} = 4.341}$ ksi

$M_{dnc} := M_{DC1}$ $\boxed{M_{dnc} = 4820}$ kip-ft

$S_c := -S_{cgb}$ $\boxed{S_c = 24650}$ ksi

$S_{nc} := -S_b$ $\boxed{S_{nc} = 18825}$ ksi

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_2 := 1.1$ prestress variability factor

$\gamma_3 := 1.0$ for prestressed concrete members

$$M_{cr} := \gamma_3 \cdot \left[S_c \cdot \left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) \cdot \frac{1}{12} - M_{dnc} \cdot \left(\frac{S_c}{S_{nc}} - 1 \right) \right]$$ $\boxed{M_{cr} = 10547}$ kip-ft

$M_{cr} = 10547$ kip-ft < $1.33Mu = 15737$, therefore M_{cr} controls

This satisfies the minimum reinforcement check since $M_{cr} < M_r$

Elastic Shortening Loss

at transfer (before ES loss) **LRFD [5.9.3.2]**

$T_{oi} := ns \cdot f_{tr} \cdot A_s$ $\boxed{= 46 \cdot 202.5 \cdot 0.217 = 2021}$ kips

The ES loss estimated above was: $\Delta f_{pES\_est} := 17$ ksi, or $ES_{loss} = 7.900$ %. The resulting force in the strands after ES loss:

$$T_o := \left(1 - \frac{ES_{loss}}{100}\right) \cdot T_{oi}$$ $\boxed{T_o = 1862}$ kips

If we assume all strands are straight we can calculate the initial elastic shortening loss;

$$f_{cgp} := \frac{T_o}{A_g} + \left(T_o \cdot e_s\right) \cdot \frac{e_s}{I_g} + M_g \cdot 12 \cdot \frac{e_s}{I_g}$$ $\boxed{f_{cgp} = 3.240}$ ksi

$\boxed{E_{ct} = 4999}$ ksi

$$E_p := E_s$$ $\boxed{E_p = 28500}$ ksi

$$\Delta f_{pES} := \frac{E_p}{E_{ct}} \cdot f_{cgp}$$ $\boxed{\Delta f_{pES} = 18.471}$ ksi

$$f_i := f_{tr} - \Delta f_{pES}$$ $\boxed{f_i = 184.029}$ ksi

Approximate Estimate of Time Dependant Losses

Calculate the components of the time dependant losses; shrinkage, creep and relaxation, using the approximate method in accordance with **LRFD [5.9.3.3]**.

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

From **LRFD [Figure 5.4.2.3.3-1]**, the average annual ambient relative humidity, $H := 72$ %.

$$\gamma_h := 1.7 - 0.01 \cdot H$$ $\boxed{\gamma_h = 0.980}$

$$\gamma_{st} := \frac{5}{1 + f'_{ci}}$$ $\boxed{\gamma_{st} = 0.641}$

$\Delta f_{pR} := 2.4$ ksi for low relaxation strands

$$\Delta f_{pCR} := 10.0 \cdot \frac{f_{tr} \cdot A_s \cdot ns}{A_g} \cdot \gamma_h \cdot \gamma_{st}$$ $\boxed{\Delta f_{pCR} = 13.878}$ ksi

$$\Delta f_{pSR} := 12.0 \cdot \gamma_h \cdot \gamma_{st}$$ $\boxed{\Delta f_{pSR} = 7.538}$ ksi

$$\Delta f_{pRE} := \Delta f_{pR}$$ $\boxed{\Delta f_{pRE} = 2.400}$ ksi

$$\Delta f_{pLT} := \Delta f_{pCR} + \Delta f_{pSR} + \Delta f_{pRE}$$ $\boxed{\Delta f_{pLT} = 23.816}$ ksi

The total estimated prestress loss (Approximate Method):

$$\Delta f_p := \Delta f_{pES} + \Delta f_{pLT}$$ $\boxed{\Delta f_p = 42.288}$ ksi

$\boxed{\frac{\Delta f_p}{f_{tr}} \cdot 100 = 20.883}$ % total prestress loss

The remaining stress in the strands and total force in the beam after all losses is:

$$f_{pe} := f_{tr} - \Delta f_p$$ $\boxed{f_{pe} = 160.21}$ ksi

E45-2.7 Compute Nominal Shear Resistance at First Critical Section

Note: **MBE [6A.5.8]** does not require a shear evaluation for the Design Load Rating or the Legal Load Rating provided the bridge shows no visible sign of shear distress. However, for this example, we will show one iteration for the Design Load Rating.

The shear analysis is always required for Permit Load Rating.

The following will illustrate the calculation at the first critical section only. Due to the variation of resistances for shear along the length of the prestressed concrete I-beam, it is not certain what location will govern. Therefore, a systematic evaluation of the shear and the longitudinal yield criteria based on shear-moment interation should be performed along the length of the beam.

Simplified Procedure for Prestressed and Nonprestressed Sections, **LRFD [5.8.3.4.3]**

$$b_v := t_w$$ $\boxed{b_v = 6.50}$ in

The critical section for shear is taken at a distance of d_v from the face of the support, **LRFD [5.7.3.2]**.

d_v = effective shear depth taken as the distance between the resultants of the tensile and compressive forces due to flexure. It need not be taken less than the greater of 0.9*d_e or 0.72h (inches). **LRFD [5.7.2.8]**

The first estimate of d_v is calculated as follows:

$$d_v := -e_s + y_t + H_{avg} + t_{se} - \frac{a}{2}$$ $\boxed{d_v = 72.50}$ in

However, since there are draped strands for a distance of $HD := 49$ from the end of the girder, a revised value of e_s should be calculated based on the estimated location of the critical section. Since the draped strands will raise the center of gravity of the strand group near the girder end, try a smaller value of "d_v" and recalculate "e_s" and "a".

Try $d_v := 65$ inches.

For the standard bearing pad of width, $w_{brg} := 8$ inches, the distance from the end of the girder to the critical section:

$$L_{crit} := \left(\frac{w_{brg}}{2} + d_v\right) \cdot \frac{1}{12} + 0.5 \qquad \boxed{L_{crit} = 6.25} \text{ ft}$$

Calculate the eccentricity of the strand group at the critical section.

$$slope = 10.274$$

$$y_{8t} := A + y_b$$

$$y_{8t} = 32.130$$

$ns_{sb} := 38$ number of undraped strands

$ns_d := 8$ number of draped strands

Find the center of gravity for the 38 straight strands from the bottom of the girder:

$$Y_S := \frac{12 \cdot 2 + 12 \cdot 4 + 12 \cdot 6 + 2 \cdot 8}{ns_{sb}} \qquad \boxed{Y_S = 4.211} \text{ in}$$

$$y_s := y_b + Y_S \qquad y_s = -30.659 \text{ in}$$

$$y_{8t\_crit} := y_{8t} - \frac{slope}{100} \cdot L_{crit} \cdot 12 \qquad \boxed{y_{8t\_crit} = 24.42} \text{ in}$$

$$e_{s\_crit} := \frac{ns_{sb} \cdot y_s + ns_d \cdot y_{8t\_crit}}{ns_{sb} + ns_d} \qquad \boxed{e_{s\_crit} = -21.08} \text{ in}$$

Calculation of compression stress block based on revised eccentricity:

$$d_{p\_crit} := y_t + H_{avg} + t_{se} - e_{s\_crit} \qquad \boxed{d_{p\_crit} = 67.71} \text{ in}$$

Note that the area of steel is based on the number of bonded strands.

$$A_{ps\_crit} := (ns) \cdot A_s \qquad \boxed{A_{ps\_crit} = 9.98} \text{ in}^2$$

Also, the value of f_{pu}, should be revised if the critical section is located less than the development length from the end of the beam. The development length for a prestressing strand is calculated in accordance with **LRFD [5.9.4.3.2]**:

$K := 1.6$ for prestressed members with a depth greater than 24 inches

$\boxed{d_b = 0.600}$ in

$l_d := K \cdot \left(f_{ps} - \frac{2}{3} \cdot f_{pe}\right) \cdot d_b$ $\boxed{l_d = 146.4}$ in

The transfer length may be taken as: $l_{tr} := 60 \cdot d_b$ $\boxed{l_{tr} = 36.00}$ in

Since $L_{crit} = 6.250$ feet is between the transfer length and the development length, the design stress in the prestressing strand is calculated as follows:

$f_{pu\_crit} := f_{pe} + \frac{L_{crit} \cdot 12 - l_{tr}}{l_d - l_{tr}} \cdot \left(f_{ps} - f_{pe}\right)$ $\boxed{f_{pu\_crit} = 195}$ ksi

For rectangular section behavior:

$c := \frac{A_{ps\_crit} \cdot f_{pu\_crit}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps\_crit} \cdot \frac{f_{pu\_crit}}{d_{p\_crit}}}$ $\boxed{c = 7.267}$ in

$a_{crit} := \beta_1 \cdot c$ $\boxed{a_{crit} = 6.177}$ in

Calculation of shear depth based on refined calculations of e_s and a:

$d_{v\_crit} := -e_{s\_crit} + y_t + H_{avg} + t_{se} - \frac{a_{crit}}{2}$ $\boxed{d_{v\_crit} = 64.62}$ in

This value matches the assumed value of d_v above. OK!

The nominal shear resistance of the section is calculated as follows, **LRFD [5.7.3.3]**:

$$V_n = \min\left(V_c + V_s + V_p, 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p\right)$$

where $V_p := 0$ in the calculation of V_n, if the simplified procedure is used (**LRFD [5.8.3.4.3]**).

V_d = shear force at section due to unfactored dead load and includes both DC and DW (kips)

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max} (kips). (Not necessarily equal to V_u.)

M_{cre} = moment causing flexural cracking at section due to externally applied loads (kip-in)

M_{max} = maximum factored moment at section due to externally applied loads (kip-in)

M_{dnc} = total unfactored dead load moment acting on the noncomposite section (kip-ft)

Values for the following moments and shears are at the critical section, $L_{crit} = 6.25$ feet from the end of the girder at the abutment.

$\boxed{V_{DCnc} = 121.7}$ kips

$\boxed{V_{DCc} = 8.7}$ kips

$\boxed{V_{DWc} = 9.0}$ kips

$Vi_{LL} := Vi_{LL\_lane} \cdot g_{vi}$ $\boxed{Vi_{LL} = 100.5}$ kips

$V_i := 1.75 \cdot Vi_{LL}$ $\boxed{V_i = 175.9}$ kips

$V_d := V_{DCc} + V_{DCnc} + V_{DWc}$ $\boxed{V_d = 139.3}$ kips

$V_u := 1.25 \cdot \left(V_{DCnc} + V_{DCc}\right) + 1.5 \cdot V_{DWc} + 1.75 \cdot Vi_{LL}$ $\boxed{V_u = 352.2}$ kips

$M_{dnc} := 730$ kip-ft

$M_{max} := 837$ kip-ft

However, the equations below require the value of M_{max} to be in kip-in:

$\boxed{M_{max} = 10044}$ kip-in

$f_r = -0.20 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := -0.20 \cdot \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = -0.566}$ ksi

$T_{crit} := A_{ps\_crit} \cdot f_{pe}$ $\boxed{T_{crit} = 1599}$ kips

$f_{cpe} := \frac{T_{crit}}{A_g} + \frac{T_{crit} \cdot e_{s\_crit}}{S_b}$ $\boxed{f_{cpe} = 3.539}$ ksi

$\boxed{M_{dnc} = 730}$ kip-ft

$\boxed{M_{max} = 10044}$ kip-in

$S_c := S_{cgb}$ $\boxed{S_c = -24650}$ in^3

$S_{nc} := S_b$ $\boxed{S_{nc} = -18825}$ in^3

$M_{cre} := S_c \cdot \left(f_r - f_{cpe} - \frac{12M_{dnc}}{S_{nc}}\right)$ $\boxed{M_{cre} = 89699}$ kip-in

Calculate V_{ci}, **LRFD [5.8.3.4.3]** λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$V_{ci1} := 0.06 \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v$ $\boxed{V_{ci1} = 71.7}$ kips

$V_{ci2} := 0.02 \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v + V_d + \frac{V_i \cdot M_{cre}}{M_{max}}$ $\boxed{V_{ci2} = 1733.9}$ kips

$V_{ci} := \max(V_{ci1}, V_{ci2})$ $\boxed{V_{ci} = 1733.9}$ kips

$f_t := \frac{T_{crit}}{A_g} + \frac{T_{crit} \cdot e_{s\_crit}}{S_t} + \frac{M_{dnc} \cdot 12}{S_t}$ $\boxed{f_t = 0.337}$ ksi

$f_b := \frac{T_{crit}}{A_g} + \frac{T_{crit} \cdot e_{s\_crit}}{S_b} + \frac{M_{dnc} \cdot 12}{S_b}$ $\boxed{f_b = 3.073}$ ksi

$\boxed{y_{cgb} = -48.78}$ in

$\boxed{ht = 72.00}$ in

$f_{pc} := f_b - y_{cgb} \cdot \frac{f_t - f_b}{ht}$ $\boxed{f_{pc} = 1.219}$ ksi

$V_{p\_cw} := ns_d \cdot A_s \cdot f_{pe} \cdot \frac{slope}{100}$ $\boxed{V_{p\_cw} = 28.6}$ kips

Calculate V_{cw}, **LRFD [5.8.3.4.3]** λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$V_{cw} := \left(0.06 \cdot \lambda \sqrt{f'_c} + 0.30 \cdot f_{pc}\right) \cdot b_v \cdot d_v + V_{p\_cw}$ $\boxed{V_{cw} = 254.8}$ kips

$V_c := \min(V_{ci}, V_{cw})$ $\boxed{V_c = 254.8}$ kips

Calculate the shear resistance at L_{crit}:

$\phi_v := 0.9$ **LRFD [5.5.4.2]**

$s := 20$ in

$A_v := 0.40$ in² for #4 rebar

$f_y := 60$ ksi

$d_v = 65.00$ in

$$\cot\theta := \begin{cases} 1 & \text{if } V_{ci} < V_{cw} \\ \min\left(1.0 + 3\cdot\dfrac{f_{pc}}{\sqrt{f'_c}}, 1.8\right) & \text{otherwise} \end{cases}$$

$\cot\theta = 1.800$

$$V_s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s}$$

LRFD Eq 5.7.3.3-4 reduced per **C5.7.3.3-1** when α = 90 degrees.

$V_s = 140$ kips

$Vn_1 := V_c + V_s + V_p$ $Vn_1 = 395$ kips

$Vn_2 := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$ $Vn_2 = 845$ kips

$V_n := \min(Vn_1, Vn_2)$ $V_n = 395$ kips

$V_r := \phi_v \cdot V_n$ $V_r = 355.69$ kips

E45-2.8 Longitudinal Tension Flange Capacity:

The total capacity of the tension reinforcing must meet the requirements of **LRFD [5.7.3.5].** The capacity is checked at the critical section for shear:

$$V_u := 1.25 \cdot (V_{DC1} + V_{DC2}) + 1.50 \cdot (V_{DW}) + 1.75 \cdot (Vu_{LL})$$

$V_u = 367.320$ kips

$$T_{ps} := \frac{M_{max}}{d_v \cdot \phi_f} + \left(\frac{V_u}{\phi_v} - 0.5 \cdot V_s - V_{p\_cw}\right) \cdot \cot\theta$$

$T_{ps} = 711$ kips

actual capacity of the straight bonded strands:

$\boxed{ns_{sb} \cdot A_s \cdot f_{pu\_crit} = 1610}$ kips

Is the capacity of the straight bonded strands greater than T_{ps}? $\boxed{check = \text{"OK"}}$

Check the tension Capacity at the edge of the bearing:

The strand is anchored $l_{px} := 10$ inches. The transfer and development lengths for a prestressing strand are calculated in accordance with **LRFD [5.9.4.3.2]**:

$\boxed{l_{tr} = 36.00}$ in

$\boxed{l_d = 146.4}$ in

Since l_{px} is less than the transfer length, the design stress in the prestressing strand is calculated as follows:

The assumed crack plane crosses the centroid of the straight strands at

$l_{px'} := l_{px} + Y_S \cdot \cot\theta$ $\boxed{Y_S = 4.211}$ in $\boxed{l_{px'} = 17.58}$ in

$f_{pb} := \dfrac{f_{pe} \cdot l_{px'}}{60 \cdot d_b}$ $\boxed{f_{pb} = 78.23}$ kips

Tendon capacity of the straight bonded strands: $\boxed{ns_{sb} \cdot A_s \cdot f_{pb} = 645}$ kips

The values of V_u, V_s, V_p and θ may be taken at the location of the critical section.

Over the length d_v, the average spacing of the stirrups is:

$s_{ave} := \dfrac{6 \cdot 4.5 + 3 \cdot s}{9}$ $\boxed{s_{ave} = 9.67}$ in

$V_s := A_v \cdot f_y \cdot d_v \cdot \dfrac{\cot\theta}{s_{ave}}$ $\boxed{V_s = 290}$ kips

The vertical component of the draped strands is: $\boxed{V_{p\_cw} = 29}$ kips

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

$$RF_{shear\_Inv} := \frac{(1)(1)(0.9)(V_n) - \gamma_{DC} \cdot (V_{DCnc} + V_{DCc})}{\gamma_{L\_inv} \cdot (Vi_{LL})}$$

$$\boxed{RF_{shear\_Inv} = 1.096}$$

Operating Level

$$RF_{shear\_Op} := \frac{(1)(1)(0.9)(V_n) - \gamma_{DC}\cdot(V_{DCnc} + V_{DCc})}{\gamma_{L\_op}\cdot(Vi_{LL})}$$

$$\boxed{RF_{shear\_Op} = 1.421}$$

At the Service III Limit State (Inventory Level):

$$RF = \frac{f_R - \gamma_D\cdot(f_D)}{\gamma_{servLL}(f_{LLIM})}$$

$T := ns\cdot A_s\cdot f_{pe}$ $\boxed{T = 1599}$ kips

$f_{pb} := \frac{T}{A_g} + \frac{T\cdot(e_s)}{S_b}$ $\boxed{f_{pb} = 4.341}$ ksi

Allowable Tensile Stress **LRFD [5.9.2.3.2b]**

$t_{all} = -0.19\cdot\lambda\sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$t_{all} := -0.19\cdot\sqrt{f'_c}$; $|t_{all}| \leq 0.6$ ksi $\boxed{t_{all} = -0.537}$ ksi

$f_R := f_{pb} - t_{all}$ $\boxed{f_R = 4.878}$ 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

$RF_{serviceIII} := \frac{f_R - 1.0\cdot(f_{DL})}{\gamma_{servLL}\cdot(f_{LLIM})}$ $\boxed{RF_{serviceIII} = 1.369}$

Calculate the composite girder section properties:

effective slab thickness; $t_{se} = 7.50$ in

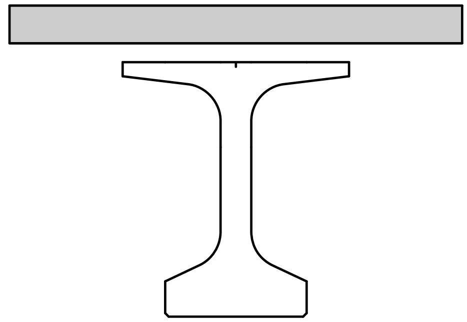
effective slab width; $w_{eadj} = 58.46$ in

haunch thickness; $h = 2.0$ in

total height; $h_c := ht + h + t_{se}$

$h_c = 63.50$ in

$n = 1.540$



Note: The area of the concrete haunch is not included in the calculation of the composite section properties.

Component	Ycg	A	AY	AY^2	I	I+AY^2
Deck	59.75	438	26197	1565294	2055	1567349
Girder	26.3	798	20987	551969	321049	873018
Haunch	55	0	0	0	0	0
Summation		1236	47185			2440367

$\Sigma A := 1236$ in^2

$\Sigma AY := 47185$ in^4

$\Sigma IplusAYsq := 2440367$ in^4

$$y_{cgb} := \frac{-\Sigma AY}{\Sigma A} \qquad y_{cgb} = -38.2 \text{ in}$$

$$y_{cgt} := ht + y_{cgb} \qquad y_{cgt} = 15.8 \text{ in}$$

$$A_{cg} := \Sigma A \quad \text{in}^2$$

$$I_{cg} := \Sigma IplusAYsq - A_{cg} \cdot y_{cgb}^{\,2} \qquad I_{cg} = 639053 \text{ in}^4$$

Deck:

$$S_c := n \cdot \frac{I_{cg}}{y_{cgt} + h + t_{se}} \qquad S_c = 38851 \text{ in}^4$$

E45-3.11 Flexural Strength Capacity at Pier

All of the continuity reinforcement is placed in the top mat. Therefore the effective depth of the section at the pier is:

$cover := 2.5$ in

$bar_{trans} := 5$ (transverse bar size)

$Bar_D(bar_{trans}) = 0.625$ in (transverse bar diameter)

$Bar_{No} = 10$

$Bar_D(Bar_{No}) = 1.27$ in (Assumed bar size)

$$d_e := ht + h + t_s - cover - Bar_D(bar_{trans}) - \frac{Bar_D(Bar_{No})}{2} \qquad \boxed{d_e = 60.24} \text{ in}$$

For flexure in non-prestressed concrete, $\phi_f := 0.9$.

The width of the bottom flange of the girder, $b_w = 30.00$ inches.

The continuity reinforcement is distributed over the effective flange width calculated earlier, $w_e = 90.00$ inches.

From E19-2, use a longitudinal bar spacing of #4 bars at $s_{longit} := 8.5$ inches. The continuity reinforcement is placed at 1/2 of this bar spacing, .

#10 bars at 4.25 inch spacing provides an $\boxed{As_{prov} = 3.57}$ in²/ft, or the total area of steel provided:

$$As := As_{prov} \cdot \frac{w_e}{12} \qquad \boxed{As = 26.80} \text{ in}^2$$

Calculate the capacity of the section in flexure at the pier:

Check the depth of the compression block:

$\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi) **LRFD [5.6.2.2]**

$$a := \frac{As \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c} \qquad \boxed{a = 7.883} \text{ in}$$

This is approximately equal to the thickness of the bottom flange height of 7.5 inches.

$$M_n := As \cdot f_y \cdot \left(d_e - \frac{a}{2}\right) \cdot \frac{1}{12} \qquad \boxed{M_n = 7544} \text{ kip-ft}$$

$$M_r := \phi_f \cdot M_n \qquad \boxed{M_r = 6790} \text{ kip-ft}$$

E45-4.2 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.

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 [C5.4.2.4]**

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

w_c = Unit weight of concrete (kcf)

f'_c = Specified compressive strength of concrete (ksi)

$w_c = 0.15$ kcf **LRFD [Table 3.5.1-1 & C3.5.1]**

$f'_c = 4.00$ ksi

$K_1 := 1.0$ **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 .

For interior beams, the effective flange width is calculated as per **LRFD [4.6.2.6]**:

1. 12.0 times the average thickness of the slab, plus the greater of web thickness or one-half the width of the top flange of the girder:

$$b_{eff2} := \frac{12 \cdot t_s + \frac{14}{2}}{12}$$

This is no longer a valid criteria, however it has been left in place to avoid changing the entire example at this time.

$\boxed{b_{eff2} = 9.08}$ ft

2. The average spacing of adjacent beams:

$$b_{eff3} := S$$

$\boxed{b_{eff3} = 9.75}$ ft

Therefore, the effective flange width is:

$$b_{effflange} := \min\left(b_{eff2}, b_{eff3}\right)$$

$\boxed{b_{effflange} = 9.08}$ ft

or

$\boxed{b_{effflange} \cdot 12 = 109.00}$ in

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 The area of the haunch is conservatively not considered in the section properties for this example.

Based on the plate sizes shown in Figure E45-4.1-4, the noncomposite and composite section properties for the positive moment region are computed as shown in the following table. The distance to the centroid is measured from the bottom of the girder.

The 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]**.

Two sections will be checked for illustrative purposes. First, the ratings will be performed for the location of maximum positive moment, which is at 0.4L in Span 1. Second, the ratings will be performed for the location of maximum negative moment and maximum shear, which is at the pier.

The following are for the location of maximum positive moment, which is at 0.4L in Span 1, as shown in Figure E45-4.4-1.

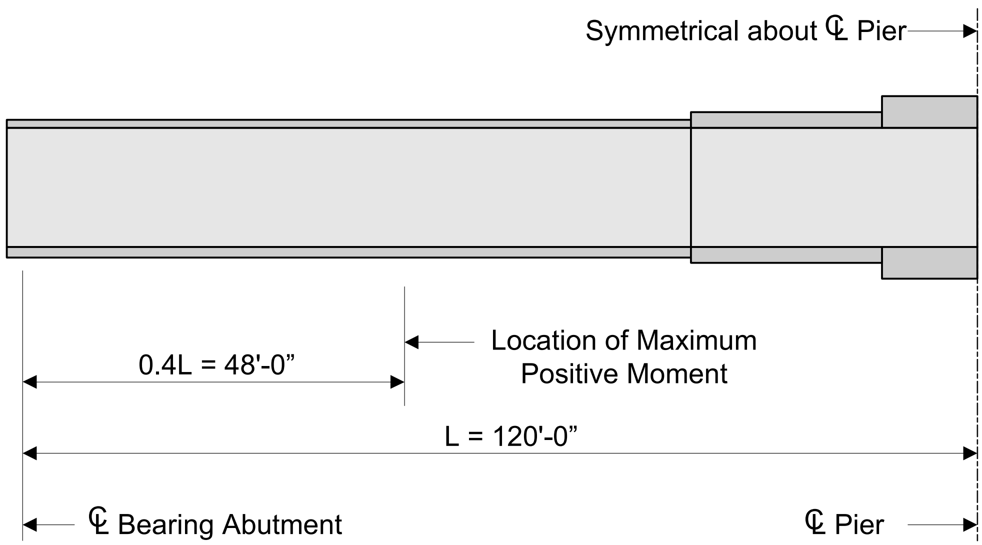


Figure E45-4.4-1
Location of Maximum Positive Moment

E45-4.5 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]**.

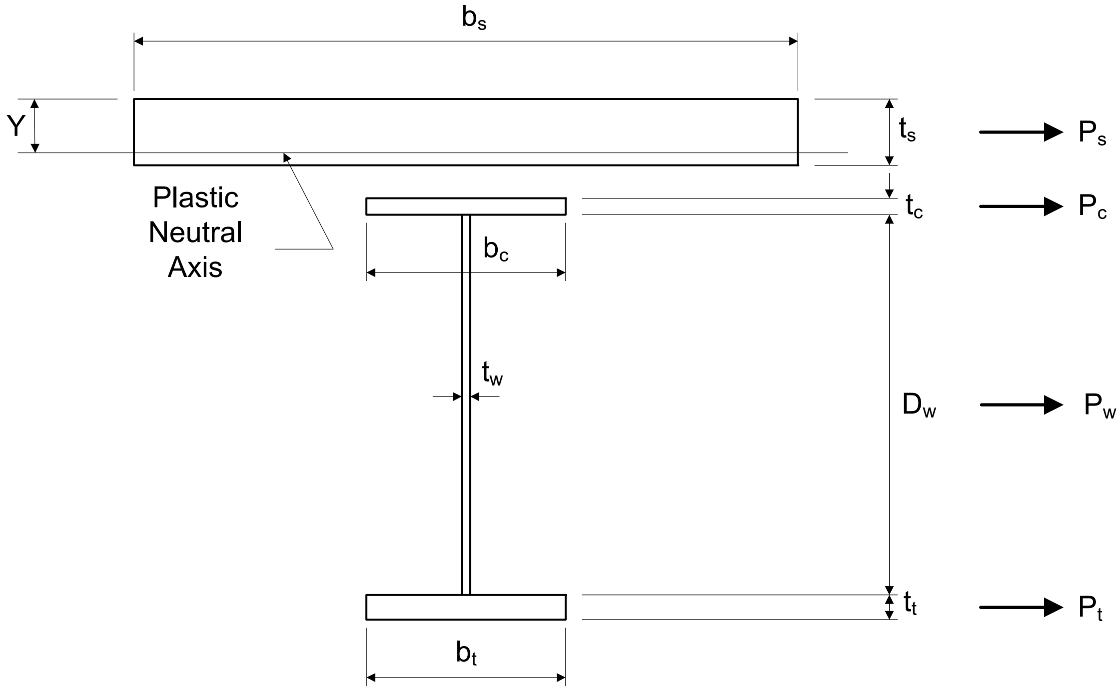


Figure E45-4.5-1
Computation of Plastic Moment Capacity for Positive Bending Sections

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

E45-8.7 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 E45-8.7-1. This is also the location of maximum shear in this case.

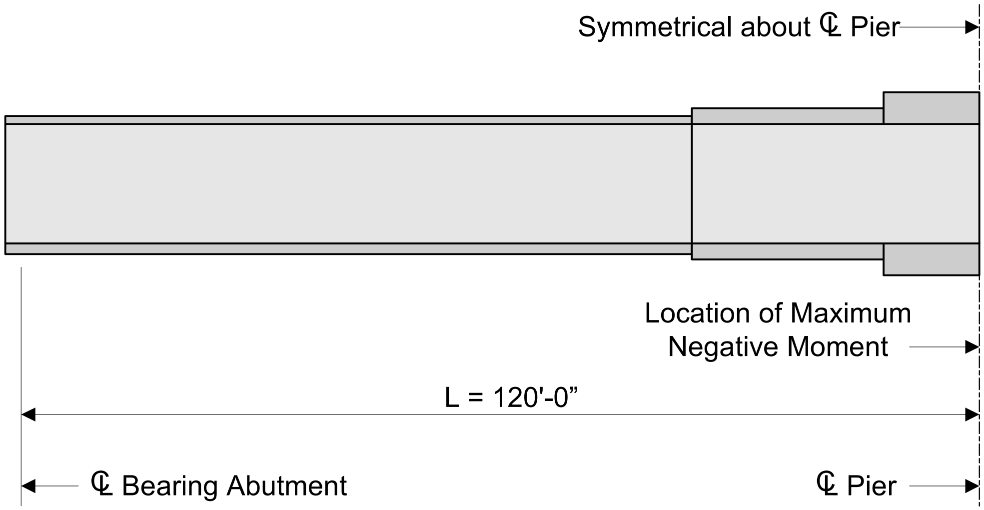


Figure E45-8.7-1
Location of Maximum Negative Moment

For a section to be compact, it must meet the proportion limits with **Std [10.48.1.1]**. For 50 ksi steel, these are as follows:

Compression Flange $\frac{b_f}{2 \cdot t_f} \le 18.4$ **Std [Eq. 10-93]**

$b_f := 14$

$t_f := 2.75$ $\boxed{\frac{b_f}{2 \cdot t_f} = 2.55}$ OK

Web Thickness $\frac{D}{t_w} \le 86$ **Std [Eq. 10-94]**

$D = 54.00$

$t_w = 0.50$ $\boxed{\frac{D}{t_w} = 108.00}$ FAILS

Therefore the section is noncompact at the pier. The requirements of Braced Noncompact Sections per **Std [10.48.2]** will be checked:

Compression Flange $\frac{b_f}{2 \cdot t_f} \le 24$ **Std [Eq. 10-100]**

$\boxed{\frac{b_f}{2 \cdot t_f} = 2.55}$ OK

Web Thickness $\frac{D}{t_w} \le 163$ **Std [Eq. 10-104]**

$\boxed{\frac{D}{t_w} = 108.00}$ OK

Lateral Bracing $L_b \le \frac{20000 \cdot A_f}{F_y \cdot d}$ **Std [Eq. 10-101]**

$A_f := (14)(2.75)$

$d := 54 + 2.75 + 2.5$

$L_b = 240.00$ $\boxed{\frac{20000 \cdot A_f}{F_y \cdot d} = 259.92}$ OK

E45-8.8 Compute Plastic Moment Capacity - Negative Moment Region

The negative moment capacity will be determined from **Std [10.50.2.2]** for noncompact negative moment sections.

Tension Flange $F_{ut} := F_y$

Compression Flange $F_{uc} := F_{cr} \cdot R_b$

$$F_{cr} := \frac{\left(4400 \cdot \frac{2t_f}{b_f}\right)^2}{1000} \le F_y$$

$$\frac{\left(4400 \cdot \frac{2t_f}{b_f}\right)^2}{1000} = 2987.96$$