Table of Contents

2.1 Organizational Charts

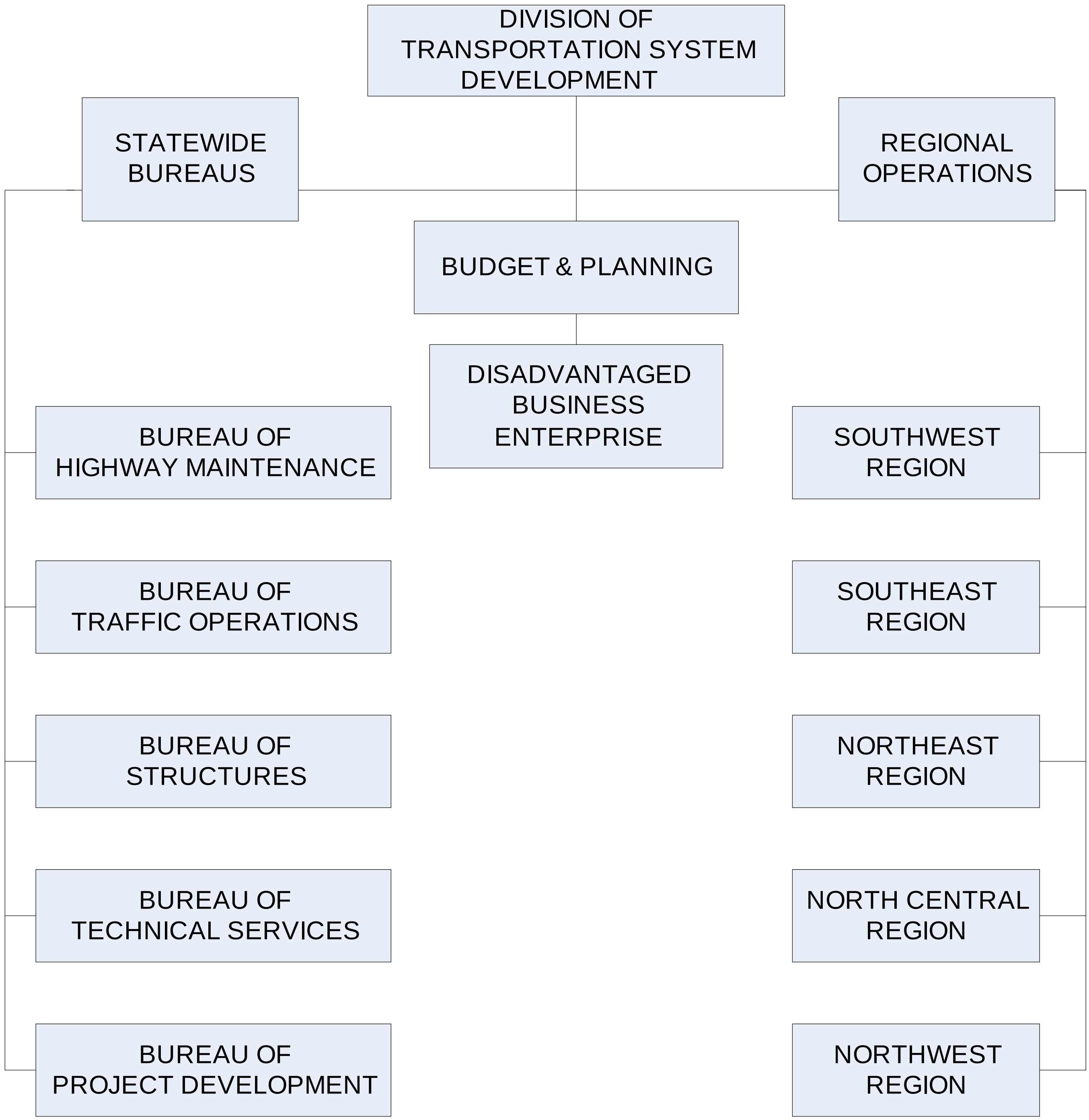


Figure 2.1-1
Division of Transportation System Development

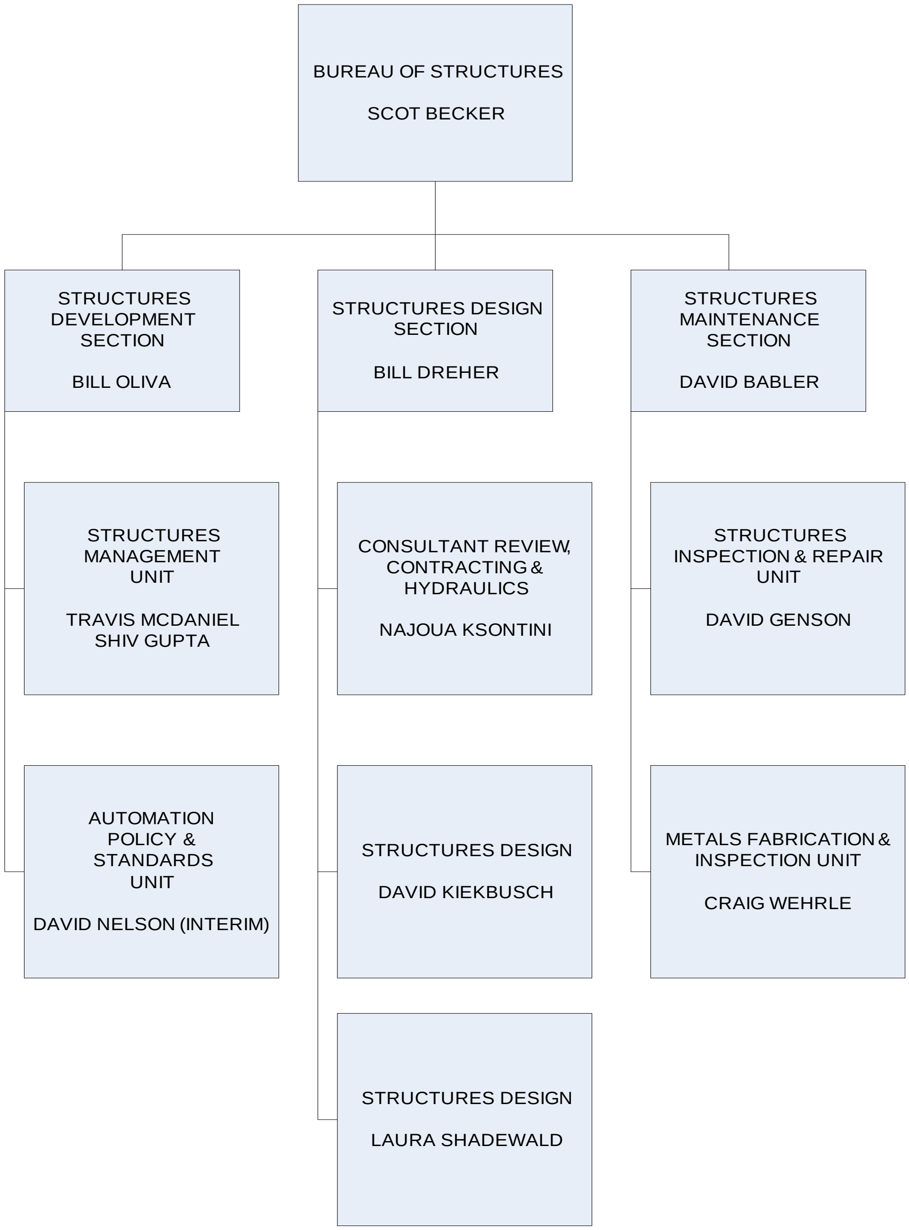
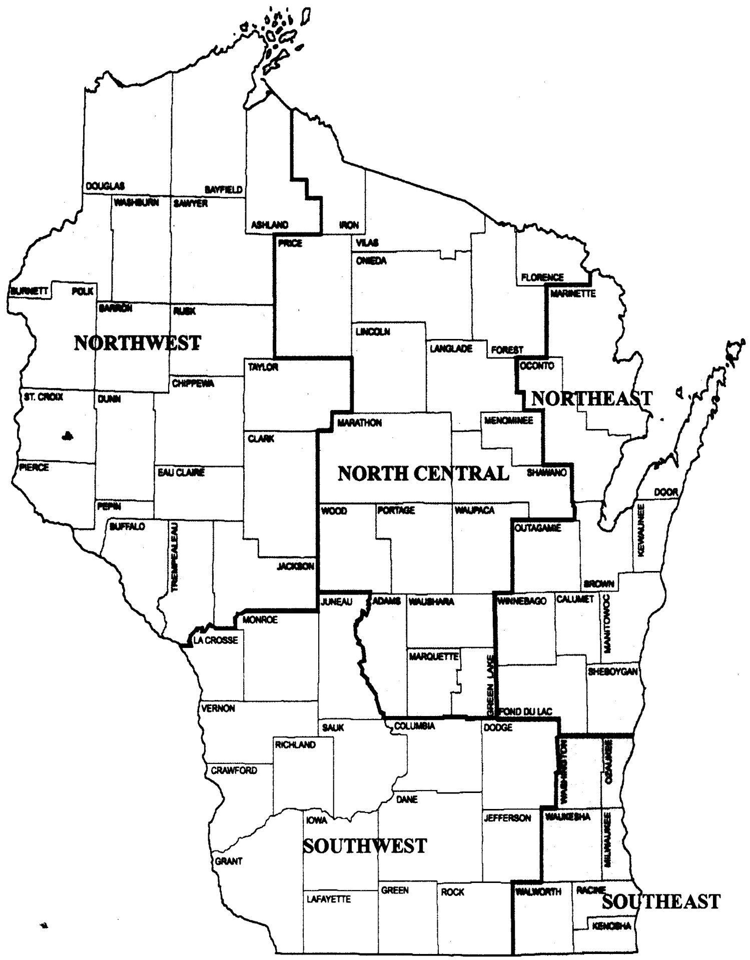


Figure 2.1-2
Bureau of Structures



NO.	COUNTY	REGION	NO.	COUNTY	REGION
1	ADAMS	NORTH CENTRAL	37	MARATHON	NORTH CENTRAL
2	ASHLAND	NORTHWEST	38	MARINETTE	NORTHEAST
3	BARRON	NORTHWEST	39	MARQUETTE	NORTH CENTRAL
4	BAYFIELD	NORTHWEST	40	MILWAUKEE	SOUTHEAST
5	BROWN	NORTHEAST	41	MONROE	SOUTHWEST
6	BUFFALO	NORTHWEST	42	OCONTO	NORTHEAST
7	BURNETT	NORTHWEST	43	ONEIDA	NORTH CENTRAL
8	CALUMET	NORTHEAST	44	OUTAGAMIE	NORTHEAST
9	CHIPPEWA	NORTHWEST	45	OZAUKEE	SOUTHEAST
10	CLARK	NORTHWEST	46	PEPIN	NORTHWEST
11	COLUMBIA	SOUTHWEST	47	PIERCE	NORTHWEST
12	CRAWFORD	SOUTHWEST	48	POLK	NORTHWEST
13	DANE	SOUTHWEST	49	PORTAGE	NORTH CENTRAL
14	DODGE	SOUTHWEST	50	PRICE	NORTH CENTRAL
15	DOOR	NORTHEAST	51	RACINE	SOUTHEAST
16	DOUGLAS	NORTHWEST	52	RICHLAND	SOUTHWEST
17	DUNN	NORTHWEST	53	ROCK	SOUTHWEST
18	EAU CLAIRE	NORTHWEST	54	RUSK	NORTHWEST
19	FLORENCE	NORTH CENTRAL	55	ST CROIX	NORTHWEST
20	FOND DU LAC	NORTHEAST	56	SAUK	SOUTHWEST
21	FOREST	NORTH CENTRAL	57	SAWYER	NORTHWEST
22	GRANT	SOUTHWEST	58	SHAWANO	NORTH CENTRAL
23	GREEN	SOUTHWEST	59	SHEBOYGAN	NORTHEAST
24	GREEN LAKE	NORTH CENTRAL	60	TAYLOR	NORTHWEST
25	IOWA	SOUTHWEST	61	TREMPEALEAU	NORTHWEST
26	IRON	NORTH CENTRAL	62	VERNON	SOUTHWEST
27	JACKSON	NORTHWEST	63	VILAS	NORTH CENTRAL
28	JEFFERSON	SOUTHWEST	64	WALWORTH	SOUTHEAST
29	JUNEAU	SOUTHWEST	65	WASHBURN	NORTHWEST
30	KENOSHA	SOUTHEAST	66	WASHINGTON	SOUTHEAST
31	KEWAUNEE	NORTHEAST	67	WAUKESHA	SOUTHEAST
32	LA CROSSE	SOUTHWEST	68	WAUPACA	NORTH CENTRAL
33	LAFAYETTE	SOUTHWEST	69	WAUSHARA	NORTH CENTRAL
34	LANGLADE	NORTH CENTRAL	70	WINNEBAGO	NORTHEAST
35	LINCOLN	NORTH CENTRAL	71	WOOD	NORTH CENTRAL
36	MANITOWOC	NORTHEAST	73	MENOMINEE	NORTH CENTRAL

Figure 2.1-3
Region Map

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

5. Observe all safety rules at bridge site.

6. Continue to communicate with all Bureau staff.

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

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

2.2.4 Public Communication Record

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

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

http://dotnet/forms/formsNUmMain.htm

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

2.3 Responsibilities of Bureau of Structures

2.3.1 Structures Design Section

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

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

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

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

- Review and approve Consultant rehabilitation proposals.

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

- Provide procedures for scour analysis of structures.

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

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

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

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

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

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

Table of Contents

4.1 Design Excellence

To achieve design excellence, one should be:

1. Creative and aesthetic
2. Analytical
3. Technical and practical

Science, technology, art and economy are the prerequisites for design excellence. Where one or more of these aspects is flawed or not fully considered during design, the final design will probably be flawed, in some cases with distressing results.

The Tacoma Narrows Bridge is an example of technology and art without sufficient science on wind dynamics, which caused the bridge to collapse.

A Rib-Arch Bridge in Ohio was built with science and art but without technology. Numerous deck joints, poor concrete and deicing salts deteriorated the spandrel columns beyond repair, thus causing the bridge to be demolished.

There are many examples of bridges built with science and technology but without art. Just look around.

4.3 Levels of Aesthetics

The Regional Office should establish one of the following levels of aesthetics and indicate it on the Structure Survey Report. This will help the structures designer decide what level of effort and possible types of aesthetics treatments to consider. If Level 2 or greater is indicated, the Regional Office personnel needs to suggest any particular requirements such as railing type, pier shape, special form liners, color, etc. in the comments area of the Structure Survey Report. Specify on the Structure Survey Report whether anti-graffiti coating is required. Areas normally requiring the coating are readily accessible to people with moderate to low levels of activity. High traffic areas or areas with continuous activity would not be as vulnerable to graffiti.

The preliminary plan should incorporate all the agreed upon aesthetic treatments so that final design can proceed efficiently. These details would be developed mutually between the preliminary bridge designer and the Regional Office.

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

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

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

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

4.4 Accent Lighting for Significant Bridges

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

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

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

Table 4.4-1
Accent Lighting for Significant Bridges

4.5 References

1. *"Bridge Aesthetics Around the World"*, Transportation Research Board, Washington, D.C., 1991.

2. *"Esthetics in Concrete Bridge Design"*, American Concrete Institute, Detroit, Michigan, 1990.

3. *"Aesthetic Guidelines for Bridge Design"*, Minnesota Dept. of Transportation, 1995.

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Table of Contents

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.

5.3 Contract Unit Bid Prices

Item No.	Bid Item	Unit	Cost
206.6010.S	Temporary Shoring	LS	--
210.0100	Backfill Structure	CY	22.49
303.0115	Pit Run	CY	9.63
311.0115	Breaker Run	CY	23.50
502.0100	Concrete Masonry Bridges	CY	460.00
502.1100	Concrete Masonry Seal	CY	153.00
502.2000	Compression Joint Sealer Preformed Elastomeric (width)	LF	42.12
502.3100	Expansion Device (structure) (LS)	LF	158.73
502.3110.S	Expansion Device Modular (structure) (LS)	LF	
502.3200	Protective Surface Treatment	SY	2.35
502.6500	Protective Coating Clear	GAL	65.00
503.0128	Prestressed Girder Type I 28-Inch	LF	97.82
503.0136	Prestressed Girder Type I 36-Inch	LF	148.35
503.0137	Prestressed Girders Type I 36W-Inch	LF	156.50
503.0145	Prestressed Girder Type I 45-Inch	LF	162.29
503.0146	Prestressed Girders Type I 45W-Inch	LF	180.85
503.0154	Prestressed Girder Type I 54-Inch	LF	
503.0155	Prestressed Girder Type I 54W-Inch	LF	178.03
503.0170	Prestressed Girder Type I 70-Inch	LF	
503.0172	Prestressed Girders Type I 72W-Inch	LF	183.14
503.0182	Prestressed Girder Type I 82W-Inch	LF	
504.0100	Concrete Masonry Culverts	CY	439.50
504.0500	Concrete Masonry Retaining Walls	CY	461.74
505.0405	Bar Steel Reinforcement HS Bridges	LB	0.86
505.0410	Bar Steel Reinforcement HS Culverts	LB	0.56
505.0415	Bar Steel Reinforcement HS Retaining Walls	LB	0.92
505.0605	Bar Steel Reinforcement HS Coated Bridges	LB	0.91
505.0610	Bar Steel Reinforcement HS Coated Culverts	LB	1.38
505.0615	Bar Steel Reinforcement HS Coated Retaining Walls	LB	0.96
506.0105	Structural Carbon Steel	LB	4.57
506.0605	Structural Steel HS	LB	1.48
506.2605	Bearing Pads Elastomeric Non-Laminated	EACH	63.22
506.2610	Bearing Pads Elastomeric Laminated	EACH	729.90
506.3005	Welded Shear Stud Connectors 7/8 x 4-Inch	EACH	3.50
506.3010	Welded Shear Stud Connectors 7/8 x 5-Inch	EACH	4.83
506.3015	Welded Shear Stud Connectors 7/8 x 6-Inch	EACH	3.54
506.3020	Welded Shear Stud Connectors 7/8 x 7-Inch	EACH	5.18
506.3025	Welded Shear Stud Connectors 7/8 x 8-Inch	EACH	3.82
506.4000	Steel Diaphragms (structure)	EACH	500.20
506.5000	Bearing Assemblies Fixed (structure)	EACH	958.92
506.6000	Bearing Assemblies Expansion (structure)	EACH	1,526.50
507.0200	Treated Lumber and Timber	MBM	
508.1600	Piling Treated Timber Delivered	LF	
510.2005	Preboring Cast-in-Place Concrete Piling	LF	27.36
510.3021	Piling CIP Concrete Delivered and Driven 10 ¾ x 0.219-Inch	LF	32.44

510.3030	Piling CIP Concrete Delivered and Driven 10 ¾ x 0.25-Inch	LF	33.20
510.3040	Piling CIP Concrete Delivered and Driven 10 ¾ x 0.365-Inch	LF	36.52
510.3050	Piling CIP Concrete Delivered and Driven 10 ¾ x 0.5-Inch	LF	--
510.3023	Piling CIP Concrete Delivered and Driven 12 ¾ x 0.219-Inch	LF	
510.3033	Piling CIP Concrete Delivered and Driven 12 ¾ x 0.25-Inch	LF	34.38
510.3043	Piling CIP Concrete Delivered and Driven 12 ¾ x 0.375-Inch	LF	39.01
510.3053	Piling CIP Concrete Delivered and Driven 12 ¾ x 0.5-Inch	LF	
510.3024	Piling CIP Concrete Delivered and Driven 14 x 0.219-Inch	LF	--
510.3034	Piling CIP Concrete Delivered and Driven 14 x 0.25-Inch	LF	
510.3044	Piling CIP Concrete Delivered and Driven 14 x 0.375-Inch	LF	
510.3054	Piling CIP Concrete Delivered and Driven 14 x 0.5-Inch	LF	
510.3026	Piling CIP Concrete Delivered and Driven 16 x 0.219-Inch	LF	--
510.3036	Piling CIP Concrete Delivered and Driven 16 x 0.25-Inch	LF	--
510.3046	Piling CIP Concrete Delivered and Driven 16 x 0.375-Inch	LF	--
510.3056	Piling CIP Concrete Delivered and Driven 16 x 0.5-Inch	LF	
511.2105	Piling Steel Delivered and Driven HP 10-Inch x 42 LB	LF	31.48
511.2110	Piling Steel Delivered and Driven HP 12-Inch x 53 LB	LF	40.18
511.2115	Piling Steel Delivered and Driven HP 12-Inch x 74 LB	LF	45.54
511.2120	Piling Steel Delivered and Driven HP 14-Inch x 73 LB	LF	
511.3000	Pile Points	EACH	80.11
511.6000	Piling Steel Preboring	LF	156.88
512.1000	Piling Steel Sheet Temporary	SF	15.00
513.4050	Railing Tubular Type F (structure) (LS)	LF	108.63
513.4052 or 3	Railing Tubular Type F- (4 or 5) Modified (structure) (LS)	LF	140.94
513.4055	Railing Tubular Type H (structure) (LS)	LF	110.96
513.4060	Railing Tubular Type M (structure) (LS)	LF	169.63
513.4065	Railing Tubular Type PF (structure) (LS)	LF	--
513.4090	Railing Tubular Screening Structure B-	LF	104.77
513.7050	Railing Type W Structure B-	LF	123.26
00000	Concrete Railing, “Texas Rail”	LF	160.00
00000	Concrete Parapet, Type ‘LF’ & ‘A’ (estimate)	LF	80.00
513.7005	Railing Steel Type C1 (structure) (LS)	LF	71.39
513.7010	Railing Steel Type C2 (structure) (LS)	LF	180.25
513.7015	Railing Steel Type C3 (structure) (LS)	LF	118.48
513.7020	Railing Steel Type C4 (structure) (LS)	LF	38.19
513.7025	Railing Steel Type C5 (structure) (LS)	LF	
513.7030	Railing Steel Type C6 (structure) (LS)	LF	114.03
514.0445	Floor Drains Type GC	EACH	1,598.00
514.2625	Downspouts 6-Inch	LF	--

Table of Contents

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. (Bur. of Structures)	Record Structure Survey Report.
Structures Design Section (Bur. of Structures)	Determine type of structure.
	Perform hydraulic analysis if required.
	Check roadway geometrics and vertical clearance.
	Review Site Investigation Report and determine foundation requirements. Check criteria for scour critical Bridges and record scour critical code on the preliminary plans.
	Draft preliminary plan layout of structure.
	Send copies of preliminary plans to Regional Office.
	If a railroad is involved, send copies of preliminary plans to the Rails & Harbors Section (Bureau of Transit, Local Roads, Rails & Harbors) who will forward details and information to the railroad company.
	If Federal aid funding is involved, send copies of preliminary plans to the Federal Highway Administration for major, moveable, and unusual bridges.
	If a navigable waterway 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.
	Assign project to a Structures Design Unit.
Structures Design Units (Bur. of Structures)	Prior to starting project, Designer contacts Regional Office to verify preliminary structure geometry, alignment, width and the presence of utilities.
	Prepare and complete design and final plans for the specified structure.
	Give completed job to Manager of Structures Design Section.
Manager, Structures Design Section (Bur. of Structures)	Review final structural plans.
	Review and revise or write special provisions as needed.
	Send copies of final structural plans and special provisions to Regional Offices.
	If a railroad is involved, send copies of final plans to the Rails & Harbors Section.
	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.

A map of navigable waterways in Wisconsin as defined by the Coast Guard is kept in the Consultant Design and Hydraulics Unit (Bureau of Structures).

6.2 Preliminary Plans

6.2.1 Structure Survey Report

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

Under the “Plans and Projects” heading:

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

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

When preparing the Structure Survey Report, 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 Geotechnical Section, attention Chief Geotechnical Engineer, through the Regional Soils Engineer, and to the Consultant Design & Hydraulics Unit, attention Consultant Design & Hydraulics Supervisor. This submittal will take place a minimum of 15 months in advance of the final plans due date shown on the Structure Survey Report. The Geotechnical Section has responsibility for conducting the necessary soil borings. The Consultant Design and Hydraulics Unit and the Geotechnical Section will coordinate activities to deliver the completed preliminary plans on schedule.

In most instances, the geotechnical work will proceed after the receipt of the Structure Survey Report, but in advance of the development of the preliminary bridge plans. However, special circumstances may require that the preliminary bridge plans precede the geotechnical work. The Geotechnical Section may request preliminary bridge plans under the following conditions.

1. A review of available subsurface information indicates the probability of very shallow and highly variable bedrock.

2. The span on the Structure Survey Report falls in the 30 to 40 range and the decision between a bridge or a box culvert is not evident.

3. The Structure Survey Report indicates a multiple span structure in excess of 100 feet over a body of water.

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

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

If the preliminary bridge plans are required more than one year in advance of the final plan due date on the Structure Survey Report due to the unique needs of the project, the project manager should discuss this situation with the Consultant Design & Hydraulics Supervisor prior to submitting the Structure Survey Report. A note discussing the agreed upon schedule should then be attached to all copies of the Structure Survey Report so all parties are aware of the schedule. The Geotechnical Section is responsible for scheduling the borings.

Coordination early in the design process with DNR regarding removal techniques for the existing structure, and new structure placement and type is very important. The status of any agreements with the DNR, that 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.

For box culverts a preliminary drawing is usually not prepared. Information required to be submitted as a part of the survey report for a box culvert is usually sufficient to serve as a preliminary layout.

The drawings for preliminary layouts are on sheets having an overall width of 11 inches and an overall length of 17 inches.

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 steel 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 ice and debris. For structures over navigable streams, the vertical and horizontal clearance 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.

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. 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).

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

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

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

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

6. Detail the extent of slope paving or riprap.

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

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

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

10. Median width if dual highway.

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

12. 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.

13. Location of and vertical clearance at point of critical vertical clearance if highway or railroad separation. (For both roadway directions on divided highways).

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

15. 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.

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

17. Structure numbers on plan. North Arrow.

18. Excavation protection for railroads.

19. Location of deck lighting or utilities if any.

20. Name Plate location. Locate the structure name plate on the roadway side of the first right wing traveling in the highway cardinal directions of North or East.

21. Bench Mark Cap Location

22. Locations of surface drains on approach pavement.

23. Tangent Offsets between reference line and tangent line along C_L substructure unit.

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 amount of minimum vertical clearance.

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. Use a scale of 1” = 10’ whenever possible.

6.2.2.3.3 Cross-Section View

The cross-section view need only be a half section if symmetrical about a reference line, otherwise it is a full section taken normal to reference line. Use a scale of (1” = 4’) whenever possible. 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. Steel beam or girder spacing with beam/girder depth.

4. For prestressed girders approximate position of exterior girders.

5. Direction and amount of crown or superelevation.

6. Point referred to on profile grade.

7. Type of pier with size and number of columns proposed.

8. For solid, hammerhead or other type pier approximate size to scale.

9. If length of concrete pier cap between outer pier columns exceeds approximately 60 feet, provide an opening in the cross girder for temperature changes and concrete shrinkage, or design the pier cap for temperature and shrinkage to eliminate the opening.

10. Dimension minimum depth of bottom of footings below ditch or finished ground line or if railroad crossing below top of rail.

11. Location for public and private utilities to be carried in the superstructure. Label owner's name of utilities.

12. 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 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>Ultimate Stresses for Materials</u>:

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

Foundations

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

Ratings

Live Load:

Design Loading: HL-93

Inventory Rating Factor: RF = X.XX

Operating Rating Factor: RF = X.XX

Wisconsin Standard Permit Vehicle (Wis-SPV)

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

Hydraulic Data

Base Flood

- 100 Year Discharge
- Stream Velocity
- 100 Year Highwater Elevation
- Q_2 & Q_2 Elevation (Based on new structure opening)
- Waterway Area
- Drainage Area
- Scour Critical

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

- Overtopping Frequency
- Overtopping Elevation
- Overtopping Discharge

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

6.2.2.4 Utilities

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

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

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

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

6.2.3 Distribution of Exhibits

6.2.3.1 Federal Highway Administration (FHWA).

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

1. Projects on the Interstate System

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

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

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

Unusual bridges have the following characteristics:

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

Examples of unusual bridges:

- Cable-stayed
- Suspension
- Arch
- Segmental concrete
- Movable
- Truss
- Bridges types that deviate from AASHTO bridge design standards or AASHTO guide specifications for highway bridges
- Major bridges using load and resistance factor design specifications
- Bridges using a three-dimensional computer analysis
- Bridges with spans exceeding 350 feet

Examples of unusual structures:

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

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

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

1. Preliminary plans (Type, Size and Location)
2. Bridge/structures related environmental concerns and suggested mitigation measures
3. Studies of bridge types and span arrangements
4. Approach bridge span layout plans and profile sheets
5. Controlling vertical and horizontal clearance requirements

6. Roadway geometry

7. Design specifications used

8. Special design criteria

9. Cost estimates

10. Hydraulic and scour design studies/reports showing scour predictions and related mitigation measures

11. Geotechnical studies/reports

12. Information on substructure and foundation types

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

6.2.3.2 Coast Guard

Current permit application guides published by the 2nd or 9th Coast Guard District should be followed. For Federal Aid projects, applicants must furnish two copies of the Final Environmental Impact Statement accepted by the lead agency. The Regional Office will also forward Water Quality Certification obtained from the Department of Natural Resources.

6.2.3.3 Regions

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

6.2.3.4 Utilities

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

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

6.2.3.5 Other Agencies

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

6.3 Final Plans

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

6.3.1 General Requirements

6.3.1.1 Drawing Size

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

6.3.1.2 Scale

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

6.3.1.3 Line Thickness

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

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

6.3.1.4 Lettering and Dimensions

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

6.3.1.5 Notes

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

6.3.1.6 Standard Insert Drawings

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

Standard insert sheets can be found at:
http://trust.dot.state.wi.us/extntgtwy/dtid_bos/extranet/structures/index.htm

6.3.1.7 Abbreviations

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

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

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

Table 6.3-1
Abbreviations

6.3.1.8 Nomenclature and Definitions

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

6.3.2 Plan Sheets

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

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

1. General Plan

2. Subsurface Exploration

3. Abutments

4. Piers

5. Superstructure and Superstructure Details

6. Railing and Parapet Details

Show all views looking up station.

6.3.2.1 General Plan (Sheet 1)

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

1. Plan View

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

   a. Sufficient dimensions to layout structure in the field.

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

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

   On Structure Replacements

   Show existing structure in dashed-lines on Plan View.

2. Elevation View

   Same requirements as specified for preliminary plan except:

   a. Show elevation at bottom of all substructure units.

   b. Give estimated pile lengths where used.

3. Cross-Section View

   Same requirements as specified for preliminary plan except:

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

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

4. Grade Line

Same requirements as specified for preliminary plan.

5. Design and Traffic Data

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

6. Hydraulic Information, if Applicable

7. Foundations

Give soil/rock bearing capacity or pile capacity.

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

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

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

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

8. Estimated Quantities

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

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

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

9. General Notes

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

10. List of Drawings

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

11. Bench Marks

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

12. Title Block

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

13. Professional Seal

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

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

6.3.2.1.1 Plan notes for New Bridge Construction

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

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

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

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

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

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

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

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

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

10. The finished graded section shall be the upper limits of excavation for structures.

11. The upper limits of excavation for structures for the abutments shall be the bottom of slope protection.

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

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

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

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

6.3.2.1.2 Plan Notes for Bridge Rehabilitation

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

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

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

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

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

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

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

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

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

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

6.3.2.2 Subsurface Exploration

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

1. Plan View

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

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

2. Elevation

   Show a centerline profile of existing ground elevation.

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

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

6.3.2.3 Abutments

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

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

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

1. Plan View

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

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

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

2. Elevation

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

    b. Give vertical dimension of wing.

3. Wing Elevation

4. Body Section

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

5. Wing Sections

6. Bar Steel Listing and Detail

Use the following views where necessary:

7. Pile Plan & Splice Detail

8. View Showing Limits of Excavation and Backfill

9. Special Details for Utilities

10. Drainage Details

6.3.2.4 Piers

Use as many sheets as necessary to show all details clearly. One sheet may show several piers if only the height, elevations and other minor details are different.

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

1. Plan View

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

2. Elevation

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

3. Footing Plan

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

4. Bar Steel Listing and Details

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

6. Cross Section thru Column and Pier Cap

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

6.3.2.5 Superstructure

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

6.3.2.5.1 All Structures

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

2. For girder bridges:
   Show the total dead load deflections, including composite dead load (without future wearing surface) acting on the composite section, at tenth points of each span. Distribute the composite dead load evenly to all girders and provide one deflection value for a typical interior girder. Chapter 17 – Superstructure-General illustrates three load cases for exterior girder design with raised sidewalks, cases that provide a conservative envelope to ensure adequate girder capacity. However, the above composite dead load distribution should be used for deflection purposes. For

prestressed concrete girders, the dead load deflection reported does not include the weight of the girder. See Chapter 24 – Steel Girder Structures for camber and blocking, top of steel elevation and deflection reporting criteria.

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

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

Deflection and camber values are to be reported to the nearest 0.1 inch.

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

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

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

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

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

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

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

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

6.3.2.5.2 Steel Structures

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

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

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

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

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

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

6.3.2.5.3 Railing and Parapet Details

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

6.3.3 Miscellaneous Information

6.3.3.1 Bill of Bars

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

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

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

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

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

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

6.3.3.2 Box Culverts

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

1. Plan View
2. Longitudinal section
3. Section thru box
4. Wing elevations
5. Section thru wings
6. Section thru cutoff wall
7. Vertical construction joint
8. Bar steel clearance details
9. Header details
10. North point, Bench mark, Quantities
11. Bill of bars, Bar details
12. General notes, List of drawings, Rip rap layout
13. Inlet nose detail on multiple cell boxes
14. Corner details

Bid items are excavation, concrete masonry, bar steel and rip rap. Non bid items are membrane waterproofing, filler and expansion bolts. In lieu of showing a contour map, show profile grade lines as described for Subsurface Exploration sheet.

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

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

6.3.3.3 Miscellaneous Structures

Detail plans for other structures such as retaining walls, sign bridges, pedestrian bridges, and erosion control structures are to be detailed with the same requirements as previously mentioned.

6.3.3.4 Standard Drawings

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

6.3.3.5 Insert Sheets

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

6.3.3.6 Change Orders and Maintenance Work

These plans are drawn on full size sheets.

6.3.3.7 Bench Marks

Bench mark caps are shown on all bridges and larger culverts. Locate the caps on a horizontal surface flush with the concrete. Show the location in close proximity to the Name Plate.

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 Chief Structural Design Engineer.

Give special attention to unique details and unusual construction problems. Take nothing for granted on the plans.

The Checkers check the final plans against the Engineer's design and sketches to be sure 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.

Check the final plan Bid Items for conformity with those scheduled in the WisDOT Standard Specifications for Highway and Structure Construction.

The Checker makes an independent Bill of Bars list to be sure the detailer has not omitted any bars when checking 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 people 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 survey folder are separated into the following groups by the Structures Design Unit Supervisor or plans checker:

6.3.4.1 Items to be Destroyed When Construction is Completed (Group A)

1. Miscellaneous correspondence and Transmittal letters
2. Preliminary drawings and computations
3. Prints of soil borings and plan profile sheets
4. Quantity computations and bill of bars
5. Shop steel quantity computations*
6. Design checker's computations
7. Designer Computations and computer runs of non-complex structures on non state maintained structures.
8. Layout sheets
9. Elevation runs and bridge geometrics
10. *Falsework plans*
11. Miscellaneous Test Report
12. Photographs of Bridge Rehabs

* These items are added to the packet during construction.

6.3.4.2 Items to be Destroyed when Plans are Completed (Group B)

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

Items in Group A should be placed together and labeled. Items in Group B should be discarded.

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. Data for filing that is generated outside the Bureau of Structures should be sent to the Structures Development Section.

1. Structure Inventory Form (Available on DOTNET) - New Bridge File – Data for this form is completed by the preliminary designer and plans checker. It is submitted to the Structures Development Section for entry into the File.

2. Load Rating Input File - Permits File - The designers submit an electronic copy of the input data for load rating the structure to the Structures Development Section. It is located for internal use at //H32751/rating.

3. Designer Computations and Inventory Superstructure Design Run (Substructure computer runs as determined by the Engineer) - **HSI – The designers record design, inventory, operating ratings and maximum vehicle weights on the plans and place into the scanned folder.

4. 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 e-mail to “DOTDTSDStructuresPiling@dot.wi.gov”. These two documents will be placed in HSI for each structure and can be found in the “Shop” folder.

5. Shop Drawings for Steel Bridges, Sign Bridges, Prestressed Girders, High Mast Poles, Retaining Walls, Floor Drains, Railings and all Steel Joints - HSI - Metals Fabrication & Inspection Unit or other source sends to the Structures Development Section to scan all data into HSI.

6. Mill Tests, Heat Numbers and Shop Inspection Reports for all Steel Main Members - HSI - Metals Fabrication & Inspection Unit sends electronic files data into HSI.

7. Hydraulic and Scour Computations, Contour Maps and Site Report - HSI - Data is placed into scanned folder by Consultant Design & Hydraulics Unit.

8. Subsurface Exploration Report - HSI - Report is placed into scanned folder by Consultant Design & Hydraulics Unit or electronic copies are loaded from Geotechnical files.

9. Structure Survey Report - HSI - Report is placed into scanned folder by Consultant Design & Hydraulics Unit.

10. As Built Plans - HSI - At bid letting, the printers place a digital image of plans in a computer folder and send to the Structures Development Section where the plan sheets are labeled and placed in HSI. As Built plans will replace bid letting plans when available and will be scanned by the Structures Development Section.

11. Inspection Reports - New Bridge File - The Structures Maintenance Section loads a copy of the following Inspection Reports into the New Bridge File.

Initial	Underwater (UW-Probe/Visual
Routine Visual	Movable
Fracture Critical	Damage
In-Depth	Interim
Underwater (UW)-Dive	Posted
Underwater (UW)-Surv	

Table 6.3-2
Various Inspection Reports

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

6.3.5 Processing Plans

1. Before P.S. & E. Process

   File plans in plan drawers by county for consultant work, or

   Maintain plans as PDF on E-plan server.

2. At P.S. & E. Processing

   Prepare plans for bid letting process.

3. After Structure Construction

   Any data in Design Folder is scanned and placed with bridge plans.

   Original plan sheets and Design Folders are discarded.

6.4 Computation of Quantities

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

Be neat and orderly with the work. 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.

Staged Construction - On projects where there is staged construction that will involve two construction seasons the following quantities should be split to match the staging to aid the contractor/fabricator: Concrete Masonry, Bar Steel Reinforcement, Structural Steel and Bar Couplers. The other items are not significant enough to justify separating.

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. 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.

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

6.4.2 Backfill Granular or Backfill Structure

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

6.4.3 Concrete Masonry Bridges

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

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

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

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

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

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

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

6.4.6 Structural Steel Carbon or Structural Steel HS

See 24.2.4.

6.4.7 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.

6.4.8 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.

6.4.9 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.

6.4.10 Preboring CIP Concrete Piling or Steel Piling

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

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

Record the type, quantity is a Lump Sum.

6.4.12 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.

6.4.13 Riprap Medium, Riprap Heavy or Grouted Riprap, Riprap Light

Record this quantity to the nearest 5 cubic yards.

6.4.14 Pile Points

When recommended in soils report. Bid as each.

6.4.15 Floordrains Type GC or Floordrains Type H

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

6.4.16 Cofferdams (Structure)

Lump Sum

6.4.17 Rubberized Membrane Waterproofing

Record the quantity to the nearest square yard.

6.4.18 Expansion Device (Structure)

Record this quantity in lump sum.

6.4.19 Electrical Work

Refer to Standard Construction Specifications for bid items.

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

Record this quantity in feet.

6.4.21 Preparation Decks Type 1 or Preparation Decks Type 2

Estimate Type 2 Deck Preparation as 40% of Type 1 Deck Preparation. Record this quantity to the nearest square yard. Use 2” for depth of each Preparation, compute concrete quantity and add to Concrete Masonry Overlay Decks.

6.4.22 Cleaning Decks

Record this quantity to the nearest square yard.

6.4.23 Joint Repair

Record this quantity to the nearest square yard.

6.4.24 Concrete Surface Repair

Record this quantity to the nearest square foot.

6.4.25 Full-Depth Deck Repair

Record this quantity to the nearest square yard.

6.4.26 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.

6.4.27 Removing Old Structure STA. XX + XX.XX

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

6.4.28 Anchor Assemblies for Steel Plate Beam Guard

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

6.4.29 Steel Diaphragms (Structure)

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

6.4.30 Welded Stud Shear Connectors X -Inch

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

6.4.31 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.

6.4.32 Geotextile Fabric Type

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

6.4.33 Masonry Anchors Type L No. Bars

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

6.4.34 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.

6.4.35 Piling Steel Sheet Temporary

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

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

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

6.4.36 Temporary Shoring

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

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

6.4.37 Concrete Masonry Deck Patching

(Deck preparation areas) x 2" deck thickness.

6.4.38 Sawing Pavement Deck Preparation Areas

Use 10 lineal feet per S.Y. of Preparation Decks.

6.4.39 Removing Bearings

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

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

The need for structures is determined during the Preliminary Site Survey and recorded in the Concept Definition or Work Study Report. On Federal (FHWA) or State Aid Projects completed Structure Survey Reports and plans are submitted to the Structures Design Section with a copy forwarded to the Regional Office for approval prior to construction. Structure and project numbers are assigned by the Regional Offices. In preparation of the structural plans, the appropriate specifications and details recommended by the Structures Design Section 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 Structures Design Section for review.

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

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

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

6.5.1 Approvals, Distribution, and Work Flow

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

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

Structures Design Section	Review modified final plans as applicable.
	Sign final plans.
Bureau of Project Development	Prepare final approved bridge plans for pre-Development contract administration.

Table 6.5-1

Approvals, Distribution and Work Flow

6.5.2 Consultant Preliminary Plan Requirements

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

1. Structure Survey Report.

2. Preliminary Drawings.

3. Log Borings shown on the Subsurface Exploration Drawing which must be submitted now and can be included with the Final Plans.

4. Evaluation Report of Borings.

5. Contour Map.

6. Typical Section for Roadway Approaches.

7. Plan and Profile of Approach Roadways.

8. Hydraulic Report (see Chapter 8 - Hydraulics) is required for Stream Crossing Structures.

9. County Map showing Location of New and/or Existing Structures.

10. Any other information or Drawings which may influence Location, Layout or Design of Structure.

The above information is also required for Box Culverts except that a separate preliminary drawing is usually not prepared unless the Box Culvert has large wings or other unique features.

The type of structure is usually determined by the local unit of government and the Regional Office. However, Bureau of Structures personnel review the structure type and may recommend that other types be considered. In this regard it is extremely important that preliminary designs be coordinated to avoid delays and unnecessary expense in plan preparation.

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

6.5.3 Final Plan Requirements

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

1. Final Drawings.

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

2. Design and Quantity Computations

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

4. QA/QC Verification Sheet

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

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

6.5.4 Design Aids & Specifications

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

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

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

Structure costs
Structure Special Provisions

http://www.dot.wisconsin.gov/business/engrserv/index.htm

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

http://www.arema.org

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Table of Contents

7.1 Introduction

Disclaimer:
This chapter is in the early stages of development. The information is limited and will develop over time. The intent of this chapter is to provide guidance to designers, but is far from all-inclusive.

The purpose of the Accelerated Bridge Construction (ABC) Chapter is to provide guidance for the planning and implementation of projects that may benefit from the application of rapid bridge construction technologies and methods. This chapter was prepared to provide planners and engineers with a basic understanding of different ABC methods available, help guide project specific selection of ABC methods, and to encourage the use of the ABC methods described in this chapter.

7.1.1 WisDOT ABC Initiative

WisDOT is following the Federal Highway Administration's (FHWA) Every Day Counts initiative "aimed at shortening project delivery, enhancing the safety of our roadways, and protecting the environment." Two of the five major methods that the FHWA has emphasized as accelerating technologies are Prefabricated Bridge Elements and Systems (PBES) and Geosynthetic Reinforced Soil – Integrated Bridge Systems (GRS-IBS). These accelerating technologies are incorporated in the following sections in this chapter, namely: Prefabricated Bridge Elements, Geosynthetic Reinforced Soil – Integrated Bridge Systems (GRS-IBS), Self-Propelled Modular Transporters (SPMTs) and Lateral Sliding (both SPMTs and Lateral Sliding are classified as Prefabricated Bridge Systems). WisDOT has had success using GRS-IBS and Prefabricated Bridge Elements, and is always looking for new technologies to improve construction and reduce impacts to traffic. For more information on the Every Day Counts Initiative, refer to www.fhwa.dot.gov/everydaycounts.

7.1.2 ABC Overview

In essence, ABC uses different methods of project delivery and construction to reduce the project schedule, on-site construction time, and public impact. With the ever increasing demand on transportation infrastructure, and the number of bridges that are approaching the end of their service lives, the need for ABC becomes more apparent.

Three main benefits of using ABC methods include minimized impact to traffic, increased safety during construction, and minimized impacts in environmentally sensitive areas. Where conventional bridge construction takes months or years, a bridge utilizing ABC may be placed in a matter of weeks, days, or even a few hours depending on the methods used. ABC methods are generally safer than conventional construction methods because much of the construction can be done offsite, away from traffic. Quality can also be improved because the construction is often completed in a more controlled environment compared to on-site conditions. On the other hand, as with the implementation of all new technologies, the use of ABC comes with challenges that need to be overcome on a project-specific basis.

Oftentimes accelerating the schedule increases the cost of the project. This increased project delivery cost can be offset by reductions in road user costs. In some states, it has been shown that a high percentage of the public approves the use of ABC knowing that the cost can be significantly higher.

WisDOT policy item:

Prior to the implementation of ABC methods on a project, contact the Bureau of Structures Development Section Chief for discussion, resources, and approval.

7.1.3 Accelerated Bridge Construction Terminology

Acronym/Term	Definition
ABC (Accelerated Bridge Construction)	Bridge construction methods that use innovative planning, design, materials, and construction techniques in a safe and cost-effective manner to reduce the onsite construction time that occurs when building new bridges or replacing and rehabilitating existing bridges.
AC (Alternative Contracting)	Nontraditional project delivery systems, bidding practices, and specifications that may be used to reduce life-cycle costs, improve quality, and accelerate the delivery of construction projects.
BSA (Bridge Staging Area)	Location where a bridge is constructed near the final location for the bridge, where the traveling public is not affected. The bridge can be moved from the staging area to the final location with SPMTs or by sliding.
CM/GC (Construction Manager/General Contractor)	Hybrid of the DBB and D/B processes that allows the owner to remain active in the design process, while the risk is still taken by the general contractor. This method is not an option for WisDOT administered projects.
D/B (Design/Build)	Accelerated project delivery method where one entity (the “designer-builder”) assumes responsibility for both the design and construction of a project. This method is not an option for WisDOT administered projects.
DBB (Design-Bid-Build)	Traditional project delivery method where the owner contracts out the design and construction of a project to two different entities.
EDC (Every Day Counts)	Initiative put forth by FHWA designed to identify and deploy innovation aimed at shortening project delivery, enhancing the safety of our roadways, and protecting the environment.

GRS-IBS (Geosynthetic Reinforced Soil – Integrated Bridge System)	An ABC technology that uses alternating layers of compacted granular fill material and fabric sheets of geotextile reinforcement to provide support for the bridge in place of a traditional abutment.
LBDB (Low Bid Design Build)	A type of D/B where the design and construction service is bundled into a single contract awarded to the lowest competent and responsible bidder.
PBES (Prefabricated Bridge Elements and Systems)	Structural components of a bridge or bridge system that are constructed offsite, or near-site of a bridge that reduce the onsite construction time and impact to the traveling public relative to conventional construction methods.
Pick Points	Locations where the SPMTs will lift and carry the bridge.
Program Initiative	The use of ABC methods to facilitate research, investigate technology, develop familiarity, or address other stakeholder needs.
Road User Costs	Costs pertaining to a project alternative borne by motorists and the community at-large as a result of work zone activity. (FDM 11-50-32)
SPMTs (Self Propelled Modular Transporters)	Remote-controlled, multi-axle platform vehicles capable of transporting several thousand tons of weight.
Stroke	Distance an SPMT can raise or lower its platform.
TMP (Transportation Management Plan)	A set of coordinated transportation management strategies that describes how they will be used to manage work zone impacts of a road project. (FDM 11-50-5)
TP (Travel Path)	Course that the SPMTs travel to carry the completed structure from the staging area to the final location.

Table 7.1-1
ABC Terminology

7.1.4 ABC Methods

7.1.4.1 Prefabricated Bridge Elements

Prefabricated bridge elements are a commonly used ABC method and can be incorporated into most bridge projects as a form of accelerated construction. Concrete bridge elements are prefabricated, transported to the construction site, placed in the final location, and tied into the structure. An entire bridge can be composed of prefabricated elements, or single bridge elements can be prefabricated as the need arises. Prefabricated bridge elements can also be used in combination with other accelerated bridge construction methods. Commonly used prefabricated bridge elements are prestressed concrete girders (including I-girders, adjacent inverted T-beams, and boxes), full depth and partial depth deck panels, abutments, pier caps, pier columns, and footings, as well as precast three-sided and four-sided box culverts.

For all prefabricated bridge elements, shop drawings shall be submitted by email to the Bureau of Structures Development Section Chief.

Figure 7.1-1
Prefabricated Pier Cap

Figure 7.1-2
Prefabricated Abutment

Prefabricated bridge elements are used to mitigate the on-site time required for concrete forming, rebar tying and concrete curing, saving weeks to months of construction time. Deck beam elements eliminate conventional onsite deck forming activities. To reduce onsite deck forming operations, deck beam elements are typically placed in an abutting manner. Prefabricated elements are often of higher quality than conventional field-constructed elements, because the concrete is cast and cured in a controlled environment. The elements are often connected using high strength grout, and post-tensioning or pretensioning. Because some previous prefabricated bridge element connections have had problems, close attention should be given to these connections.

7.1.4.2 Geosynthetic Reinforced Soil – Integrated Bridge Systems (GRS-IBS)

GRS is an engineered fill of closely spaced alternating layers of compacted fill and geosynthetic reinforcement that eliminates the need for traditional concrete abutments. IBS is a fast, cost-effective method of bridge support that blends the roadway into the superstructure using GRS technology. This integration system creates a transition area that allows for uniform settlement between the bridge substructure and the roadway approach, alleviating the “bump at the bridge” problem caused from uneven settlement. The result of this system is a smoother bridge approach.

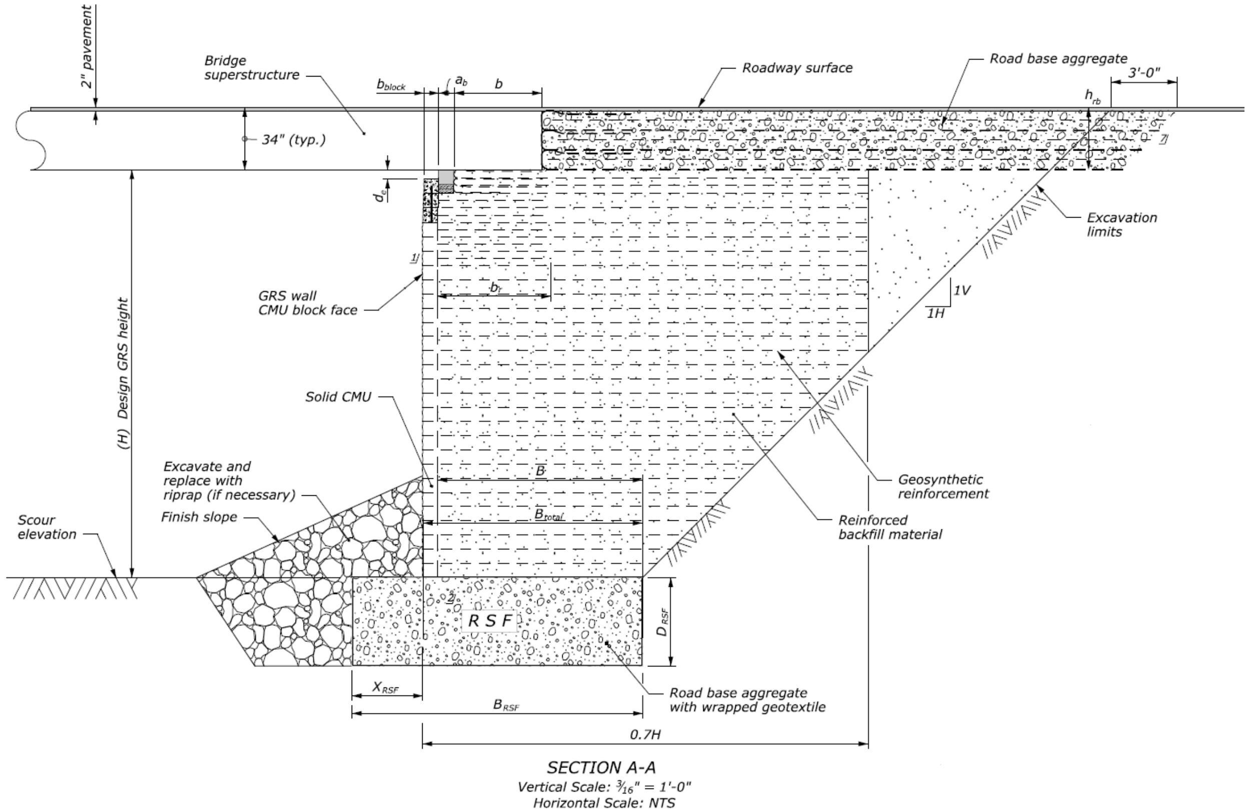

Figure 7.1-3
GRS-IBS Cross Section

GRS-IBS structures are very easy to build, as they use very common equipment and materials. Each reinforced soil layer is easy to construct. There are many variables that affect the speed of construction, but typically a two-man labor crew with one excavator/operator can complete one layer in about one hour. Since the reinforced soil layers eliminate the need for traditional abutments, no cast-in-place concrete is required for the substructure. One exception is if I-girders are used, a CIP backwall may be required to prevent the GRS mass from spilling through.

Figure 7.1-4
GRS-IBS Structure

Figure 7.1-5
GRS Abutment Layer during Construction

Typically, the bridge superstructure is placed directly on the reinforced soil abutment. Prestressed girders are often placed on top of the GRS substructure, followed by a traditional cast-in-place deck or precast deck panels.

This method is very cost effective and is typically used for short single span bridges. GRS-IBSs have the potential to reduce construction costs by approximately 30% as compared to traditional methods. This type of construction can occur in more variable weather conditions than conventional construction, which ultimately results in fewer delays and faster project delivery.

WisDOT policy item:

The use of GRS-IBS is subject to prior-approval by the Bureau of Structures for hydraulic design. Evaluation of scour vulnerability will include assessment of long-term aggradation and degradation, potential for lateral migration of the stream, and calculation of contraction scour and abutment scour. The conservative nature of abutment scour calculations is acknowledged. Placement of adequately designed permanent scour countermeasures will be required to resist calculated scour.

FHWA initially developed this accelerated construction technology and several states have successfully constructed bridges using this technique in recent years. The first bridge constructed in Wisconsin using the GRS-IBS technology was built in the spring of 2012. This structure is a research structure, and will be closely monitored for two years to assess its performance.

Refer to FHWA publication number FHWA-HRT-11-026 “Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide” for more information.

7.1.4.3 Lateral Sliding

Bridge placement using lateral sliding is another type of ABC where the entire superstructure is constructed in a temporary location and is moved into place over a night or weekend. This method is typically used for bridge replacement of a primary roadway where the new superstructure is constructed on temporary supports adjacent and parallel to the bridge being replaced. Once the superstructure is fully constructed, the existing bridge structure is demolished, and the new bridge is moved transversely into place. In some instances, a more complicated method known as a bridge launch has been used, which involves longitudinally moving a bridge into place.

Figure 7.1-6
Lateral Sliding

Several different methods have been used to slide a bridge into place. One common method is to push the bridge using a hydraulic ram while the bridge slides on a smooth surface and Teflon coated elastomeric bearing pads. Other methods have also been used, such as using rollers instead of sliding pads, and winches in place of a hydraulic ram. The bridge can also be built on a temporary support frame equipped with rails and pushed or pulled into place along those rails. Many DOTs have successfully replaced bridges overnight using lateral sliding.

This ABC method is used to replace bridges that are part of a main transportation artery traversing a minor road, waterway, or other geographic feature. The limiting factor with using lateral slide is having sufficient right-of-way, and space adjacent to the existing bridge to construct the new superstructure.

7.1.4.4 Self Propelled Modular Transporter (SPMT)

SPMTs are remote-controlled, multi-axle platform vehicles capable of transporting several thousand tons of weight. They have traditionally been used to move heavy equipment that is too large for standard trucks to carry. Over the past decade, the use of these SPMTs has been applied to rapid bridge replacement. The use of SPMTs for bridge placement has been used in Europe for more than 30 years. The United States has recently implemented this technique following the FHWA's recommendation in 2004 to learn how other countries used prefabricated bridge components to minimize traffic disruption, improve work zone safety, reduce environmental impact, improve constructability, enhance quality, and lower life-cycle costs.

Figure 7.1-7
Self Propelled Modular Transporters Moving a Bridge

When replacing a bridge using SPMTs the new superstructure is built on temporary abutments off-site in a designated bridge staging area (BSA) near the bridge site. Once the new superstructure is constructed, the removal of the existing structure can also be expedited with SPMTs or can be demolished in conventional time frames, depending on the project-specific needs. Once the existing structure is removed, the new superstructure is moved from the staging area to the final location using two or more lines of SPMTs. SPMTs have the ability to move laterally, rotate 360° with carousel steering, and have a vertical lifting stroke of approximately 18-24 inches. The SPMTs lift the superstructure off of the temporary abutments and transport it to the permanent abutments. The placement of the bridge superstructure using SPMTs often requires only one evening of full road closure, and many bridges in the United States have been placed successfully in a matter of hours.

SPMTs are typically used to replace bridges that span major roads or highways, and the traffic closure restrictions govern the need for a quick replacement. Locating an off-site staging area to build the superstructure is a critical component to use SPMTs. There needs to be a clearly defined travel path between the staging area and the final bridge location that can support the SPMT movements (vertical clearances, horizontal clearances, turning radii, soil conditions, utility conflicts, etc.). SPMTs can also be used to place a bridge over a waterway. In this case, the bridge superstructure is constructed offsite, and then SPMTs transport the superstructure onto a barge which travels the waterway to the bridge site where it can be set in place.

To date, mostly single span bridges with spans ranging from approximately 100 to 200 feet have been moved. There have been a few two-span bridge moves with SPMTs in the United States. Many of the superstructure types that have been moved successfully are prestressed I-girders or steel plate girders. Moving forward, as SPMTs are used more frequently, the limits of SPMT move capabilities will expand.

7.1.5 Project Delivery Methods/Bidding Process

In addition to the accelerating technologies discussed in this chapter, the Every Day Counts initiative includes accelerated project delivery methods as a way to shorten the project 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 Chapter 11-50-32 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.

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

% Weight	Category	Decision-Making Item	Possible Points	Points Allocated		Scoring Guidance
17%	Disruptions (on/under Bridge)	**Railroad on Bridge?**	8		0	No railroad track on bridge
					4	Minor railroad track on bridge
					8	Major railroad track on bridge
		Railroad under Bridge?	3		0	No railroad track under bridge
					1	Minor railroad track under bridge
					3	Major railroad track(s) under Bridge
		Over Navigation Channel that needs to remain open?	6		0	No navigation channel that needs to remain open
					3	Minor navigation channel that needs to remain open
					6	Major navigation channel that needs to remain open
8%	Urgency	**Emergency Replacement?**	8		0	Not emergency replacement
					4	Emergency replacement on minor roadway
					8	Emergency replacement on major roadway
23%	User Costs and Delays	**ADT and/or ADTT** (Combined Construction Year ADT on and under bridge)	6		0	No traffic impacts
					1	ADT under 10,000
					2	ADT 10,000 to 25,000
					3	ADT 25,000 to 50,000
					4	ADT 50,000 to 75,000
					5	ADT 75,000 to 100,000
					6	ADT 100,000+
		Required Lane Closures/Detours? (Length of Delay to Traveling Public)	6		0	Delay 0-5 minutes
					1	Delay 5-15 minutes
					2	Delay 15-25 minutes
					3	Delay 25-35 minutes
					4	Delay 35-45 minutes
					5	Delay 45-55 minutes
					6	Delay 55+ minutes
		Are only Short Term Closures Allowable?	5		0	Alternatives available for staged construction
					3	Alternatives available for staged construction, but undesirable
					5	No alternatives available for staged construction
		Impact to Economy (Local business access, impact to manufacturing etc.)	6		0	Minor or no impact to economy
					3	Moderate impact to economy
					6	Major impact to economy
14%	Construction Time	**Impacts Critical Path of the Total Project?**	6		0	Minor or no impact to critical path of the total project
					3	Moderate impact to critical path of the total project
					6	Major impact to critical path of the total project
		Restricted Construction Time (Environmental schedules, Economic Impact – e.g. local business access, Holiday schedules, special events, etc.)	8		0	No construction time restrictions
					3	Minor construction time restrictions
					6	Moderate construction time restrictions
					8	Major construction time restrictions
5%	Environment	**Does ABC mitigate a critical environmental impact or sensitive environmental issue?**	5		0	ABC does not mitigate an environmental issue
					2	ABC mitigates a minor environmental issue
					3	ABC mitigates several minor environmental issues
					4	ABC mitigates a major environmental issue
					5	ABC mitigates several major environmental issues
3%	Cost	**Compare *Comprehensive* Construction Costs** (Compare conventional vs. prefabrication)	3		0	ABC costs are 25%+ higher than conventional costs
					1	ABC costs are 1% to 25% higher than conventional costs
					2	ABC costs are equal to conventional costs
					3	ABC costs are lower than conventional costs
18%	Risk Management	**Does ABC allow management of a particular risk?**	6		0 - 6	Use judgment to determine if risks can be managed through ABC that aren't covered in other topics
		Safety (Worker Concerns)	6		0	Short duration impact with TMP Type 1
					3	Normal duration impact with TMP Type 2
					6	Extended duration impact with TMP Type 3-4
		Safety (Traveling Public Concerns)	6		0	Short duration impact with TMP Type 1
					3	Normal duration impact with TMP Type 2
					6	Extended duration impact with TMP Type 3-4
12%	Other	**Economy of Scale** (repetition of components in a bridge or bridges in a project) (Total spans = sum of all spans on all bridges on the project)	5		0	1 total span
					1	2 total spans
					2	3 total spans
					3	4 total spans
					4	5 total spans
					5	6+ total spans
		Weather Limitations for conventional construction?	2		0	No weather limitations for conventional construction
					1	Moderate limitations for conventional construction
					2	Severe limitations for conventional construction
		Use of Typical Standard Details (Complexity)	5		0	No typical standard details will be used
					3	Some typical standard details will be used
					5	All typical standard details will be used
			Sum of Points:	0		(100 Possible Points)

Figure 7.2-1

ABC Decision-Making Matrix

7.2.1 Description of terms in ABC Decision-Making Matrix

The following text describes each item in the ABC Decision-Making Matrix (Figure 7.2-1). The points associated with the scoring guidance in the matrix and in the text below are simply *guidance*. Use engineering judgment and interpolate between the point ranges as necessary.

Decision-Making Item	Scoring Guidance Description
Railroad on Bridge?	This is a measure of how railroad traffic on the bridge will be affected by the project. If a major railroad line runs over the bridge that requires minimum closures and a shoo fly (a temporary railroad bridge bypass) cannot be used, provide a high score here. If a railroad line that is rarely used runs over the bridge, consider providing a mid-range or low score here. If there is no railroad on the bridge, assign a value of zero here.
Railroad under Bridge?	This is a measure of how railroad traffic under the bridge will be affected by the project. If a major railroad line runs under the bridge that would disrupt construction progress significantly, provide a high score here. If a railroad track runs under the structure, but it is used rarely enough that it will not disrupt construction progress significantly, provide a low score here. Consider if the railroad traffic is able to be suspended long enough to move a new bridge into place. If there is not a large enough window to move a new bridge into place, SPMT could be eliminated as an alternative for this project. For this case, PBES may be a more applicable alternative. If there is no railroad under the bridge, assign a value of zero here.
Over Navigation Channel that needs to remain open?	This is a measure of how a navigation channel under a bridge will be affected by the project. If a navigation channel is highly traveled and needs to remain open for shipments, provide a high score here. If a navigation channel is rarely traveled and there are not requirements for it to remain open at certain time periods, provide a low score here. If there is no navigation channel under the bridge, assign a value of zero here.
Emergency Replacement?	This is a measure of the urgency of the bridge replacement. A more urgent replacement supports the use of accelerated bridge construction methods, since demolition and construction can be progressing concurrently. Depending on the particular project, accelerated bridge construction methods can also allow multiple components of the bridge to be constructed concurrently. If the bridge replacement is extremely urgent and the bridge can be replaced quicker by using accelerated construction methods, provide a high score here.

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 *Facilities Development Manual (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 *Facilities Development Manual (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

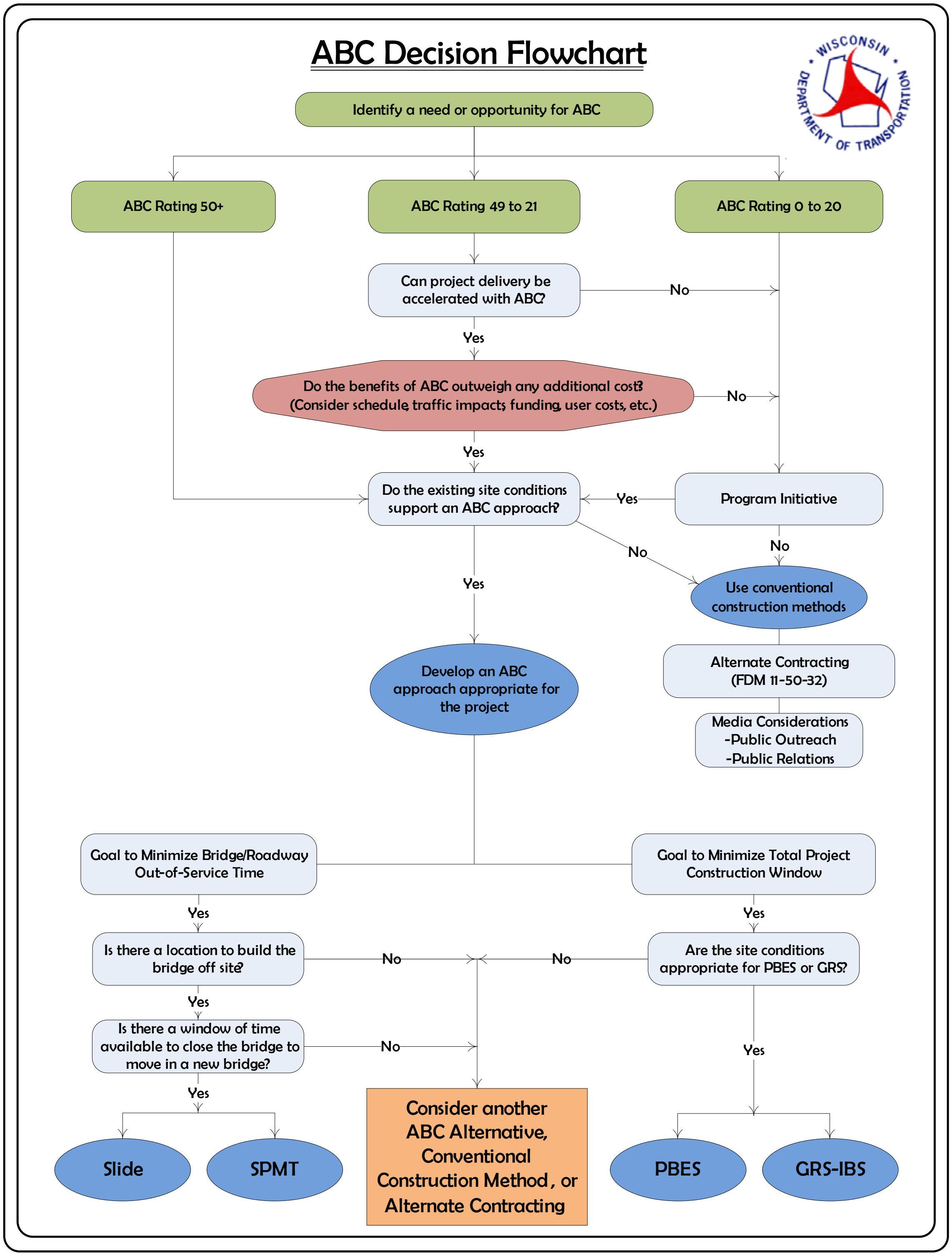

Figure 7.2-2
ABC Decision-Making Flowchart

7.3 References

1. Every Day Counts Initiative. Federal Highway Administration. 23 May. 2012. <http://www.fhwa.dot.gov/everydaycounts/>

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Table of Contents

8.1 Introduction

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

8.1.1 Objectives of Highway Drainage

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

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

8.1.2 Basic Policy

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

8.1.3 Design Frequency

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

8.4.2.6 Roadway Overflow

See 8.3.2.6.

8.4.2.7 Outlet Scour and Energy Dissipators

Energy dissipating devices are used where it is desirable to reduce the discharge velocity by inducing high energy losses at the inlet or discharge ends of the structure. They are generally warranted when discharge velocities exceed 14 feet per second.

Energy losses may be induced at the culvert entrance with a drop inlet, or at the outlet using energy dissipating devices and stilling basins to form a hydraulic jump.

Drop inlets are used where headroom is limited, and energy dissipating devices and stilling basins at the discharge are used where headroom is not critical.

The use of drop inlets should generally be reserved for areas where channel slopes are steep. Under these conditions drop inlets enable the reduction of culvert grades and in turn lower discharge velocities. When evaluating a site, a drop inlet may also be applicable on drainage ditches, in addition to channels that are normally dry or do not support fish or other aquatic organism habitat of pronounced significance. The use of a drop inlet requires approval from the Bureau of Structures, as well as coordination with the Department of Natural Resources early in project development.

For outlet devices utilizing the hydraulic jump, two conditions must be present for the formation of a hydraulic jump; the approach depth must be less than critical depth (supercritical flow); and the tailwater depth must be deeper than critical depth (subcritical flow) and of sufficient depth to control the location of the hydraulic jump. Where the tailwater depth is too low to cause a hydraulic jump at the desired location, the required depth can be provided by either depressing the discharge apron or utilizing a broad-crested weir at the end of the apron to provide a pool of sufficient depth. The depressed apron method is preferred since there is less scouring action at the end of the apron. The amount of depression is determined as the difference between the natural tailwater depth and the depth required to form a jump.

There are numerous design concepts of energy dissipating devices and stilling basins that may be adapted for energy dissipation to reduce the velocity and avoid scour at the culvert outlet. The more common type of designs are drop inlets, drop outlets, hydraulic jump stilling basins and riprap stilling basins.

More discussion on energy dissipators for culverts is available in 8.5 references (19), (20), (21), and (22). The designer is strongly advised to closely examine and study reference (20). More detailed discussions about the various types of energy dissipators and their designs are presented in that reference.

8.4.2.7.1 Drop Inlet.

In drop inlet design, flow is controlled at the inlet crest by the weir effect of the drop opening. Drop inlet culverts operate most satisfactorily when the height of drop is sufficient to permit considerable submergence of the culvert entrance without submerging the weir or exceeding limiting headwater depths.

Referring to Figure 8.4-7, the general formula for flow into the horizontal drop opening is:

$$Q = C_1 (2g)^{1/2} L H^{3/2}$$

Where Q is the discharge in c.f.s., L is the crest length 2B+W, H is the depth of flow plus velocity head, and C_1 is a dimensionless discharge coefficient taken as 0.4275. The formula is expressed in english units as:

$$Q = 3.43 LH^{3/2}$$

and

$$L = Q/(3.43H^{3/2})$$

There are four corrections which have to be multiplied times the discharge coefficient C_1, or times the factor 3.43:

1. Correction for head H/W (Table 8.4-1)

2. Correction for box-inlet shape B/W. (Table 8.4-2)

3. Correction for approach channel width W_c/L (Table 8.4-3).

   Where: W_c = approach channel width = Area/Depth

4. Correction for dike effect X/W (Table 8.4-4)

The size of the culvert should be determined by using the discharge (Q) and not allowing the height of water (HW) to exceed the inlet drop plus the critical depth of the weir which is given as:

$$d_c = [(Q/L)^2/g]^{1/3}$$

When using the hydraulic charts of 8.4.2.5, consider the culvert to have a wingwall flare of 0 degrees (extension of sides).

Sample computations are shown in 8.4.2.7.1.1.

Table of Contents

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 www.atwoodsystems.com/materials.

The *Wisconsin Construction and Materials Manual* (CMM) contains a description of procedures for material testing and acceptance requirements in Chapter 45, Section 25. 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. The current paint system used is the three coat epoxy system specified in Section 517 of the *Standard Specifications*.

Recommended standard colors and paint color numbers for steel girders in Wisconsin in accordance with Federal Standard No. 595B as printed 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.

Federal Standard No. 595B can be found at www.colorserver.net/

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.

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.

Refer to Section 3.14 of the *Wisconsin Structure Inspection Manual* for the criteria covering spot painting versus complete painting of existing structures. This Section provides information for evaluating the condition of a paint system and recommended maintenance.

Recommended standard colors and color numbers for concrete in Wisconsin in accordance with Federal Standard No. 595B as printed 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.

Table of Contents

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 and geotechnical 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 site 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 sheets 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 loading.

- Depth to suitable bearing material.

- Potential for liquefaction, undermining or scour.

- Frost potential.

Foundations without piles (spread footings) should be given consideration at sites where pile penetrations of less than 10 feet are anticipated. The economics of the following two alternatives should be investigated:

1. Design for a shallow foundation founded at a depth where the foundation material is adequate. Embed the footing 6 inches into sound rock for lateral stability.

2. Excavate to an elevation where foundation material is adequate, and backfill to the bottom of footing elevation with suitable granular material or concrete.

If a substructure unit is located in a stream, consideration should be given to the effects of the anticipated stream bed scour when selecting the footing type. Pile length computations should not incorporate pile resistance developed within the scour zone. The pile cross section should also be checked to ensure it can withstand the driving necessary to penetrate through the anticipated scour depth and reach design capacity plus the frictional capacity within the scour zone.

11.3.1.2 Pile Spacing

Arbitrary pile spacing rules specifying maximums and minimums are extensively used in foundation design. Proper spacing is dependent upon length, size, shape and surface texture of piles, as well as soil characteristics. A wide spacing of piles reduces heaving and possible uplifting of the pile, damage by tension due to heaving and the possibility of crushing from soil compression. Wider spacing more readily permits the tips of later-driven piles in the group to reach the same depths as the first piles and result in more even bearing and settlement. Large horizontal pressures are created when driving in relatively uncompressible strata, and damage may occur to piles already driven if piles are too closely spaced. In order to account for this, a minimum center-to-center spacing of 2.5 times the pile diameter is often required. LRFD [10.7.1.2] calls for a center-to-center pile spacing of not less than 2'-6" or 2.5 pile diameters (widths).

<u>WisDOT policy item:</u>

WisDOT's minimum pile spacing is 2'-6" or 2.5 pile diameters, whichever is greater. For displacement piles located within cofferdams, or with estimated lengths ≥ 100 ft., the minimum pile spacing is 3.0 pile diameters. The minimum pile spacing for pile encased piers and pile bents is 3'-0". WisDOT's maximum pile spacing is 8'-0" for abutments, pile encased piers, and pile bents.

See Chapter 13 – Piers for criteria on battered piles in cofferdams. The distance from the side of any pile to the nearest edge of footing shall not be less than 9". Piles shall project at least 6" into the footings.

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. 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 FDM Procedure 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.

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, or driving through dense granular materials and hardpan layers. 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.

Pile Size	Shell Thickness (inches)	Concrete or Steel Area (A_g or A_s) (in^2)	Nominal Axial Compression Resistance (Pn) (tons) (2)(3)(6)	Resistance Factor (ϕ)	Factored Axial Compression Resistance (Pr) (tons) (4)	Resistance Factor φ_{dyn}	Required Driving Resistance (Rn_{dyn}) (tons) (5)
Cast in Place Piles							
10 ¾“	0.219	83.5	99.4	0.75	55(8)	0.5	110
10 ¾“	0.250	82.5	98.2	0.75	65(8)	0.5	130
10 ¾“	0.365	78.9	93.8	0.75	75	0.5	150
10 ¾“	0.500	74.7	88.8	0.75	75(9)	0.5	150
12 ¾”	0.250	118.0	140.4	0.75	80(8)	0.5	160
12 ¾”	0.375	113.1	134.6	0.75	105	0.5	210
12 ¾”	0.500	108.4	129.0	0.75	105(9)	0.5	210
14”	0.250	143.1	170.3	0.75	85(8)	0.5	170
14”	0.375	137.9	164.1	0.75	120	0.5	240
14”	0.500	132.7	158.0	0.75	120(9)	0.5	240
16”	0.375	182.6	217.3	0.75	145(8)	0.5	290
16”	0.500	176.7	210.3	0.75	160	0.5	320
H-Piles							
10 x 42	NA(1)	12.4	310.0	0.50	90(10)	0.5	180
12 x 53	NA(1)	15.5	387.5	0.50	110(10)	0.5	220
14 x 73	NA(1)	21.4	535.0	0.50	125(10)	0.5	250

Table 11.3-5
Typical Pile Resistance Values

Notes

1. NA – not applicable
2. For CIP Piles: Pn = 0.8 (0.85 * f'c * Ag + fy * As) **LRFD [5.5.4.2.1]**. Neglecting the steel shell, equation reduces to 0.68 * f'c * Ag.

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

3. For H-Piles: Pn = 0.66^{λ} * Fy * As (λ = 0 for piles embedded below the substructure)

   fy = yield strength of steel = 50,000 psi

4. Pr = ϕ * Pn

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

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

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

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

     φ_{dyn} = 0.5 (**LRFD [Table 10.5.5.2.3-1]** for construction driving criteria using modified Gates dynamic formula)

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

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

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

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

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

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

11.3.1.18 Construction Considerations

Construction considerations generally include selection of pile hammers, use of driving formulas and installation of test piles, when appropriate, as described below.

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

11.3.1.19 Construction Monitoring for Economic Evaluation of Deep Foundations

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

Methods Used to Determine Required Driving Resistance	Resistance Factor
FHWA-modified Gates dynamic pile driving formula (end of drive condition only).	0.50
Driving criteria established by dynamic test [Pile Driving Analyzer, (PDA)] with signal matching [CAse Pile Wave Analysis Program, (CAPWAP)] at beginning of redrive conditions only, of at least one production pile per substructure, but no less than the number of tests per site provided in LRFD[Table 10.5.5.2.3-3]. Quality control of remaining piles by calibrated wave equation and/or dynamic testing.	0.65
Static Pile Load Test(s) and dynamic test (PDA) with signal matching (CAPWAP) at beginning of redrive conditions only, of at least one production pile per substructure, but no less than the number of tests per site provided in LRFD[Table 10.5.5.2.3-3]. Quality control of remaining piles by calibrated wave equation and/or dynamic testing.	0.80

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

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

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

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

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

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

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

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

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

The following two examples assume the use of the Department's PDA and the Department's personnel to collect the required data and illustrate the potential cost savings/expenses. If the consultant's PDA and/or the consultant's personnel are used to collect the required data then the cost savings will be significantly reduced:

Pier
Pier Example: 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $50/foot.
Modified Gates: RDR = 220 tons, FACR = 110 tons, Total Load on Pier = 3,500 tons, Number of Piles = 3,500 tons / 110 tons = 32 piles (**Note:** It is realized that for pier design the number of piles is not exclusively related to the vertical load, but this example is simplified for illustrative purposes). Total Cost = 32 piles x 100 feet x $50/ft = $160,000
PDA/CAPWAP: RDR = 220 tons, FACR = 143 tons, Load on Pier = 3,500 tons, Number of Piles = 3,500 tons / 143 tons = 25 piles, 3 tests at end of initial driving (EOID), 3 tests at beginning of restrike (BOR) Total Cost =Number of Piles x Estimated Length of Pile x Cost/Linear Foot of Pile + ((CAPWAP cost ($1,000/test) + contractor cost ($100/pile) + DOT cost ($125/pile)) Total Cost = 25 piles x 100 feet x $50/ft + ((6 tests x $1,000/test) + (6 piles x $100/pile) + (6 piles x $125/pile)) = $132,1350
PDA/CAPWAP Savings = $27,650/pier
Abutment
Abutment Example: 12 x 53 H-piles to an estimated length of 40 feet at a unit cost of $50/foot.
Modified Gates: RDR = 220 tons, FACR = 110 tons, Total Load on Abut = 980 tons, Number of Piles = 980 tons / 110 tons = 9 piles Total Cost = 9 piles x 40 feet x $50/ft = $18,000
PDA/CAPWAP: RDR = 220 tons, FACR = 143 tons, Load on Abut = 980 tons, Number of Piles = 980 tons / 140 tons = 7 piles, however because of maximum spacing requirements the design will

need 8 piles, 1 test at end of initial driving (EOID), 1 test at beginning of restrike (BOR) Total Cost =Number of Piles x Estimated Length of Pile x Cost/Linear Foot of Pile + ((CAPWAP cost ($1,000/test) + contractor cost ($100/pile) + DOT cost ($125/pile)) Total Cost = 8 piles x 40 feet x $50/ft + ((2 test x $1,000/test) + (2 piles x $100/pile) + (2 piles x $125/pile)) = $18,450
PDA/CAPWAP Increase = $450/abutment

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

11.3.2 Drilled Shafts

11.3.2.1 General

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

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

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

Drilled shafts are generally considered fixed to the substructure unit if the reinforcing steel from the shaft is fully developed within the substructure unit.

Drilled shafts vary in diameter from approximately 2.5 to 10 feet. Drilled shafts with diameters greater than 6 feet are generally referred to as piers. Shafts may be designed to transfer load to the bearing stratum through side friction, point-bearing or a combination of both. The drilled shaft may be cased or uncased, depending on the subsurface conditions and depth of bearing.

Drilled shafts have been used on only a small number of structures in Wisconsin. For unusual site conditions, the use of drilled shafts may be advantageous. Design methodologies for drilled shafts can be found in FHWA Publication IF-99-025, *Drilled Shafts: Construction Procedures and Design Methods*.

Strength limit states for drilled shafts are evaluated in the same way as for driven piles. Drivability is not required to be evaluated. The structural resistance of drilled shafts is evaluated in accordance with **LRFD [5.7 and 5.8]**. This includes evaluation of axial resistance, combined axial and flexure, shear and buckling. It is noted that the critical load case for combined axial and flexure may be a load case that results in the minimum axial load or tension.

11.3.2.2 Resistance Factors

Resistance factors for drilled shafts are presented in Table 11.3-6 and are selected based on the method used to determine the nominal (ultimate) resistance capacity of the drilled shaft. The design intent is to adjust the resistance factor based on the reliability of the method used to determine the nominal shaft resistance. As with driven piles, the selection of a geotechnical resistance factor should be based on the intended method of resistance verification in the field. Because of the cost and difficulty associated with testing drilled shafts, much more reliance is placed on static analysis methods.

Condition/Resistance Determination Method				Resistance Factor
Static Analysis – Used in Design Phase	Nominal Resistance of Single-Drilled Shaft in Axial Compression, φ_{stat}	Shaft Resistance in Clay	Alpha Method	0.45
		Point Resistance in Clay	Total Stress	0.40
		Shaft Resistance in Sand	Beta Method	0.55
		Point Resistance in Sand	O'Neill and Reese	0.50
		Shaft Resistance in IGMs	O'Neill and Reese	0.60
		Point Resistance in IGMs	O'Neill and Reese	0.55
		Shaft Resistance in Rock	Horvath and Kenney O'Neill and Reese	0.55
			Carter and Kulhawy	0.50
		Point Resistance in Rock	Canadian Geotech. Soc. Pressuremeter Method O'Neill and Reese	0.50
	Block Failure, φ_{bl}	Clay		0.55
	Uplift Resistance of Single-Drilled Shaft, φ_{up}	Clay	Alpha Method	0.35
		Sand	Beta Method	0.45
		Rock	Horvath and Kenney Carter and Kulhawy	0.40
	Group Uplift Resistance, φ_{ug}	Sand and Clay		0.45
	Horizontal Geotechnical Resistance of Single Shaft or Pile Group	All Soil Types and Rock		1.0

Table 11.3-8

Geotechnical Resistance Factors for Drilled Shafts **LRFD [Table 10.5.5.2.4-1]**

For drilled shafts, the base geotechnical resistance factors in Table 11.3-6 assume groups containing two to four shafts, which are slightly redundant. For groups containing at least four elements, the base geotechnical resistance factors in Table 11.3-6 should be increased by 20%. WisDOT generally uses 4 or more shafts per substructure unit.

WisDOT policy item:

WisDOT policy requires a multi-column bent to be designed as a redundant rigid frame. Hence when a bent contains at least 4 columns then the resistance factors in Table 11.3-6 should be increased by 20 percent.

Resistance factors for structural design of drilled shafts are obtained from **LRFD [5.5.4.2.1]**.

11.3.2.3 Bearing Resistance

Most drilled shafts provide geotechnical resistance in both end bearing and side friction. Because the rate at which side friction mobilizes is usually much higher than the rate at which end bearing mobilizes, past design practice has been to ignore either end bearing for shafts with significant sockets into the bearing stratum or to ignore skin friction for shafts that do not penetrate significantly into the bearing stratum. This makes evaluation of the geotechnical resistance slightly more complex, because in most cases it is not suitable to simply add the nominal (ultimate) end bearing resistance and the nominal side friction resistance in order to obtain the nominal axial geotechnical resistance.

When computing the nominal geotechnical resistance, consideration must be given to the anticipated construction technique and the level of construction control. If it is anticipated to be difficult to adequately clean out the bottom of the shafts due to the construction technique or subsurface conditions, the end bearing resistance may not be mobilized until very large deflections have occurred. Similarly, if construction techniques or subsurface conditions result in shaft walls that are very smooth or smeared with drill cuttings, side friction may be far less than anticipated.

Because these resistances mobilize at different rates, it may be more appropriate to add the ultimate end bearing to that portion of the side resistance remaining at the end of bearing failure. Or it may be more appropriate to add the ultimate side resistance to that portion of the end bearing mobilized at side resistance failure. Note that consideration of deflection, which is a service limit state, may control over the axial geotechnical resistance since displacements required to mobilize the ultimate end bearing can be excessive. More detailed discussion of design parameters is provided in Appendices C and D of FHWA Publication IF-99-025, *Drilled Shafts: Construction Procedures and Design Methods*.

11.3.2.3.1 Shaft Resistance

The shaft resistance is estimated by summing the friction developed in each stratum. When drilled shafts are socketed in rock, the shaft resistance that is developed in soil is generally ignored to satisfy strain compatibility. The following analysis methods are typically used to compute the static shaft resistance in soil and rock:

- Alpha method for cohesive soil, as specified in **LRFD [10.8.3.5.1]**
- Beta method (β-method) for cohesionless soil, as specified in **LRFD [10.8.3.5.2]**
- Horvath and Kenny method for rock, as specified in **LRFD [10.8.3.5.4]**

11.3.2.3.2 Point Resistance

The following analysis methods are typically used to compute the static shaft resistance in soil:

- Alpha method for cohesive soil, as specified in **LRFD [10.8.3.5.1]**
- Beta method (β-method) for cohesionless soil, as specified in **LRFD [10.8.3.5.2]**

The ultimate unit point resistance of a drilled shaft in intact or tightly jointed rock is computed as 2.5 times the unconfined compressive strength of the rock. For rock containing open or filled joints, the geomechanics RMR system is used to characterize the rock, and the ultimate point resistance in rock can be computed as specified in **LRFD [10.8.3.5.4c]**.

11.3.2.3.3 Group Capacity

For drilled shaft groups bearing in cohesive soils or ending in a strong layer overlying a weaker layer, the axial resistance is determined using the same approach as used for driven piles. For drilled shaft groups in cohesionless soil, a group efficiency factor is applied to the ultimate resistance of a single drilled shaft. The group efficiency factor is a function of the center-to-center shaft spacing and is linearly interpolated between a value of 0.65 at a center-to-center spacing of 2.5 shaft diameters and a value of 1.0 at a center-to-center spacing of 6.0 shaft diameters. This reduction is more than for driven piles at similar spacing, because construction of drilled shafts tends to loosen the soil between the shafts rather than densify it as with driven piles.

11.3.2.4 Lateral Load Resistance

Because drilled shafts are made of reinforced concrete, the lateral analysis should consider the nonlinear variation of bending stiffness with respect to applied bending moment. At small applied moments, the reinforced concrete section performs elastically based on the size of the section and the modulus of elasticity of the concrete. At larger moments, the concrete cracks in tension and the stiffness drops significantly.

11.3.2.5 Other Considerations

Detailing of the reinforcing steel in a drilled shaft must consider the constructability of the shaft. The reinforcing cages must be stiff enough to resist bending during handling and concrete placement. In addition, the spaces between reinforcement bars must be kept large enough to permit easy flow of the concrete from the center of shaft to the outside of shaft. These two requirements will generally force the use of larger, more widely spaced longitudinal and transverse reinforcement bars than would be used in the design of an above-grade column. In addition, when using hooked bars to tie the shaft to the foundation, consideration must also be given to concrete placement requirements and temporary casing removal requirements.

11.3.3 Micropiles

11.3.3.1 General

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

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

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

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

11.3.3.2 Design Guidance

Micropiles shall be designed using an Allowable Stress Design approach until an LRFD approach has been developed and approved by the AASHTO Bridge Subcommittee. The design of micropiles shall be done in accordance with FHWA Publication SA-97-070, *Micropile Design and Construction Guidelines Implementation Manual*. When site-specific load tests are performed, the factor of safety can be reduced from 2.5 to 2.0 to determine the allowable axial compressive load capacity of the micropile. The reduction in factor of safety is consistent with the 2005 update to the FHWA guidelines for micropile design.

11.3.4 Augered Cast-In-Place Piles

11.3.4.1 General

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

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

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

11.3.4.2 Design Guidance

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

11.4 References

1. State of Wisconsin, Department of Transportation. *Standard Specifications for Highway and Structure Construction*.

2. American Association of State Highway and Transportation Officials. *Standard Specifications for Highway Bridges*.

3. American Association of State Highway and Transportation Officials. *AASHTO LRFD Bridge Design Specifications*.

4. Armour, T., P. Groneck, J. Keeley and S. Sharma. *Micropile Design and Construction Guidelines, Implementation Manual*. FHWA Publication SA-97-070. June 2000.

5. Chellis, R.D. *Pile Foundations*. McGraw Hill. 1961.

6. Hannigan, P.J., G.G. Goble, G. E. Likins and F. Rausche. *Design and Construction of Driven Pile Foundations – Volumes 1 and 2*. FHWA Publication NHI-05-042. 2006.

7. Kimmerling, R.E., R.C. Bachus, P.W. Mayne, J.A. Scheider and T.E. Zettler. *Geotechnical Engineering Circular No. 6: Shallow Foundations*. FHWA Publication SA-02-054. September 2002.

8. Lagrasse, P.F., J.D. Schall and E.V. Richardson. *Hydraulic Engineering Circular No. 20: Stream Stability at Highway Structures*, 3rd Edition. FHWA Publication NHI 01-002. March 2001.

9. Lagrasse, P.F., L.W. Zevenbergen, J.D. Schall and P.E. Clopper. *Hydraulic Engineering Circular No. 23: Bridge Scour and Stream Instability Countermeasures, Experience, Selection, and Design Guidance*, 2nd Edition. FHWA Publication NHI 01-003. March 2001.

10. Leonards, G.A. *Foundation Engineering*. McGraw Hill. 1962.

11. Michael Baker Jr., Inc. *LRFD for Highway Bridge Substructures and Earth Retaining Structures*. FHWA Publication NHI-05-095. Revised October 2006.

12. *Micropile Design and Construction Reference Manual*. FHWA Publication NHI-05-039. 2005.

13. O'Neil, M.W. and L.C. Reese. *Drilled Shafts: Construction Procedures and Design Methods*. FHWA Publication IF-99-025. August 1999.

14. O'Neil, M.W., F.C. Townsend, K.M. Hassan, A. Butler and P.S. Chan. *Load Transfer for Drilled Shafts in Intermediate Geomaterials*. FHWA Publication RD-95-172. November 1996.

15. Richardson, E.V. and S.R. Davis. *Hydraulic Engineering Circular No. 18: Evaluating Scour at Bridges*, 4th Edition. FHWA Publication NHI 01-001. May 2001.

16. Sabatini, P.J. et al. *Geotechnical Engineering Circular No. 5: Evaluation of Soil and Rock Properties*. FHWA Publication IF-02-034. April 2002.

17. Terzaghi, K. and R.B. Peck. *Soil Mechanics in Engineering Practice*, 2nd Edition. Wiley. 1967.

18. Tschebotrarioff, G.P. *Foundations, Retaining and Earth Structures*, 2nd Edition. McGraw Hill. 1973.

11.5 Design Examples

WisDOT will provide design examples.

This section will be expanded later when the design examples are available.

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Table of Contents

for substructure wind loads is 0.00, because wind loads in the transverse direction may be ignored in abutment and wing wall design.

Direction of Load	Specific Loading	Load Factor		
		Strength I		Service I
		Max.	Min.	
Load factors for vertical loads	Superstructure DC dead load	1.25	0.90	1.00
	Superstructure DW dead load	1.50	0.65	1.00
	Superstructure live load	1.75	1.75	1.00
	Approach slab dead load	1.25	0.90	1.00
	Approach slab live load	1.75	1.75	1.00
	Wheel loads located directly on the abutment backwall	1.75	1.75	1.00
	Earth surcharge	1.50	0.75	1.00
	Earth pressure	1.35	1.00	1.00
	Water load	1.00	1.00	1.00
	Live load surcharge	1.75	1.75	1.00
Load factors for horizontal loads	Substructure wind load	0.00	0.00	0.00
	Superstructure wind load	0.00	0.00	0.30
	Vehicular braking force from live load	1.75	1.75	1.00
	Temperature and shrinkage	1.20	0.50	1.00
	Earth pressure (active)	1.50	0.90	1.00
	Earth surcharge	1.50	0.75	1.00
	Live load surcharge	1.75	1.75	1.00

Table 12.8-2
Load Factors Used in Abutment Design

12.8.3 Live Load Surcharge

The equivalent heights of soil for vehicular loading on abutments perpendicular to traffic are as presented in **LRFD [Table 3.11.6.4-1]** and in Table 12.8-3. Values are presented for various abutment heights. The abutment height, as used in Table 12.8-3, is taken as the distance between the top surface of the backfill at the back face of the abutment and the bottom of the footing along the pressure surface being considered. Linear interpolation should be used for intermediate abutment heights. The load factors for both vertical and horizontal components of live load surcharge are as specified in **LRFD [Table 3.4.1-1]** and in Table 12.8-2.

Abutment Height (Feet)	h_{eq} (Feet)
5.0	4.0
10.0	3.0
≥ 20.0	2.0

Table 12.8-3
Equivalent Height, h_{eq}, of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

WisDOT policy item:

The equivalent height of soil for vehicular loading on retaining walls parallel to traffic shall be 2.0 feet, regardless of the wall height. For standard unit weight of soil equal to 120 pcf, the resulting live load surcharge is 240 psf.

For abutments with concrete approach slabs, one half of the equivalent height of soil shall be used to calculate the horizontal load on the abutment. The design lane load shall be applied to the concrete approach slab and this reaction shall be applied as a vertical load on the abutment.

12.8.4 Other Abutment Design Parameters

The equivalent fluid unit weights of soils are as presented in **LRFD [Table 3.11.5.5-1]**. Values are presented for loose sand or gravel, medium dense sand or gravel, and dense sand or gravel. Values are also presented for level or sloped backfill and for at-rest or active soil conditions.

Table 12.8-4 presents other parameters used in the design of abutments and wing walls. Standard details are based on the values presented in Table 12.8-4.

Description	Value
Bottom reinforcing steel cover	3.0 inches
Top reinforcing steel cover	2.0 inches
Unit weight of concrete	150 pcf
Concrete strength, f'_c	3.5 ksi
Reinforcing steel yield strength, f_y	60 ksi
Reinforcing steel modulus of elasticity, E_s	29,000 ksi
Unit weight of soil	120 pcf
Unit weight of structural backfill	120 pcf
Soil friction angle	30 degrees

Table 12.8-4
Other Parameters Used in Abutment Design

12.11 Bridge Approach Design and Construction Practices

While most bridge approaches are reasonably smooth and require a minimum amount of maintenance, there are also rough bridge approaches with maintenance requirements. The bridge designer should be aware of design and construction practices that minimize bridge approach maintenance issues. Soils, design, construction and maintenance engineers must work together and are jointly responsible for efforts to eliminate rough bridge approaches.

An investigation of the foundation site is important for bridge design and construction. The soils engineer, using tentative grades and foundation site information, can provide advice on the depth of material to be removed, special embankment foundation drainage, surcharge heights, waiting periods, construction rates and the amount of post-construction settlement that can be anticipated. Some typical bridge approach problems include the following:

- Settlement of pavement at end of approach slab
- Uplift of approach slab at abutment caused from swelling soils or freezing
- Backfill settlement under flexible pavement
- Approach slab not adequately supported at the abutments
- Erosion due to water infiltration

Most bridge approach problems can be minimized during design and construction by considering the following:

- Embankment height, material and construction methods
- Subgrade, subbase and base material
- Drainage-runoff from bridge, surface drains and drainage channels
- Special approach slabs allowing for pavement expansion

Post-construction consolidation of material within the embankment foundation is the primary contributor to rough bridge approaches. Soils which consist predominantly of sands and gravels are least susceptible to consolidation and settlement. Soils with large amounts of shales, silts and plastic clays are highly susceptible to consolidation.

The following construction measures can be used to stabilize foundation materials:

- Consolidate the natural material. Allow sufficient time for consolidation under the load of the embankment. When site investigations indicate an excessive length of time is required, other courses of corrective action are available. Use of a surcharge fill is effective where the compressive stratum is relatively thin and sufficient time is available for consolidation.

- Remove the material either completely or partially. This procedure is practical if the foundation depth is less than 15 feet and above the water table.

- Use lightweight embankment materials. Lightweight materials (fly ash, expanded shale and cinders) have been used with apparent success for abutment embankment construction to lessen the load on the foundation materials.

Abutment backfill practices that help minimize either settlement or swell include the following:

- Use of select materials

- Placement of relatively thin 4- to 6-inch layers

- Strict control of moisture and density

- Proper compaction

- Installation of moisture barriers

It is generally recognized by highway and bridge engineers that bridge abutments cause relatively few of the problems associated with bridge approaches. Proper drainage needs to be provided to prevent erosion of embankment or subgrade material that could cause settlement of the bridge approach. It is essential to provide for the removal of surface water that leaks into the area behind the abutment by using weepholes and/or drain tile. In addition, water infiltration between the approach slab and abutment body and wings must be prevented.

Reinforced concrete approach slabs are the most effective means for controlling surface irregularities caused by settlement. It is also important to allow enough expansion movement between the approach slab and the approach pavement to prevent horizontal thrust on the abutment.

WisDOT policy item:

Structural approach slabs shall be used on all Interstate Highway bridges and U.S.H. bridges. Other locations can be considered with the approval of the Chief Structural Design Engineer.

Standards for Structural Approach Slab for Type A1 Abutments and Structural Approach Slab Details for Type A1 Abutments are available for guidance.

Table of Contents

13.2 Pier Types

The pier types most frequently used in Wisconsin are:

- Multi-column piers (Standards for Multi-Columned Pier and for Multi-Columned Pier – Type 2)
- Pile bents (Standard for Pile Bent)
- Pile encased piers (Standard for Pile Encased Pier)
- Solid single shaft / hammerheads (Standards for Hammerhead Pier and for Hammerhead Pier – Type 2)

Design loads shall be calculated and applied to the pier in accordance with 13.4 and 13.5. The following sections discuss requirements specific to each of the four common pier types.

13.2.1 Multi-Column Piers

Multi-column piers, as shown in Standard for Multi-Columned Pier, are the most commonly used pier type for grade separation structures. Refer to 13.6 for analysis guidelines.

A minimum of three columns shall be provided to ensure redundancy should a vehicular collision occur. If the pier cap cantilevers over the outside columns, a square end treatment is preferred over a rounded end treatment for constructability. WisDOT has traditionally used round columns. Column spacing for this pier type is limited to a maximum of 25'.

Multi-column piers are also used for stream crossings. They are especially suitable where a long pier is required to provide support for a wide bridge or for a bridge with a severe skew angle.

Continuous or isolated footings may be specified for multi-column piers. The engineer should determine estimated costs for both footing configurations and choose the more economical configuration. Where the clear distance between isolated footings would be less than 4'-6", a continuous footing shall be specified.

A variation of the multi-column pier in Standard for Multi-Columned Pier is produced by omitting the cap and placing a column under each girder. This detail has been used for steel girders with girder spacing greater than 12'. This configuration is treated as a series of single column piers. The engineer shall consider any additional forces that may be induced in the superstructure cross frames at the pier if the pier cap is eliminated. The pier cap may not be eliminated for piers in the floodplain, or for continuous slab structures which need the cap to facilitate replacement of the slab during future rehabilitation.

See Standard for Highway Over Railroad Design Requirements for further details on piers supporting bridges over railways.

13.2.2 Pile Bents

Pile bents are most commonly used for small to intermediate stream crossings and are shown on the Standard for Pile Bent.

Pile bents shall not be used to support structures over roadways or railroads due to their susceptibility to severe damage should a vehicular collision occur.

For pile bents, pile sections shall be limited to 12¾” or 14” diameter cast-in-place reinforced concrete piles with steel shells spaced at a minimum center-to-center spacing of 3’. A minimum of five piles per pier shall be used on pile bents. When a satisfactory design cannot be developed with one of these pile sections at the minimum spacing, another pier type should be selected. The outside piles shall be battered 2” per foot, and the inside piles shall be driven vertically. WisDOT does not rely on the shell of CIP piles for capacity; therefore these piles are less of a concern for long term reduced capacity due to corrosion than steel H-piles. For that reason the BOS Development Chief must give approval for the use of steel H-piles in open pile bents.

Because of the minimum pile spacing, the superstructure type used with pile bents is generally limited to cast-in-place concrete slabs, prestressed girders and steel girders with spans under approx. 70’ and precast, prestressed box girders less than 21” in height.

To ensure that pile bents are capable of resisting the lateral forces resulting from floating ice and debris or expanding ice, the maximum distance from the top of the pier cap to the stable streambed elevation, including scour, is limited to:

- 15’ for 12¾” diameter piles (or 12” H-piles if exception is granted).
- 20’ for 14” diameter piles (or 14” H-piles if exception is granted).

Use of the pile values in Table 11.3-5 or Standard for Pile Details is valid for open pile bents due to the exposed portion of the pile being inspectable.

The minimum longitudinal reinforcing steel in cast-in-place piles with steel shells is 6-#7 bars in 12” piles and 8-#7 bars in 14” piles. The piles are designed as columns fixed from rotation in the plane of the pier at the top and at some point below streambed.

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

Pile bents shall meet the following criteria:

- If the water velocity, Q_{100}, is greater than 7 ft/sec, the quantity of the 100-year flood shall be less than 12,000 ft^3/sec.
- If the streambed consists of unstable material, the velocity of the 100-year flood shall not exceed 9 ft/sec.

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

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

13.2.3 Pile Encased Piers

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

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

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

Pile encased piers should not be used for normal water depths greater than 10', since this is the maximum practical depth for setting formwork and placing the reinforcing steel. Total pier height shall be less than 25 feet.

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

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

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

The concrete in the encasement wall may be placed under water if the procedure detailed in 13.14 is followed.

13.2.4 Solid Single Shaft / Hammerheads

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

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

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

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

13.2.5 Aesthetics

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

Refer to Chapter 4 for additional information about aesthetics.

13.3 Location

Piers shall be located to provide a minimum interference to flood flow. In general, place the piers parallel with the direction of flood flow. Make adequate provision for drift and ice by increasing span lengths and vertical clearances, and by selecting proper pier types. Special precautions against scour are required in unstable streambeds. Navigational clearance shall be considered when placing piers for bridges over navigable waterways. Coordination with the engineer performing the hydraulic analysis is required to ensure the design freeboard is met, the potential for scour is considered, the hydraulic opening is maintained and the flood elevations are not adversely affected upstream or downstream. Refer to Chapter 8 for further details.

In the case of railroad and highway separation structures, the spacing and location of piers and abutments is usually controlled by the minimum horizontal and vertical clearances required for the roadway or the railroad. Other factors such as utilities or environmental concerns may influence the location of the piers. Sight distance can impact the horizontal clearance required for bridges crossing roadways on horizontally curved alignments. Requirements for vertical and horizontal clearances are specified in Chapter 3 – Design Criteria. Crash wall requirements are provided on Standard for Highway Over Railroad Design Requirements.

Cost may also influence the number of piers, and therefore the number of spans, used in final design. During the planning stages, an analysis should be performed to determine the most economical configuration of span lengths versus number of piers that meet all of the bridge site criteria.

13.4 Loads on Piers

The following loads shall be considered in the design of piers. Also see 13.5 for additional guidance regarding load application.

13.4.1 Dead Loads

The dead load forces, DC and DW, acting on the piers shall include reactions from the superstructure. DC dead loads include structural components and nonstructural attachments, and DW dead loads include wearing surfaces and utilities. The pier diaphragm weight may be applied through the girders. Different load factors are applied to each of these dead load types.

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

13.4.2 Live Loads

The HL-93 live load shall be used for all new bridge designs and is placed in 12'-wide design lanes. If fewer lane loads are used than what the roadway width can accommodate, the loads shall be kept within their design lanes. The design lanes shall be positioned between the curbs, ignoring shoulders and medians, to maximize the effect being considered. Refer to 17.2.4.2 for a detailed description of the HL-93 live load. For pier design, particular attention should be given to the double truck load described in 17.2.4.2.4. This condition places two trucks, spaced a minimum of 50' apart, within one design lane and will often govern the maximum vertical reaction at the pier.

WisDOT policy items:

A 10 foot design lane width may be used for the distribution of live loads to a pier cap.

The dynamic load allowance shall be applied to the live load for all pier elements located above the ground line per **LRFD [3.6.2]**.

For girder type superstructures, the loads are transmitted to the pier through the girders. For pier design, simple beam distribution is used to distribute the live loads to the girders. The wheel and lane loads are therefore transversely distributed to the girders by the lever rule as opposed to the Distribution Factor Method specified in **LRFD [4.6.2.2.2]**. The lever rule linearly distributes a portion of the wheel load to a particular girder based upon the girder spacing and the distance from the girder to the wheel load. The skew of the structure is not considered when calculating these girder reactions. Refer to 17.2.10 for additional information about live load distribution to the substructure and to Figure 17.2-17 for application of the lever rule.

For slab type superstructures, the loads are assumed to be transmitted directly to the pier without any transverse distribution. This assumption is used even if the pier cap is not integral with the superstructure. The HL-93 live load is applied as concentrated wheel loads combined with a uniform lane load. The skew of the structure is considered when applying

WisDOT policy item:

Since the angle of inclination of the pier nose with respect to the vertical is always less than or equal to 15° on standard piers in Wisconsin, the flexural ice failure mode does not need to be considered for these standard piers ($f_b = 0$).

WisDOT policy item:

If the pier is approximately aligned with the direction of the ice flow, only the first design case as specified in **LRFD [3.9.2.4]** shall be investigated due to the unknowns associated with the friction angle defined in the second design case.

A longitudinal force equal to F shall be combined with a transverse force of 0.15F

Both the longitudinal and transverse forces act simultaneously at the pier nose.

If the pier is located such that its longitudinal axis is skewed to the direction of the ice flow, the ice force on the pier shall be applied to the projected pier width and resolved into components. In this condition, the transverse force to the longitudinal axis of the pier shall be a minimum of 20% of the total force.

WisDOT exception to AASHTO:

Based upon the pier geometry in the Standards, the ice loadings of **LRFD [3.9.4]** and **LRFD [3.9.5]** shall be ignored.

13.4.8.2 Force Exerted by Expanding Ice Sheet

Expansion of an ice sheet, resulting from a temperature rise after a cold wave, can develop considerable force against abutting structures. This force can result if the sheet is restrained between two adjacent bridge piers or between a bluff type shore and bridge pier. The force direction is therefore transverse to the direction of stream flow.

Force from ice sheets depends upon ice thickness, maximum rate of air-temperature rise, extent of restraint of ice and extent of exposure to solar radiation. In the absence of more precise information, estimate an ice thickness and use a force of 8.0 ksf.

It is not necessary to design all bridge piers for expanding ice. If one side of a pier is exposed to sunlight and the other side is in the shade along with the shore in the pier vicinity, consider the development of pressure from expanding ice. If the central part of the ice is exposed to the sun's radiation, consider the effect of solar energy, which causes the ice to expand.

13.4.9 Centrifugal Force

Centrifugal force, CE, is specified in **LRFD [3.6.3]** and is included in the pier design for structures on horizontal curves. The lane load portion of the HL-93 loading is neglected in the computation of the centrifugal force.

The centrifugal force is taken as the product of the axle weights of the design truck or tandem and the factor, C, given by the following equation:

$$C = \frac{4}{3}\frac{v^2}{gR}$$

Where:

V = Highway design speed (ft/sec)

g = Gravitational acceleration = 32.2 (ft/sec^2)

R = Radius of curvature of travel lane (ft)

The multiple presence factors specified in **LRFD [3.6.1.1.2]** shall apply to centrifugal force.

Centrifugal force is assumed to act radially and horizontally 6' above the roadway surface. The point 6' above the roadway surface is measured from the centerline of roadway. The design speed may be determined from the Wisconsin *Facilities Development Manual*, Chapter 11. It is not necessary to consider the effect of superelevation when centrifugal force is used, because the centrifugal force application point considers superelevation.

13.4.10 Extreme Event Collision Loads

WisDOT policy item:

With regards to **LRFD [3.6.5]** and vehicular collision force, CT, protecting the pier and designing the pier for the 600 kip static force are each equally acceptable. The bridge design engineer should work with the roadway engineer to determine which alternative is preferred.

WisDOT policy item:

Designs for bridge piers adjacent to roadways with a design speed ≤ 40 mph need not consider the provisions of **LRFD [3.6.5]**. As for all multi-columned piers, a minimum of 3 columns is still required.

If the design speed of a roadway adjacent to a pier is > 40 mph and the pier is not protected by a TL-5 barrier, embankment or adequate offset, the pier columns/shaft, *only*, shall be strengthened to comply with **LRFD [3.6.5]**. For a multi-column pier the minimum size column shall be 3x4 ft rectangular or 4 ft diameter (consider clearance issues and/or the wide cap required when using 4 ft diameter columns). Solid shaft and hammerhead pier shafts are considered adequately sized.

The vertical reinforcement for the columns/shaft shall be the greater of what is required by design (not including the Extreme Event II loading) or a minimum of 1.0% of the gross concrete section (total cross section without deduction for rustications less than or equal to 1-1/2" deep) to address the collision force for the 3x4 ft rectangular and 4 ft diameter columns.

For the 3x4 ft rectangular columns, use double #5 stirrups spaced at 6" vertically as a minimum. For the 4 ft diameter columns, use #4 spiral reinforcement (smooth bars) spaced vertically at 6" as a minimum. Hammerhead pier shafts shall have, as a minimum, the horizontal reinforcement as shown on the Standards.

See Standard for Multi-Columned Pier with Rectangular Columns for an acceptable design to meet **LRFD [3.6.5]**.

WisDOT exception to AASHTO:

The vessel collision load, CV, in **LRFD [3.14]** will not be applied to every navigable waterway of depths greater than 2'. For piers located in navigable waterways, the engineer shall contact the WisDOT project manager to determine if a vessel collision load is applicable.

13.5 Load Application

When determining pier design forces, a thorough understanding of the load paths for each load is critical to arriving at loads that are reasonable per *AASHTO LRFD*. The assumptions associated with different pier, bearing and superstructure configurations are also important to understand. This section provides general guidelines for the application of forces to typical highway bridge piers.

13.5.1 Loading Combinations

Piers are designed for the Strength I, Strength III, Strength V and Extreme Event II load combinations as specified in **LRFD [3.4.1]**. Reinforced concrete pier components are also checked for the Service I load combination. Load factors for these load combinations are presented in Table 13.5-1. See 13.10 for loads applicable to pile bents and pile encased piers.

Load Combination Limit State	Load Factor										
	DC		DW		LL+IM BR CE	WA	WS	WL	FR	TU CR SH	IC CT CV
	Max.	Min.	Max.	Min.							
Strength I	1.25	0.90	1.50	0.65	1.75	1.00	0.00	0.00	1.00	0.50	0.00
Strength III	1.25	0.90	1.50	0.65	0.00	1.00	1.40	0.00	1.00	0.50	0.00
Strength V	1.25	0.90	1.50	0.65	1.35	1.00	0.40	1.00	1.00	0.50	0.00
Service I	1.00	1.00	1.00	1.00	1.00	1.00	0.30	1.00	1.00	1.00	0.00
Extreme Event II	1.25	0.90	1.50	0.65	0.50	1.00	0.00	0.00	1.00	0.00	1.00

Table 13.5-1
Load Factors

13.5.2 Expansion Piers

See 13.4 for additional guidance regarding the application of specific loads.

Transverse forces applied to expansion piers from the superstructure include loads from one-half of the adjacent span lengths, and are applied at the location of the transverse load.

For expansion bearings other than elastomeric, longitudinal forces are transmitted to expansion piers through friction in the bearings. These forces, other than temperature, are based on loading one-half of the adjacent span lengths, with the maximum being no greater than the maximum friction force (dead load times the maximum friction coefficient of a sliding bearing). See 27.2.2 to determine the bearing friction coefficient. The longitudinal forces are applied at the bearing elevation.

13.7.4 Design Tension Tie Reinforcement

Tension ties shall be designed to resist the strength limit state force per **LRFD [5.6.3.4.1]**. For non-prestressed caps, the tension tie steel shall satisfy:

$$A_{st} \geq \frac{P_u}{\phi f_y}$$

Where:

A_{st} = Total area of mild steel reinforcement in the tie (in^2)

P_u = Tension tie force from strength limit state (kips)

ϕ = Resistance factor for tension on reinforced concrete, equal to 0.90, as specified in **LRFD [5.5.4.2]** (dimensionless)

f_y = Yield strength of reinforcement (ksi)

Horizontal tension ties, such as ties A-B and E-F in Figure 13.7-2, are used to determine the longitudinal reinforcement required in the top of the pier cap. The maximum tension tie value should be used to calculate the top longitudinal reinforcement.

Vertical tension ties, such as ties B-H and C-I, are used to determine the vertical stirrup requirements in the cap. Similar to traditional shear design, two stirrup legs shall be accounted for when computing A_{st}. In Figure 13.7-2, the number of stirrups, n, necessary to provide the A_{st} required for tie B-H shall be spread out across Stirrup Region 2. The length limits of Stirrup Region 2 are from the midpoint between nodes A and B to the midpoint between nodes B and C. When vertical ties are located adjacent to columns, such as with tie C-I, the stirrup region extends to the column face. Therefore, the length limits of Stirrup Region 1 are from the column face to the midpoint between nodes B and C. The stirrup spacing shall then be determined by the following equation:

$$s_{max} = \frac{L}{n}$$

Where:

s_{max} = Maximum allowable stirrup spacing (in)

L = Length of stirrup region (in)

n = Number of stirrups required to satisfy the A_{st} required to resist the vertical tension tie force

Skin reinforcement on the side of the cap, shall be determined as per **LRFD [5.7.3.4]**. This reinforcement shall not be included in any strength calculations.

13.7.5 Check the Compression Strut Capacity

Compression struts shall be designed to resist the strength limit state force per **LRFD [5.6.3.3]**. The resistance of an unreinforced compression strut shall be taken as:

$$P_r = \phi f_{cu} A_{cs} \geq P_u$$

Where:

P_r	=	Factored resistance of compression strut (kips)
P_u	=	Compression strut force from strength limit state (kips)
ϕ	=	Resistance factor for compression in strut-and-tie models, equal to 0.70, as specified in **LRFD [5.5.4.2]** (dimensionless)
f_{cu}	=	Limiting compressive stress (ksi)
A_{cs}	=	Effective cross-sectional area of strut (in^2)

The limiting compressive stress shall be given by:

$$f_{cu} = \frac{f'_c}{0.8 + 170\varepsilon_1} \leq 0.85f'_c$$

In which:

$$\varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002)\cot^2 \alpha_s$$

Where:

ε_s	=	Concrete tensile strain in the direction of the tension tie at the strength limit state (in/in)
α_s	=	Smallest angle between the compression strut and the adjoining tension ties (°)
f'_c	=	Specified compressive strength (ksi)

The concrete tensile strain is given by:

$$\varepsilon_s = \frac{P_u}{A_{st}E_s}$$

Where:

E_s = Modulus of elasticity of steel, taken as 29,000 (ksi)

The cross-sectional area of the strut, A_{cs}, is determined by considering both the available concrete area and the anchorage conditions at the end of the strut. Figure 13.7-3, Figure 13.7-4 and Figure 13.7-5 illustrate the computation of A_{cs}.

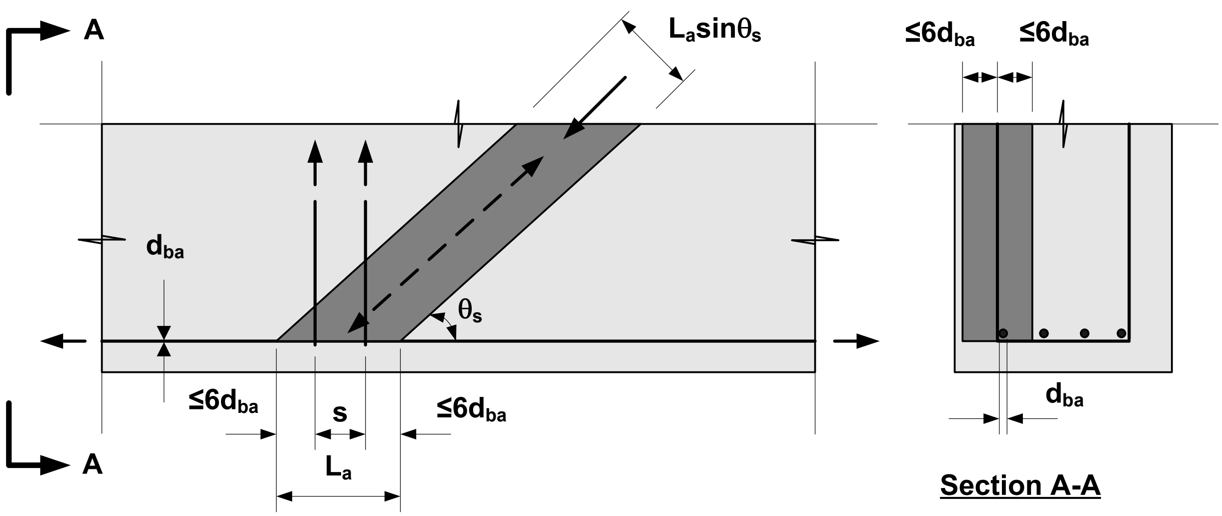


Figure 13.7-3
Strut Anchored by Tension Reinforcement Only

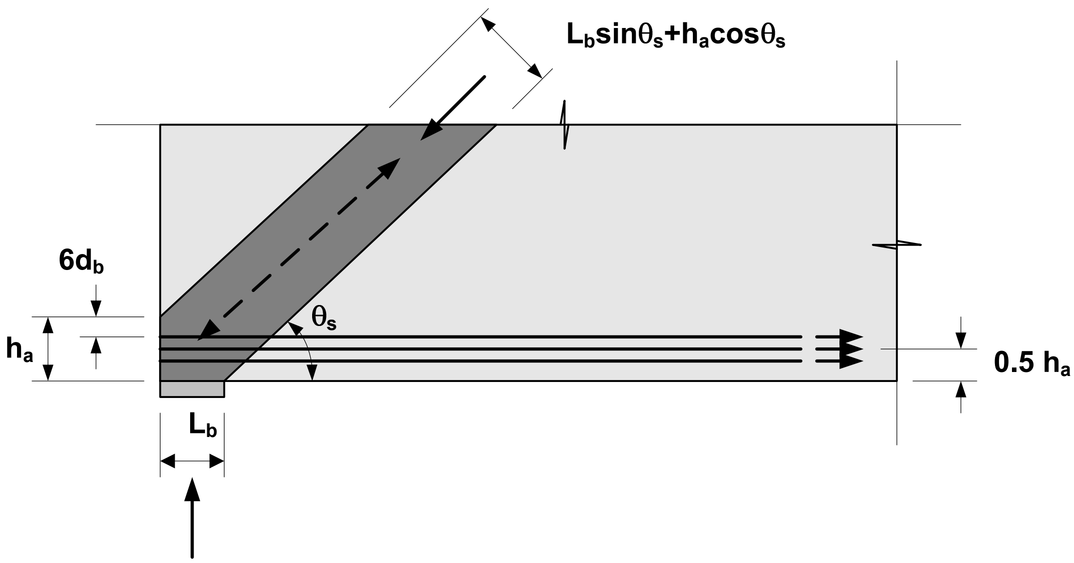


Figure 13.7-4
Strut Anchored by Bearing and Tension Reinforcement

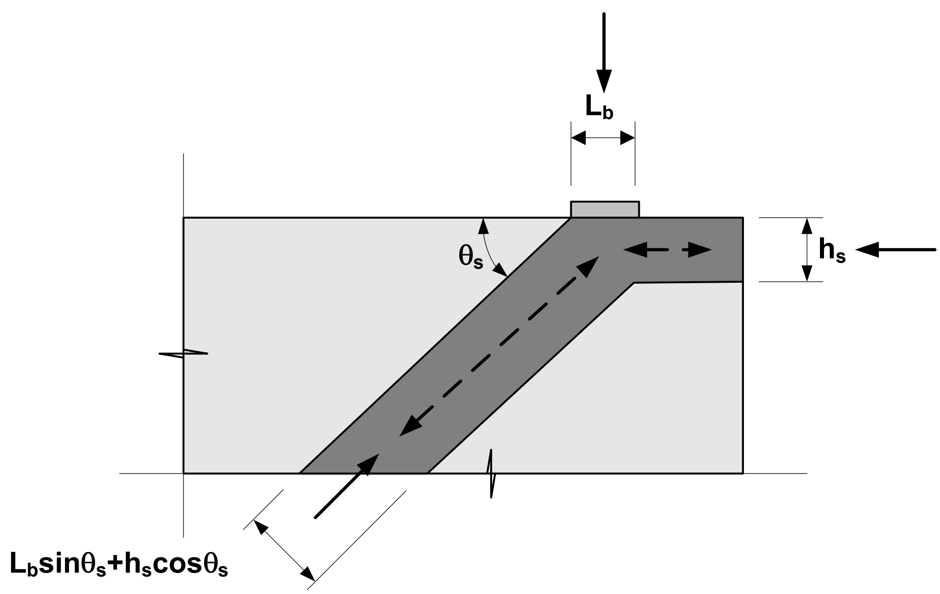


Figure 13.7-5
Strut Anchored by Bearing and Strut

In Figure 13.7-3, the strut area is influenced by the stirrup spacing, s, as well as the diameter of the longitudinal tension steel, d_{ba}. In Figure 13.7-4, the strut area is influenced by the bearing dimensions, L_b, in both directions, as well as the location of the center of gravity of the longitudinal tension steel, $0.5h_a$. In Figure 13.7-5, the strut area is influenced by the bearing dimensions, L_b, in both directions, as well as the height of the compression strut, h_s. The value of h_s shall be taken as equal to "a" as shown in Figure 13.7-2. The strut area in each of the three previous figures depends upon the angle of the strut with respect to the horizontal, θ_s.

If the initial strut width is inadequate to develop the required resistance, the engineer should increase the bearing block size.

13.7.6 Check the Tension Tie Anchorage

Tension ties shall be anchored to the nodal zones by either specified embedment length or hooks so that the tension force may be transferred to the nodal zone. As specified in **LRFD [5.6.3.4]**, the tie reinforcement shall be fully developed at the inner face of the nodal zone. In Figure 13.7-4, this location is given by the edge of the bearing where θ_s is shown.

13.7.7 Provide Crack Control Reinforcement

Pier caps designed using the strut-and-tie method shall contain an orthogonal grid of reinforcing bars near each face in accordance with **LRFD [5.6.3.6]**. This reinforcement will control crack widths and ensure a minimum ductility. The ratio of reinforcement area to gross concrete area shall not be less than 0.003 in both directions. Maximum bar spacing shall not exceed 12". The crack control steel, when located within the tension tie, may be considered as part of the tension tie reinforcement.

13.8 General Pier Cap Information

The minimum cap dimension to be used is 3’ deep by 2’-6” wide, with the exception that a 2’-6” deep section may be used for caps under slab structures. If a larger cap is needed, use 6” increments to increase the size. The multi-column cap width shall be a minimum of 1 1/2" wider than the column on each side to facilitate construction forming. The pier cap width shall extend a minimum of 2’ transversely beyond the centerline of bearing and centerline of girder intersection.

On continuous slab structures, the moment and shear forces are proportional between the transverse slab section and the cap by the ratio of their moments of inertia. The effective slab width assumed for the transverse beam is the minimum of 1/2 the center-to-center column spacing or 8.0’.

$$M_{cap} = M_{total} \frac{I_{cap}}{I_{cap} + I_{slab}}$$

Where:

M_{cap}	=	Cap moment (kip-ft)
M_{total}	=	Total moment (kip-ft)
I_{cap}	=	Moment of inertia of pier cap (in^4)
I_{slab}	=	Moment of inertia of slab (in^4)

The concrete slab is to extend beyond the edge of pier cap as shown on Standards for Continuous Haunched Slab and for Continuous Flat Slab. If the cap is rounded, measure from a line tangent to the pier cap end and parallel to the edge of the deck.

Reinforcement bars are placed straight in the pier cap. Determine bar cutoff points on wide caps. If the pier cap is cantilevered over exterior columns, the top negative bar steel may be bent down at the ends to ensure development of this primary reinforcement.

Do not place shear stirrups closer than 4” on centers. Generally only double stirrups are used, but triple stirrups may be used to increase the spacing. If these methods do not work, increase the cap size. Stirrups are generally not placed over the columns. The first stirrup is placed one-half of the stirrup spacing from the edge of the column into the span.

The cap-to-column connection is made by extending the column reinforcement straight into the cap the necessary development length. Stirrup details and bar details at the end of the cap are shown on Standard for Multi-Columned Pier.

13.9 Column / Shaft Design

See 13.4.10 for minimum shaft design requirements regarding the Extreme Event II collision load of **LRFD [3.6.5]**.

Use an accepted analysis procedure to determine the axial load as well as the longitudinal and transverse moments acting on the column. These forces are generally largest at the top and bottom of the column. Apply the load factors for each applicable limit state. The load factors should correspond to the gross moment of inertia. Load factors vary for the gross moment of inertia versus the cracked moment as defined in **LRFD [3.4.1]** for γ_{TU}, γ_{CR}, γ_{SH}. Choose the controlling load combinations for the column design.

Columns that are part of a pier frame have transverse moments induced by frame action from vertical loads, wind loads on the superstructure and substructure, wind loads on live load, thermal forces and centrifugal forces. If applicable, the load combination for Extreme Event II must be considered. Longitudinal moments are produced by the above forces, as well as the braking force. These forces are resolved through the skew angle of the pier to act transversely and longitudinally to the pier frame. Longitudinal forces are divided equally among the columns.

Wisconsin uses tied columns following the procedures of **LRFD [5.7.4]**. The minimum allowable column size is 2'-6" in diameter. The minimum steel bar area is as specified in **LRFD [5.7.4.2]**. For piers not requiring a certain percentage of reinforcement as per 13.4.10 to satisfy **LRFD [3.6.5]** for vehicular collision load, a reduced effective area of reinforcement may be used when the cross-section is larger than that required to resist the applied loading.

The computed column moments are to consider moment magnification factors for slenderness effects as specified in **LRFD [5.7.4.3]**. Values for the effective length factor, K, are as follows:

- 1.2 for longitudinal moments with a fixed seat supporting prestressed concrete girders
- 2.1 for longitudinal moments with a fixed seat supporting steel girders and all expansion bearings
- 1.0 for all transverse moments

The computed moments are multiplied by the moment magnification factors, if applicable, and the column is designed for the combined effects of axial load and bending. According to **LRFD [5.7.4.1]** all force effects, including magnified moments, shall be transferred to adjacent components. The design resistance under combined axial load and bending is based on stress-strain compatibility. A computer program is recommended for determining the column's resistance to the limit state loads.

As a minimum, the column shall provide the steel shown on the Pier Standards.

On large river crossings, it may be necessary to protect the piers from damage. Dolphins may be provided.

The column-to-cap connection is designed as a rigid joint considering axial and bending stresses. Column steel is run through the joint into the cap to develop the compressive stresses or the tensile steel stresses at the joint.

In general, the column-to-footing connection is also designed as a rigid joint. The bar steel from the column is generally terminated at the top of the footing. Dowel bars placed in the footing are used to transfer the steel stress between the footing and the column.

13.9.1 Tapered Columns of Concrete and Timber

Design tapered columns of concrete and timber using the existing column formulas, taking the cross-sectional area at the small end. However, d, as used in L/d, is taken as follows:

1. For round columns or rectangular columns tapered in both directions, use:

$$d = d_B$$

2. For rectangular columns tapered in the plane of bending only, use:

$$d = (d_A)2(d_B)8$$

3. For rectangular columns tapered perpendicular to the plane of bending, use:

$$d = (d_A)7(d_B)3$$

Where:

d_A = Dimension at the small end

d_B = Dimension at the large end

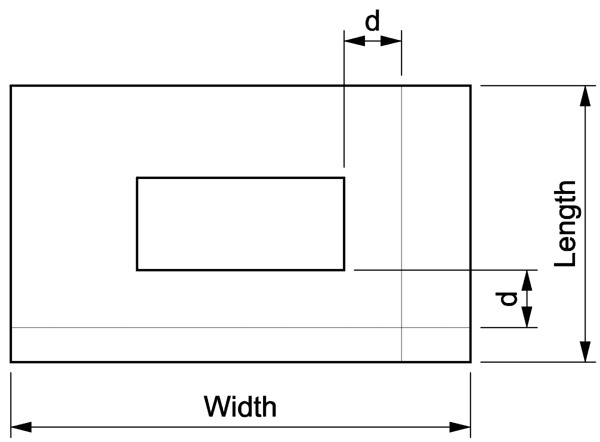


Figure 13.11-2
Shear Location for One-Way Action

The footing weight and the soil above the areas are used to reduce the shear force.

9. The bottom layer of reinforcing steel is placed 3” clear from the bottom of the footing.

10. If adjacent edges of isolated footings are closer than 4’-6”, a continuous footing shall be used.

13.11.3 Isolated Pile Footings

WisDOT policy item:

Pile footings are designed using LRFD strength limit state loads and resistance for moment and shear as specified in **LRFD [5.13.3]**. The pile design shall use LRFD strength limit state loads to compare to the factored axial compression resistance specified in Table 11.3-5.

The nominal geotechnical pile resistance shall be provided in the Site Investigation Report. The engineer shall then apply the appropriate resistance factor from Table 11.3.1 to the nominal resistance to determine the factored pile resistance. The footing is proportioned so that when it is loaded with the strength limit state loads, the factored pile resistance is not exceeded.

The following steps are used to design pile-supported footings:

1. The minimum depth of pile footing is 2’-6”. The minimum pile embedment is 6”. See 13.2.2 for additional information about pile footings used for pile bents.

2. Pile footings in uplift are usually designed by method (a) stated below. However, method (b) may be used if there is a substantial cost reduction.

a. Over one-half of the piles in the footings must be in compression for the Strength limit states. The section properties used in analysis are based only on the piles in compression. Pile and footing (pile cap) design is based on the Strength limit states. Service limit states require check for overall stability; however a check of crack control is not required per 13.11. The 600 kip collision load need not be checked per 13.4.10.

b. Piles may be designed for upward forces provided an anchorage device, sufficient to transfer the load, is provided at the top of the pile. Provide reinforcing steel to resist the tension stresses at the top of the footing.

3. Same as spread footing.

4. Same as spread footing.

5. The minimum number of piles per footing is four.

6. Pile footings on seals are analyzed above the seal. The only effect of the seal is to reduce the pile resistance above the seal by the portion of the seal weight carried by each pile.

7. If no seal is required but a cofferdam is required, design the piles to use the minimum required batter. This reduces the cofferdam size necessary to clear the battered piles since all piles extend above water to the pile driver during driving.

8. The pile footing reinforcing steel is determined from the flexural requirements of **LRFD [5.7.3]**. The design moment and shear are determined from the force of the piles which act outside of the section being considered. The weight of the footing and the soil above the footing are used to reduce the magnitude of the bending moment and shear force.

9. Shear resistance is determined by the following two methods:

a. Two-way action

The summation of the pile forces outside the critical perimeter lines placed at a distance d/2 from the face of the column (where d equals the effective footing depth) determines the shear force. The shear resistance is influenced by the concrete strength, the footing depth and the critical perimeter length. The critical perimeter length is 2 (L + d + W + d) for rectangular columns and π(2R + d) for round columns. The critical perimeter location for pile footings with rectangular columns is illustrated in Figure 13.11-3.

Table of Contents

E13-1 Hammerhead Pier Design Example

This example shows design calculations conforming to the ***AASHTO LRFD Bridge Design Specifications (Sixth Edition - 2012)*** as supplemented by the *WisDOT Bridge Manual (July 2012)*. The design methods presented throughout the example are meant to be the most widely used in general bridge engineering practice.

The first design step is to identify the appropriate design criteria. This includes, but is not limited to, defining material properties, identifying relevant superstructure information, and determining the required pier geometry.

E13-1.1 Obtain Design Criteria

This pier is designed for the superstructure as detailed in example **E24-1**. This is a two-span steel girder stream crossing structure. Expansion bearings are located at the abutments, and fixed bearings are used at the pier.

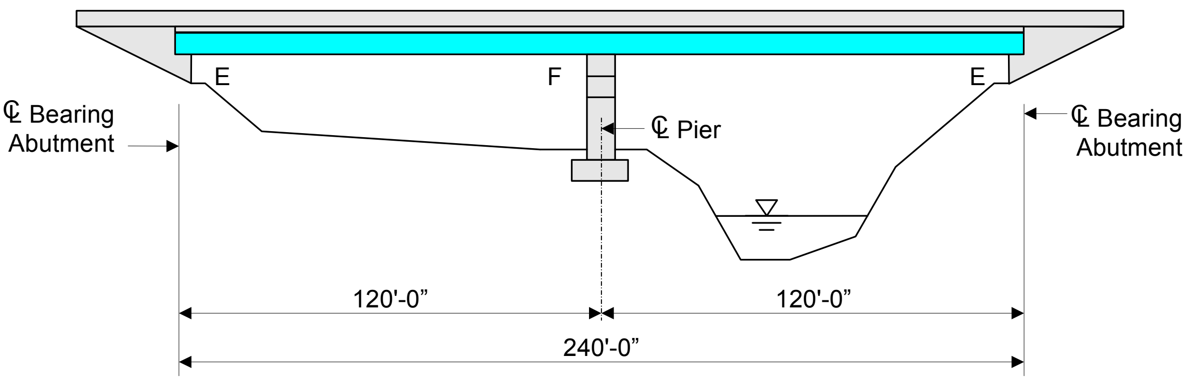


Figure E13-1.1-1
Bridge Elevation

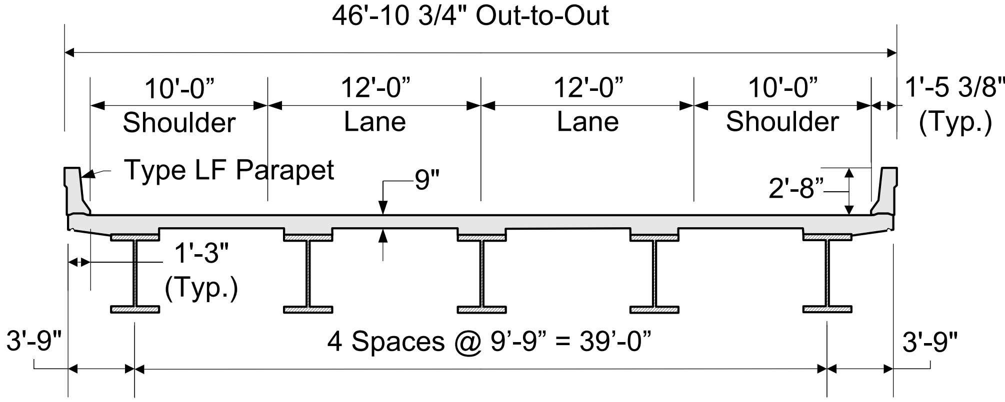


Figure E13-1.1-2
Bridge Cross Section

$R_{xx} := \dfrac{Pu_{Mom\_xx}}{L_{ftg\_xx}}$ $\boxed{R_{xx} = 57.43}$ kips per foot

Estimation of applied factored load per foot in the "Y" direction:

$Pu_{Mom\_yy} := Pu \cdot 4$ $\boxed{Pu_{Mom\_yy} = 1056.79}$ kips

$R_{yy} := \dfrac{Pu_{Mom\_yy}}{L_{ftg\_yy}}$ $\boxed{R_{yy} = 88.07}$ kips per foot

$arm_{xx} := 2.5$ feet

$arm_{yy} := 2.25$ feet

The moment on a per foot basis is then:

$Mu_{xx} := R_{xx} \cdot arm_{xx}$ $\boxed{Mu_{xx} = 143.59}$ kip-ft per foot

$Mu_{yy} := R_{yy} \cdot arm_{yy}$ $\boxed{Mu_{yy} = 198.15}$ kip-ft per foot

Once the maximum moment at the critical section is known, flexure reinforcement must be determined. The footing flexure reinforcement is located in the bottom of the footing and rests on top of the piles.

Assume #8 bars:

$bar\_diam8 := 1.0$ inches

$bar\_area8 := 0.79$ in^2

$\boxed{f_y = 60}$ ksi

The footing minimum tensile reinforcement requirements will be calculated. The tensile reinforcement provided must be enough to develop a factored flexural resistance at least equal to the lesser of the cracking strength or 1.33 times the factored moment from the applicable strength load combinations, **LRFD [5.7.3.3.2]**.

The cracking strength is calculated as follows, **LRFD[5.7.3.3.2]**:

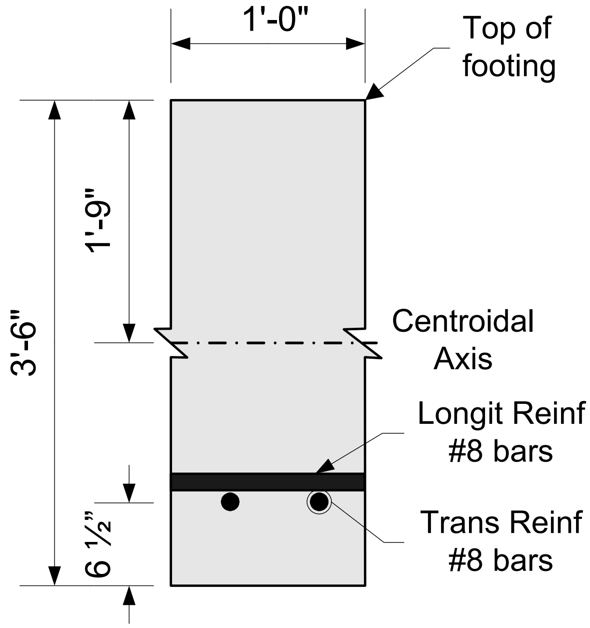


Figure E13-1.11-1
Footing Cracking Moment Dimensions

$f_r := 0.37 \cdot \sqrt{f'_c}$ $\boxed{f_r = 0.69}$ ksi

$S_g := \frac{b\left(H_{ftg} \cdot 12\right)^2}{6}$ $\boxed{S_g = 3528}$ in^4

$y_t := \frac{H_{ftg} \cdot 12}{2}$ $\boxed{y_t = 21}$ in

$M_{cr} = \gamma_3\left(\gamma_1 \cdot f_r\right) S_g$ therefore, $M_{cr} = 1.1\left(f_r\right) S_g$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$M_{cr} := 1.1 f_r \cdot S_g \cdot \frac{1}{12}$ $\boxed{M_{cr} = 223.86}$ kip-ft

1.33 times the factored controlling footing moment is:

$Mu_{ftg} := \max(Mu_{xx}, Mu_{yy})$ $\boxed{Mu_{ftg} = 198.15}$ kip-ft

$\boxed{1.33 \cdot Mu_{ftg} = 263.54}$ kip-ft

$M_{Design} := \min(M_{cr}, 1.33 \cdot Mu_{ftg})$ $\boxed{M_{Design} = 223.86}$ kip-ft

Since the transverse moment controlled, M_{yy}, detail the transverse reinforcing to be located directly on top of the piles.

Effective depth, d_e = total footing thickness - cover - 1/2 bar diameter

$$d_e := H_{ftg} \cdot 12 - Cover_{fb} - \frac{bar\_diam8}{2}$$ $\boxed{d_e = 35.5}$ in

Solve for the required amount of reinforcing steel, as follows:

$\phi_f := 0.90$

$\boxed{b = 12}$ in

$\boxed{f'_c = 3.5}$ ksi

$$Rn := \frac{M_{Design} \cdot 12}{\phi_f \cdot b \cdot d_e^2}$$ $\boxed{Rn = 0.197}$

$$\rho := 0.85 \left(\frac{f'_c}{f_y}\right) \cdot \left(1.0 - \sqrt{1.0 - \frac{2 \cdot Rn}{0.85 \cdot f'_c}}\right)$$ $\boxed{\rho = 0.00341}$

$As_{ftg} := \rho \cdot b \cdot d_e$ $\boxed{As_{ftg} = 1.45}$ in^2 per foot

Required bar spacing = $\boxed{\frac{bar\_area8}{As_{ftg}} \cdot 12 = 6.53}$ in

Use #8 bars @ $bar\_space := 6$

$$A_{sftg} := bar\_area8 \cdot \left(\frac{12}{bar\_space}\right)$$ $\boxed{A_{sftg} = 1.58}$ in^2 per foot

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

Similar calculations can be performed for the reinforcing in the longitudinal direction. The effective depth for this reinforcing is calculated based on the longitudinal bars resting directly on top of the transverse bars.

E13-1.11.2 Punching Shear Check

The factored force effects from E13-1.7 for the punching shear check at the column are:

$\boxed{Pu3_{ftgStr1} = 3124.66}$ kips

$\boxed{MuT3_{ftgStr1} = 4541.55}$ kip-ft

$\boxed{MuL3_{ftgStr1\delta} = 2467.46}$ kip-ft

$$Pu3 = \begin{pmatrix} 251 & 228.29 & 205.58 & 182.87 & 160.17 \\ 218.1 & 195.39 & 172.68 & 149.97 & 127.27 \\ 185.2 & 162.49 & 139.78 & 117.08 & 94.37 \\ 152.3 & 129.59 & 106.88 & 84.18 & 61.47 \end{pmatrix}$$

$\boxed{Pu3_{pile} = 251}$ kips

With the applied factored loads determined, the next step in the column punching shear check is to define the critical perimeter, b_o. The Specifications require that this perimeter be minimized, but need not be closer than $d_v/2$ to the perimeter of the concentrated load area. In this case, the concentrated load area is the area of the column on the footing as seen in plan.

The effective shear depth, d_v, must be defined in order to determine b_o and the punching (or two-way) shear resistance. An average effective shear depth should be used since the two-way shear area includes both the "X-X" and "Y-Y" sides of the footing. In other words, d_{ex} is not equal to d_{ey}, therefore d_{vx} will not be equal to d_{vy}. This is illustrated as follows assuming a 3'-6" footing with #8 reinforcing bars at 6" on center in both directions in the bottom of the footing:

$\boxed{b = 12}$ in

$h_{ftg} := H_{ftg} \cdot 12$ $\qquad \boxed{h_{ftg} = 42}$ in

$A_{s\_ftg} := 2 \cdot (bar\_area8)$ $\qquad \boxed{A_{s\_ftg} = 1.58}$ in² per foot width

Effective depth for each axis:

$\boxed{Cover_{fb} = 6}$

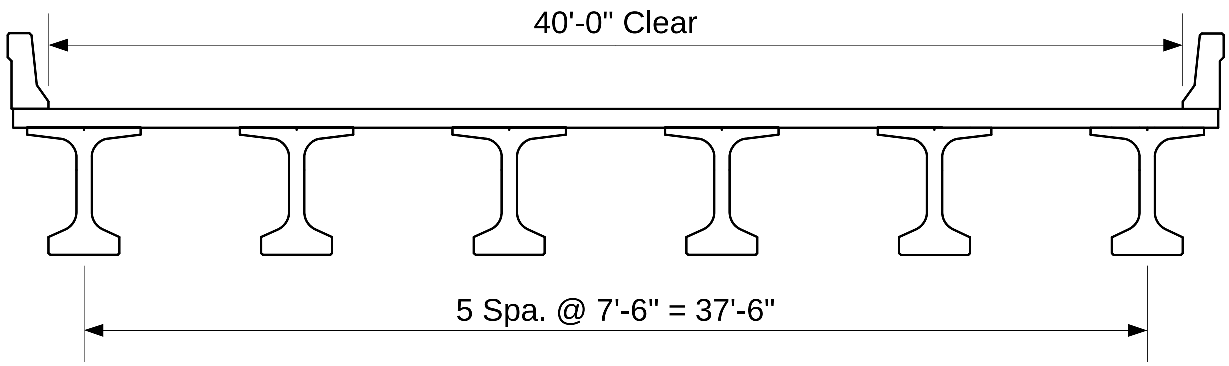
$$d_{ey} := h_{ftg} - Cover_{fb} - \frac{bar\_diam8}{2} \qquad \boxed{d_{ey} = 35.5} \text{ in}$$

$$d_{ex} := h_{ftg} - Cover_{fb} - bar\_diam8 - \frac{bar\_diam8}{2} \qquad \boxed{d_{ex} = 34.5} \text{ in}$$

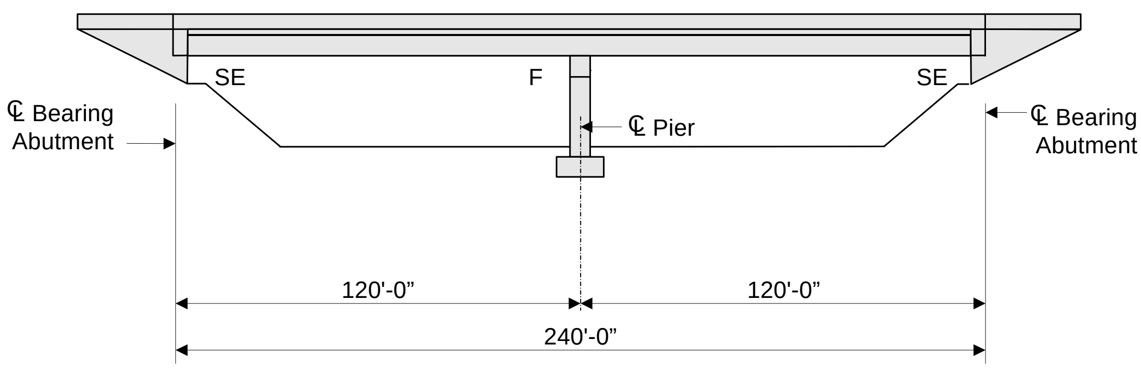
Table of Contents

E13-2 Multi-Column Pier Design Example - LRFD

2 Span Bridge, 54W, LRFD Design



This pier is designed for the superstructure as detailed in example **E19-2**. This is a two-span prestressed girder grade separation structure. Semi-expansion bearings are located at the abutments, and fixed bearings are used at the pier.



E13-2.1 Obtain Design Criteria

This multi-column pier design example is based on ***AASHTO LRFD Bridge Design Specifications, (Sixth Edition - 2012).*** The design methods presented throughout the example are meant to be the most widely used in general bridge engineering practice. Calculations are only shown for the pier cap. For example column and footing calculations, see example E13-1.

The first design step is to identify the appropriate design criteria. This includes, but is not limited to, defining material properties, identifying relevant superstructure information, and determining the required pier geometry.

E13-2.1.1 Material Properties:

$w_c := 0.150$ Concrete density, kcf

E13-2.6 Pier Cap Design

E13-2.6.1 Positive Moment Capacity Between Columns

It is assumed that there will be two layers of positive moment reinforcement. Therefore the effective depth of the section at the pier is:

$cover := 2.5$ in

In accordance with **LRFD [5.10.3.1.3]** the minimum clear space between the bars in layers is one inch or the nominal diameter of the bars.

$spa_{clear} := 1.75$ in

$bar_{stirrup} := 5$ (transverse bar size)

$\boxed{Bar_D(bar_{stirrup}) = 0.63}$ in (transverse bar diameter)

$Bar_{No\_pos} := 9$

$\boxed{Bar_D(Bar_{No\_pos}) = 1.13}$ in (Assumed bar size)

$$d_e := cap_H \cdot 12 - cover - Bar_D(bar_{stirrup}) - Bar_D(Bar_{No\_pos}) - \frac{spa_{clear}}{2}$$

$\boxed{d_e = 42.87}$ in

For flexure in non-prestressed concrete, $\phi_f := 0.9$.

The width of the cap:

$b_w := cap_W \cdot 12$ $\boxed{b_w = 42}$ in

$\boxed{Mu_{pos} = 2372}$ kip-ft

$$R_u := \frac{Mu_{pos} \cdot 12}{\phi_f \cdot b_w \cdot d_e^2}$$ $\boxed{R_u = 0.4097}$ ksi

$$\rho := 0.85 \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R_u}{0.85 \cdot f'_c}}\right)$$ $\boxed{\rho = 0.00738}$

$A_s := \rho \cdot b_w \cdot d_e$ $\boxed{A_s = 13.28}$ in²

This requires $n_{bars\_pos} := 14$ bars. Use $n_{bars\_pos1} := 9$ bars in the bottom layer and $n_{bars\_pos2} := 5$ bars in the top layer. Check spacing requirements.

$$spa_{pos} := \frac{b_w - 2 \cdot (cover - Bar_D(bar_{stirrup})) - Bar_D(Bar_{No\_pos})}{n_{bars\_pos1} - 1}$$

$\boxed{spa_{pos} = 4.64}$ in

$clear_{spa} := spa_{pos} - Bar_D(Bar_{No\_pos})$ $\boxed{clear_{spa} = 3.51}$ in

The minimum clear spacing is equal to 1.5 times the maximum aggregate size of 1.5 inches.

$spa_{min} := 1.5 \cdot 1.5$ $\boxed{spa_{min} = 2.25}$ in

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

$As_{prov\_pos} := Bar_A(Bar_{No\_pos}) \cdot n_{bars\_pos}$ $\boxed{As_{prov\_pos} = 14}$ in^2

$a := \frac{As_{prov\_pos} \cdot f_y}{0.85 \cdot b_w \cdot f'_c}$ $\boxed{a = 6.72}$ in

$Mn_{pos} := As_{prov\_pos} \cdot f_y \cdot \left(d_e - \frac{a}{2}\right) \cdot \frac{1}{12}$ $\boxed{Mn_{pos} = 2766}$ kip-ft

$Mr_{pos} := \phi_f \cdot Mn_{pos}$ $\boxed{Mr_{pos} = 2489}$ kip-ft

$\boxed{Mu_{pos} = 2372}$ kip-ft

Is M_u less than M_r? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$S_{cap} := \frac{(cap_W \cdot 12) \cdot (cap_H \cdot 12)^2}{6}$ $\boxed{S_{cap} = 16128}$ in^3

$f_r := 0.37 \cdot \sqrt{f'_c}$ $\boxed{f_r = 0.69}$ ksi

$M_{cr} = \gamma_3(\gamma_1 \cdot f_r) S_{cap}$ therefore, $M_{cr} = 1.1(f_r) S_{cap}$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$M_{cr} := 1.1 \cdot f_r \cdot S_{cap} \cdot \frac{1}{12}$ $\boxed{M_{cr} = 1023}$ kip-ft

$\boxed{1.33 \cdot Mu_{pos} = 3155}$ kip-ft

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

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$\rho := \frac{As_{prov\_pos}}{b_w \cdot d_e}$ $\boxed{\rho = 0.00778}$

$n := \text{floor}\left(\frac{E_s}{E_c}\right)$ $\boxed{n = 8}$

$k := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n} - \rho \cdot n$ $\boxed{k = 0.3}$

$j := 1 - \frac{k}{3}$ $\boxed{j = 0.9}$

$d_c := cover + Bar_D(bar_{stirrup}) + \frac{Bar_D(Bar_{No\_pos})}{2}$ $\boxed{d_c = 3.69}$ in

$\boxed{Ms_{pos} = 1634}$ kip-ft

$f_s := \frac{Ms_{pos}}{As_{prov\_pos} \cdot j \cdot d_e} \cdot 12$ $\boxed{f_s = 36.24}$ ksi

The height of the section, h, is:

$h := cap_H \cdot 12$ $\boxed{h = 48}$ in

$\beta := 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$ $\boxed{\beta = 1.12}$

$\gamma_e := 0.75$ for Class 2 exposure

$S_{max} := \frac{700 \gamma_e}{\beta \cdot f_s} - 2 \cdot d_c$ $\boxed{S_{max} = 5.57}$ in

$\boxed{spa_{pos} = 4.64}$ in

Is the bar spacing less than S_{max}? $\boxed{check = \text{"OK"}}$

E13-2.6.2 Positive Moment Reinforcement Cut Off Location

Terminate the top row of bars where bottom row of reinforcement satisfies the moment diagram.

$spa' := spa_{pos}$ $\boxed{spa' = 4.64}$ in

$As' := Bar_A(Bar_{No\_pos}) \cdot n_{bars\_pos1}$ $\boxed{As' = 9}$ in^2

$$a' := \frac{As' \cdot f_y}{0.85 \cdot b_w \cdot f'_c}$$ $\boxed{a' = 4.32}$ in

$$d_{e'} := cap_H \cdot 12 - cover - Bar_D(bar_{stirrup}) - \frac{Bar_D(Bar_{No\_pos})}{2}$$

$\boxed{d_e = 42.87}$ in

$$M_{n'} := As' \cdot f_y \cdot \left(d_{e'} - \frac{a'}{2}\right) \cdot \frac{1}{12}$$ $\boxed{M_{n'} = 1897}$ kip-ft

$$M_{r'} := \phi_f \cdot M_{n'}$$ $\boxed{M_{r'} = 1707}$ kip-ft

Based on the moment diagram, try locating the first cut off at $cut_{pos} := 10.1$ feet from the CL of the left column. Note that the Service I crack control requirements control the location of the cut off.

$\boxed{M_{r'} = 1707}$ kip-ft

$\boxed{Mu_{cut1} = 1665}$ kip-ft

$\boxed{Ms_{cut1} = 1140}$ kip-ft

Is Mu_{cut1} less than $M_{r'}$? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$\boxed{M_{cr} = 1023}$ kip-ft

$\boxed{1.33 \cdot Mu_{cut1} = 2215}$ 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 [5.7.3.4]**:

$$\rho' := \frac{As'}{b_w \cdot d_e}$$ $\boxed{\rho' = 0.00500}$

$$k' := \sqrt{(\rho' \cdot n)^2 + 2 \cdot \rho' \cdot n} - \rho' \cdot n$$ $\boxed{k' = 0.25}$

$$j' := 1 - \frac{k'}{3}$$ $\boxed{j' = 0.92}$

$$As_{prov\_neg} := Bar_A(Bar_{No\_neg}) \cdot n_{bars\_neg}$$ $\boxed{As_{prov\_neg} = 7.07}$ in^2

$$a_{neg} := \frac{As_{prov\_neg} \cdot f_y}{0.85 \cdot b_w \cdot f'_c}$$ $\boxed{a_{neg} = 3.39}$ in

$$Mn_{neg} := As_{prov\_neg} \cdot f_y \cdot \left(d_e - \frac{a_{neg}}{2}\right) \cdot \frac{1}{12}$$ $\boxed{Mn_{neg} = 1455}$ kip-ft

$$Mr_{neg} := \phi_f \cdot Mn_{neg}$$ $\boxed{Mr_{neg} = 1310}$ kip-ft

$\boxed{Mu_{neg} = 1174}$ kip-ft

Is M_u less than M_r? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$\boxed{M_{cr} = 1023}$ kip-ft

$\boxed{1.33 \cdot Mu_{neg} = 1561}$ kip-ft

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

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$$\rho_{neg} := \frac{As_{prov\_neg}}{b_w \cdot d_{e\_neg}}$$ $\boxed{\rho_{neg} = 0.00379}$

$\boxed{n = 8}$

$$k_{neg} := \sqrt{(\rho_{neg} \cdot n)^2 + 2 \cdot \rho_{neg} \cdot n} - \rho_{neg} \cdot n$$ $\boxed{k_{neg} = 0.22}$

$$j_{neg} := 1 - \frac{k_{neg}}{3}$$ $\boxed{j_{neg} = 0.93}$

$$d_{c\_neg} := cover + Bar_D(bar_{stirrup}) + \frac{Bar_D(Bar_{No\_neg})}{2}$$ $\boxed{d_{c\_neg} = 3.63}$ in

$\boxed{Ms_{neg} = 844}$ kip-ft

$$f_{s\_neg} := \frac{Ms_{neg}}{As_{prov\_neg} \cdot j_{neg} \cdot d_{e\_neg}} \cdot 12$$ $\boxed{f_s = 36.24}$ ksi

The height of the section, h, is: $\boxed{h = 48}$ in

$\beta_{neg} := 1 + \dfrac{d_{c\_neg}}{0.7 \cdot (h - d_{c\_neg})}$ $\boxed{\beta_{neg} = 1.12}$

$\gamma_e := 0.75$ for Class 2 exposure

$S_{max\_neg} := \dfrac{700\gamma_e}{\beta_{neg} \cdot f_{s\_neg}} - 2 \cdot d_{c\_neg}$ $\boxed{S_{max\_neg} = 6.26}$ in

$\boxed{spa_{neg} = 4.66}$ in

Is the bar spacing less than S_{max}? $\boxed{check = \text{"OK"}}$

E13-2.6.4 Negative Moment Reinforcement Cut Off Location

Cut 4 bars where the remaining 5 bars satisfy the moment diagram.

$n_{bars\_neg'} := 5$

$spa'_{neg} := spa_{neg} \cdot 2$ $\boxed{spa'_{neg} = 9.31}$ in

$As'_{neg} := Bar_A(Bar_{No\_neg}) \cdot n_{bars\_neg'}$ $\boxed{As'_{neg} = 3.93}$ in^2

$a'_{neg} := \dfrac{As'_{neg} \cdot f_y}{0.85 \cdot b_w \cdot f'_c}$ $\boxed{a'_{neg} = 1.89}$ in

$\boxed{d_{e\_neg} = 44.38}$ in

$M_{n'\_neg} := As'_{neg} \cdot f_y \cdot \left(d_{e\_neg} - \dfrac{a'_{neg}}{2}\right) \cdot \dfrac{1}{12}$ $\boxed{M_{n'\_neg} = 853}$ kip-ft

$M_{r'\_neg} := \phi_f \cdot M_{n'\_neg}$ $\boxed{M_{r'\_neg} = 768}$ kip-ft

Based on the moment diagram, try locating the cut off at $cut_{neg} := 15.3$ feet from the CL of the left column. Note that the Service I crack control requirements control the location of the cut off.

$\boxed{M_{r'\_neg} = 768}$ kip-ft

$\boxed{Mu_{neg\_cut} = 577}$ kip-ft

$\boxed{Ms_{neg\_cut} = 381}$ kip-ft

Is Mu_{cut1} less than $M_{r'}$? $\boxed{check = "OK"}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$\boxed{M_{cr} = 1023}$ kip-ft

$\boxed{1.33 \cdot Mu_{neg\_cut} = 767}$ kip-ft

Is $M_{r'}$ greater than the lesser value of M_{cr} and 1.33*Mu_{cut1}? $\boxed{check = "OK"}$

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$$\rho'_{neg} := \frac{As'_{neg}}{b_w \cdot d_e}$$ $\boxed{\rho'_{neg} = 0.00218}$

$$k'_{neg} := \sqrt{(\rho'_{neg} \cdot n)^2 + 2 \cdot \rho'_{neg} \cdot n} - \rho'_{neg} \cdot n$$ $\boxed{k'_{neg} = 0.17}$

$$j'_{neg} := 1 - \frac{k'_{neg}}{3}$$ $\boxed{j'_{neg} = 0.94}$

$\boxed{Ms_{neg\_cut} = 381}$ kip-ft

$$f_{s'\_neg} := \frac{Ms_{neg\_cut}}{As'_{neg} \cdot j'_{neg} \cdot d_{e\_neg}} \cdot 12$$ $\boxed{f_{s'\_neg} = 27.82}$ ksi

$\boxed{\beta_{neg} = 1.12}$

$\boxed{\gamma_e = 0.75}$

$$S_{max'\_neg} := \frac{700\gamma_e}{\beta_{neg} \cdot f_{s'\_neg}} - 2 \cdot d_{c\_neg}$$ $\boxed{S_{max'\_neg} = 9.65}$ in

$\boxed{spa'_{neg} = 9.31}$ in

Is the bar spacing less than $S_{max'}$? $\boxed{check = "OK"}$

The bars shall be extended past this cut off point for a distance not less than the following, **LRFD [5.11.1.2.3]**:

$\boxed{d_e = 42.87}$ in

$\boxed{12 \cdot Bar_D(Bar_{No\_neg}) = 12}$ in

$\boxed{\dfrac{(col_{spa} - col_w) \cdot 12}{16} = 10.69}$ in

$\boxed{BarExtend_{neg} = 42.87}$ in

These bars also must be extended past the point required for flexure the development length of the bar. From Chapter 9, **Table 9.9-1**, the development length for an epoxy coated number $\boxed{\rightarrow 8}$ top bar with spacing greater than 6-inches, is:

$l_{d\_8} := 3.25$ ft

The cut off location is determined by the following:

$\boxed{cut_{neg} - \dfrac{BarExtend_{neg}}{12} = 11.73}$ ft

$\boxed{col_{spa} - \dfrac{col_w}{2} - l_{d\_8} = 13}$ ft

Therefore, the cut off location is located at the following distance from the CL of the left column:

$\boxed{cutoff_{location} = 11.73}$ ft

By inspection, the remaining top mat reinforcement is adequate over the exterior columns. The inside face of the exterior column is located at:

$col_{face} := \dfrac{col_w}{2} \cdot \dfrac{1}{col_{spa}}$ $\boxed{col_{face} = 0.11}$ % along cap

$\boxed{Mu_{negative}(col_{face}) = -378.37}$ kip-ft

$\boxed{Ms_{negative}(col_{face}) = -229.74}$ kip-ft

Table of Contents

14.1 Introduction

Retaining walls are used to provide lateral resistance for a mass of earth or other material to accommodate a transportation facility. These walls are used in a variety of applications including right-of-way restrictions, protection of existing structures that must remain in place, grade separations, new highway embankment construction, roadway widening, stabilization of slopes, protection of environmentally sensitive areas, staging, and temporary support including excavation or underwater construction support, etc.

Several types of retaining wall systems are available to retain earth and meet specific project requirements. Many of these wall systems are proprietary wall systems while others non-proprietary or design-build in Wisconsin. The wall selection criteria and design policies presented in this chapter are to ensure consistency of standards and applications used throughout WisDOT projects.

WisDOT policy item:

Retaining walls (such as MSE walls with precast concrete panel facing) that are susceptible to damage from vehicular impact shall be protected by a roadway barrier.

14.1.1 Wall Development Process

Overall, the wall development process requires an iterative collaboration between WisDOT Regions, Structures Design Section, Geotechnical Unit and WisDOT Consultants.

Retaining wall development is described in Section 11-55-5 of the *Facilities Development Manual*. WisDOT Regional staff determines the need for permanent retaining walls on highway projects. A wall number is assigned as per criteria discussed in 14.1.1.1 of this chapter. The Regional staff prepares a Structures Survey Report (SSR) that includes a preliminary evaluation of wall type, location, and height including a preliminary layout plan.

Based on the SSR, a Geotechnical site investigation may be required to determine foundation and retained soil properties. A hydraulic analysis is also conducted, if required, to asses scour potential. The Geotechnical investigation generally includes a subsurface and laboratory investigation. For the departmental-designed walls, the Bureau of Technical Services, Foundation & Pavement Unit (Geotechnical Unit) can recommend the scope of soil exploration needed and provide/recommend bearing resistance, overall stability, and settlement of walls based on the geotechnical exploration results.

The SSR is sent to the wall designer (Structures Design Section or WisDOT's Consultant) for wall selection, design and contract plan preparation. Based on the wall selection criteria discussed in 14.3, either a proprietary or a non-proprietary wall system is selected.

Proprietary walls, as defined in 14.2, are pre-approved by the WisDOT's Structures Design Section. Preapproval process for the proprietary walls is explained in 14.16. The structural design, internal and final external stability of proprietary wall systems are the responsibility of the supplier/contractor. The design and shop drawing computations of the proprietary wall systems are also reviewed by the Structures Design Section in accordance with the plans and special provisions. The preliminary external stability, overall stability and settlement computations of these walls are performed by the Geotechnical Unit or the WisDOT's Consultant. Design and shop drawings must be approved by the Structures Design Section prior to start of the construction. Design of all temporary walls is the responsibility of the contractor.

Non-proprietary retaining walls are designed by WisDOT or its Consultant. The internal stability and the structural design of such walls are performed by the Structures Design Section or WisDOT's Consultant. The external and overall stability is performed by the Geotechnical Engineering Unit or Geotechnical Engineer of record.

The final contract plans of retaining walls include final plans, details, special provisions, contract requirements, and cost estimate for construction. The Subsurface Exploration is part of the final contract plans.

The wall types and wall selection criteria to be used in wall selection are discussed in 14.2 and 14.3 of this chapter respectively. General design concepts of a retaining wall system are discussed in 14.4. Design criteria for specific wall systems are discussed in sections 14.5 thru 14.11. The plan preparation process is briefly described in Chapter 2 – General and Chapter 6 – Plan Preparation. The contract documents and contract requirements are discussed in 14.14 and 14.15 respectively.

For further information related to wall selection, design, approval process, pre-approval and review of proprietary wall systems please contact Structures Design Section of the Bureau of Structures at 608-266-8489. For questions pertaining to geotechnical analyses and geotechnical investigations please contact the Geotechnical Unit at 608-246-7940.

14.1.1.1 Wall Numbering System

Permanent retaining walls that are designed for a design life of 75 years or more should be identified by a wall number, R-XX-XXX, as assigned by the Region. For a continuous wall consisting of various wall types, the numbering system should include unit numbers so that the numbering appears as R-XX-XXX-001, R-XX-XXX-002, and so on. The first two digits represent the county the wall is located in and the next set(s) of digits represent the undivided wall.

Retaining walls whose height exceeds the following criteria require R numbers:

- Modular block gravity walls and MSE walls: If the top of leveling pad to top of wall height (including any coping) exceeds 5.0 ft. at any point along the wall length.

- Cast-in-place, sheet pile, and all other walls: If the exposed height from the plan ground line to top of wall (including any coping) exceeds 5.0 ft. at any point along the wall length.

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

14.6 Mechanically Stabilized Earth Retaining Walls

14.6.1 General Considerations

Mechanically Stabilized Earth (MSE) is the term used to describe the practice of reinforcing a mass of soil with either metallic or geosynthetic soil reinforcement which allows the mass of soil to function as a gravity retaining wall structure. The soil reinforcement is placed horizontally across potential planes of shear failure and develops tension stresses to keep the soil mass intact. The soil reinforcement is attached to a wall facing located at the front face of the wall.

The design of MSE walls shall meet the *AASHTO LRFD* requirements in accordance with 14.4.2. The service life requirement for both permanent and temporary MSE wall systems is presented in 14.4.3.

The MSE walls shall be designed for external stability of the wall system and internal stability of the reinforced soil mass. The global stability shall also be considered as part of design evaluation. MSE walls are proprietary wall systems and the design responsibilities with respect to global, external, and internal stability as well as settlement are shared between the designer (WisDOT or Consultant) and contractor. The designer is responsible for the overall stability, preliminary external stability and settlement whereas the contractor is responsible for the internal stability, compound stability and structural design of the wall. The responsibilities of the designer and contractor are outlined in 14.6.3.2. The design and drawings of MSE walls provided by the contractor must also be in compliance with the WisDOT special provisions as stated in 14.15.2 and 14.16

The guidelines provided herein for MSE walls do not apply to geometrically complex MSE wall systems such as tiered walls (walls stacked on top of one another), back-to-back walls, or walls which have trapezoidal sections. Design guidelines for these cases are provided in publications FHWA-NHI-10-024 and FHWA-NHI-10-025.

Horizontal alignment and grades at the bottom and top of the wall are determined by the design engineer. The design must be in compliance with the WisDOT special provisions for the project and the policy and procedures as stated in the *Bridge Manual* and *FDM*.

14.6.1.1 Usage Restrictions for MSE Walls

Construction of MSE walls with either block or panel facings should not be used when any of the following conditions exist:

1. If the available construction limit behind the wall does not meet the soil reinforcement length requirements.

2. Sites where extensive excavation is required or sites that lack granular soils and the cost of importing suitable fill material may render the system uneconomical.

3. At locations where erosion or scour may undermine or erode the reinforced fill zone or any supporting leveling pad.

4. Soil is contaminated by corrosive material such as acid mine, drainage, other industrial pollutants, or any other condition which increases corrosion rate, such as the presence of stray electrical currents.

5. There is potential for placing buried utilities within (or below) the reinforced zone unless access is provided to utilities without disrupting reinforcement and breakage or rupture of utility lines will not have a detrimental effect on the stability of the wall. Contact WisDOT's Structures Design Section.

14.6.2 Structural Components

The main structural elements or components of an MSE wall are discussed below. General elements of a typical MSE wall are shown in Figure 14.6-1. These include:

- Selected Earthfill in the Reinforced Earth Zone
- Reinforcement
- Wall Facing Element
- Leveling Pad
- Wall Drainage

A combination of different wall facings and reinforcement provide a choice of selecting an MSE wall which can be used for several different functions.

MSE Wire-Faced Facing

Welded wire fabric facing is used to build MSE wire-faced walls. These are essentially MSE walls with a welded wire fabric facing instead of a precast concrete facing. The wire size, spacing and patterns used in the facing are developed from performance data of full size wall tests and from applications in actual walls. A test to determine the connection strength between the soil reinforcement and the facing panels is required. Some systems do not use a connection because the ground reinforcement and facing panel are of one piece construction.

MSE wire-faced wall systems usually incorporate a backing mat behind the front facing. A fine metallic screen or filter fabric is placed behind the backing mat (or behind the facing if a backing mat is not used) to prevent the backfill from passing thru the front face.

MSE wire-faced walls can tolerate considerable differential settlement because of the flexibility of the wire facing. The limiting differential settlement is 1/50. The flexibility of the wire facing results in face bulging between ground reinforcement. The actual amount varies per system but normally is less than one inch. Recommended limits on bulging are 2" for permanent walls and 3" for temporary walls.

When MSE wire-faced walls are used for permanent wall applications, all steel components must be galvanized. When used for temporary wall applications black steel (non-galvanized) may be used since the walls are usually left in place and buried.

Temporary MSE wire-faced walls can be used as temporary shoring if site conditions permit. This wall type can also be used when staged construction is required to maintain traffic when an existing roadway is being raised and/or widened in conjunction with bridge approaches, railroad crossings or road reconstruction.

Cast-In- Place Concrete Facing

MSE walls with cast in place concrete facings are identical to MSE wire faced walls except a cast-in-place concrete facing is added after the wire face wall is erected. Modifications are made to the standard wire face wall detail to anchor the concrete facing to the wire facing and soil reinforcement. They are usually used when a special aesthetic facial treatment is required without the numerous joints that are common to precast panels. They can also be used where differential settlement is above tolerable limits for other wall types. A MSE wire faced wall can be constructed and allowed to settle with the concrete facing added after consolidation of the foundation soils has occurred.

The cast-in-place concrete facing shall be a minimum of 8-inches thick and contain coated or galvanized reinforcing steel. This is required because the panels and/or anchor that extend into the cast-in-place concrete are galvanized and a corrosion cell would be created if black steel contacts galvanized steel. All wire ties and bar chairs used in the cast–in-place concrete must also be coated or galvanized. Note that the 8-inch minimum wall thickness will occur at the points of maximum panel bulging and that the wall will be thicker at other locations. Also note that the 8-inch minimum is measured from the trough of any form liner or rustication.

Vertical construction joints are required in the cast-in-place concrete facing to allow for expansion and contraction and to allow for some differential settlement. Closer spacing of vertical construction joints is required when differential settlement may occur, but by delaying the placement of the cast-in-place concrete, the effects of differential settlement is minimized. Higher walls also require closer spacing of vertical construction joints if differential settlement is anticipated. Horizontal construction joints may disrupt the flow of a special aesthetic facial treatment and are sometimes not allowed for that reason. The designer should specify if optional horizontal construction joints are allowed. Cork filler is placed at vertical construction joints because cork is compressible and will allow some expansion and rotation to occur at the joint. An expandable polyvinyl chloride waterstop (PCW) is used on the back side of a vertical construction joint. Since forms are only used at the front face of the wall the PCW can be attached to a 10-inch board which is supported by the wire facing. The 8-inch minimum wall thickness may be decreased at the location of the vertical construction joint to accommodate the PCW and its support board.

Geosynthetic Facing

Geosynthetic reinforcements are looped around at the facing to form the exposed face of the MSE Wall. These faces are susceptible to ultraviolet light degradation, vandalism, and damage due to fire. Geogrid used for soil reinforcement can be looped around to form the face of the completed retaining structure in a similar manner to welded wire mesh and fabric facing. This facing is generally used in temporary applications. Similar to wire faced walls, these walls typically have a geotextile behind the geogrids, to prevent material from passing through the face.

14.6.3 Design Procedure

14.6.3.1 General Design Requirements

The procedure for design of an MSE wall requires evaluation of external stability and internal stability (structural design) at Strength Limit States and overall stability and vertical/lateral movement at Service Limit State. The Extreme Event II load combination is used to design and analyze for vehicle impact where traffic barriers are provided to protect MSE walls. The design and stability is performed in accordance with *AASHTO LRFD* and design guidance discussed in 14.4.

14.6.3.2 Design Responsibilities

MSE walls are proprietary wall systems and the structural design of the wall system is provided by the contractor. The structural design of the MSE wall system must include an analysis of internal stability (soil reinforcement pullout and stress) and local stability (facing connection forces and internal panel stresses). Additionally, the contractor should also provide internal drainage. Design drawings and calculations must be submitted to the Structures Design Section for acceptance.

External stability, overall stability and settlement calculations are the responsibility of the WISDOT/Consultant designer. Compound stability is the responsibility of the Contractor. Soil borings and soil design parameters are provided by Geotechnical Engineer.

Table of Contents

E14-1 Cast-In-Place Concrete Cantilever Wall on Spread Footing, LRFD

General

This example shows design calculations for a cast-in-place (CIP) concrete wall supported on a spread footing conforming to the LRFD Bridge Design Specifications and the WisDOT Bridge Manual. *(Example is current through LRFD Sixth Edition - 2012)*

Sample design calculations for bearing resistance, external stability (sliding, eccentricity and bearing) and wall strength design will be presented. The overall stability and settlement calculations will not be shown in this example, but are required.

Design steps presented in 14.5.2.1 are used for the wall design.

E14-1.1 Establish Project Requirements

The CIP concrete wall shown in Figure E14-1.1-1 will be designed appropriately to accommodate a State Trunk Highway. External stability, overall stability and wall strength are the designer's (WisDOT/Consultant) responsibility.

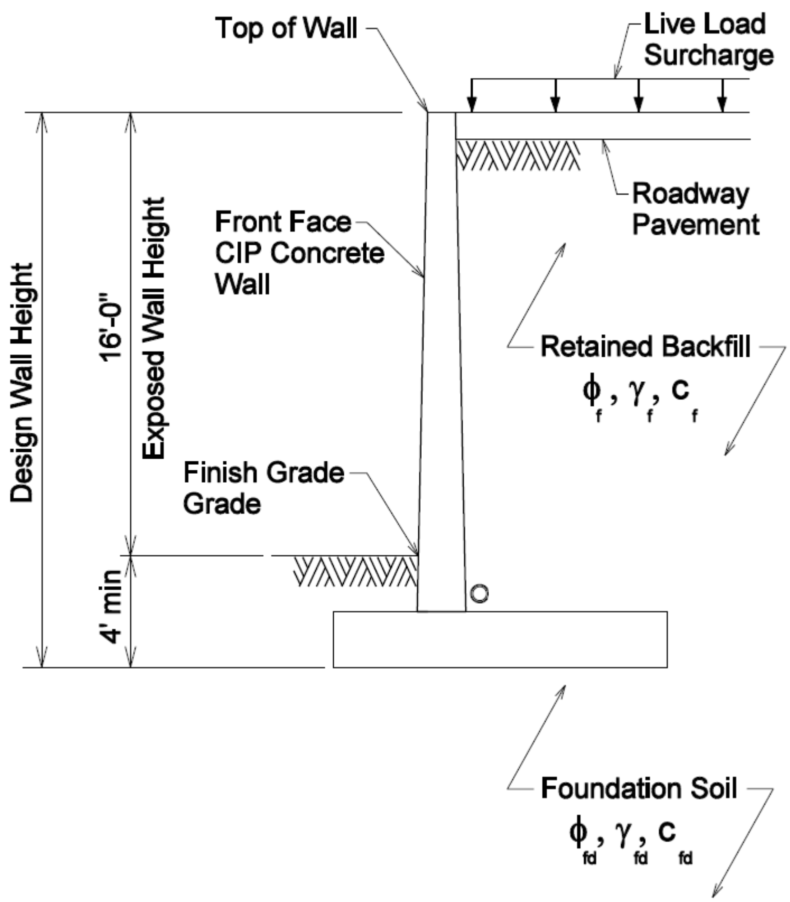


Figure E14-1.1-1
CIP Concrete Wall Adjacent to Highway

E14-1.2 Design Parameters

Project Parameters

$Design\_Life = 75\ years$ — Wall design life (min) **LRFD [11.5.1]**

Soil Properties (From Geotechnical Site Investigation Report)

Designer to determine if long-term or short-term soil strength parameters govern external stability.

Backfill Soil Design Parameters

$\phi_f = 30\ deg$ — Angle of internal friction

$\gamma_f = 0.120$ — Unit weight, kcf

$c_f = 0$ — Cohesion, ksf

$\delta = 21\ deg$ — Friction angle between fill and wall

Note: Per WisDOT Bridge Manual and Standard Specifications, structural backfill shall be granular and non-expansive.

Foundation Soil Design Parameters

$\phi_{fd} = 34\ deg$ — Angle of internal friction

$\gamma_{fd} = 0.120$ — Unit of weight, kcf

$c_{fd} = 0$ — Cohesion, ksf

Reinforced Concrete Parameters

$f'_c = 3.5$ — Concrete compressive design strength, ksi (14.5.9)

$\gamma_c = 0.150$ $\quad w_c = \gamma_c$ — Unit weight of concrete, ksf

$E_c = 33000\ w_c^{1.5}\sqrt{f'_c}$ — Modulus of elasticity of concrete, ksi **LRFD [5.4.2.4]**

$\boxed{E_c = 3587}$ ksi

$f_y = 60$ — Yield strength of reinforcing bars, ksi (14.5.9)

$E_s = 29000$ — Modulus of elasticity of reinforcing bars, ksi

Live Load Surcharge Parameters

Live load surcharge shall be used when vehicular load is located within H/2 of the backface of the wall **LRFD [3.11.6.4].** The equivalent height of soil for vehicular load, H_{eq}, used for surcharge loads shall be in accordance to **LRFD [Table 3.11.6.4-2]**. However, WisDOT policy for most cases requires an equivalent height of 2.0 feet. The following procedure is used for determining live load surcharge:

$L_{traffic} = 1.0$ Distance from wall backface to edge of traffic, ft

$\frac{H}{2} = 10.00$ Distance from wall backface where live load surcharge shall be considered in the wall design, ft

Note: The wall height used is the exposed height plus an assumed 4 feet embedment ($H=H_e+4$ feet)

Shall live load surcharge be included? check = "YES"

$h_{eq} = 2.0$ Equivalent height of soil for surcharge load, ft (14.4.5.4.2)

Pavement Parameters

$\gamma_p = 0.150$ Pavement unit weight, kcf

Resistance Factors

$\phi_b = 0.55$ Bearing resistance (gravity and semi-gravity walls) **LRFD [Table 11.5.7-1]**

$\phi_s = 1.00$ Sliding resistance **LRFD [Table 11.5.7-1]**

$\phi_\tau = 1.00$ Sliding resistance (shear resistance bewtween soil and foundation) **LRFD [Table 11.5.7-1]**

$\phi_{ep} = 0.50$ Sliding resistance (passive resistance) **LRFD [Table 10.5.5.2.2-1]**

$\phi_F = 0.90$ Concrete flexural resistance (Assuming tension-controlled) **LRFD [5.5.4.2.1]**

$\phi_v = 0.90$ Concrete shear resistance **LRFD [5.5.4.2.1]**

E14-1.5 Compute Bearing Resistance, q_R

Nominal bearing resistance, q_n **LRFD [Eq 10.6.3.1.2a-1]**

$$q_n = c_{fd} N_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5 \gamma B' N_{\gamma m} C_{w\gamma}$$

Compute the resultant location (distance from Point 'O' Figure E14-4.4-3)

$\Sigma M_R = MV\_Ib$ $\boxed{\Sigma M_R = 205.8}$ Summation of resisting moments for Strength Ib

$\Sigma M_O = MH\_Ib$ $\boxed{\Sigma M_O = 81.3}$ Summation of overturning moments for Strength Ib

$\Sigma V = V\_Ib$ $\boxed{\Sigma V = 29.3}$ Summation of vertical loads for Strength Ib

$x = \dfrac{\Sigma M_R - \Sigma M_O}{\Sigma V}$ Distance from Point "O" the resultant intersects the base $\boxed{x = 4.25}$ ft

Compute the wall eccentricity

$e = \dfrac{B}{2} - x$ $\boxed{e = 0.75}$ ft

Define the foundation layout

$B' = B - 2\,e$ Footing width $\boxed{B' = 8.5}$ ft

$L' = 90.0$ Footing length (Assumed) $\boxed{L' = 90.0}$ ft

$H' = H\_Ib$ Summation of horizontal loads for Strength Ib $\boxed{H' = 11.7}$ kip/ft

$V' = V\_Ib$ Summation of vertical loads for Strength Ib $\boxed{V' = 29.3}$ kip/ft

$D_f = 4.00$ Footing embedment

$\theta' = \operatorname{atan}\left(\dfrac{V'}{H'}\right)$ Direction of resultant from horizontal $\boxed{\theta' = 68.3\,deg}$

Compute bearing capacity factors per **LRFD [Table 10.6.3.1.2a-1]**

$\phi_{fd} = 34.0\,deg$ $N_q = 29.4$ $N_c = 42.2$ $N_\gamma = 41.1$

Compute shape correction factors per **LRFD [Table 10.6.3.1.2a-3]**

Since the friction angle, ϕ_f, is > 0 the following equations are used:

$s_c = 1 + \left(\dfrac{B'}{L'}\right)\left(\dfrac{N_q}{N_c}\right)$ $\boxed{s_c = 1.07}$

$s_q = 1 + \left(\dfrac{B'}{L'}\tan(\phi_{fd})\right)$ $\boxed{s_q = 1.06}$

$s_\gamma = 1 - 0.4\left(\dfrac{B'}{L'}\right)$ $\boxed{s_\gamma = 0.96}$

Compute load inclination factors using **LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9]**

$$n = \frac{2 + \frac{L'}{B'}}{1 + \frac{L'}{B'}} \cos(\theta')^2 + \frac{2 + \frac{B'}{L'}}{1 + \frac{B'}{L'}} \sin(\theta')^2$$ $\boxed{n = 1.80}$

$$i_q = \left(1 - \frac{H'}{V' + c_{fd}\, B'\, L' \frac{1}{\tan(\phi_{fd})}}\right)^n$$ $\boxed{i_q = 0.40}$

$$i_\gamma = \left(1 - \frac{H'}{V' + c_{fd}\, B'\, L' \frac{1}{\tan(\phi_{fd})}}\right)^{n+1}$$ $\boxed{i_\gamma = 0.24}$

$$i_c = i_q - \left(\frac{1 - i_q}{N_q - 1}\right)$$ For $\phi_{fd} > 0$: $\boxed{i_c = 0.38}$

Compute depth correction factor per **LRFD [Table 10.6.3.1.2a-4]**. While it can be assumed that the soils above the footing are as competent as beneath the footing, the depth correction factor is taken as 1.0 since D_f/B is less than 1.0.

$d_q = 1.00$

Determine coefficients C_{wq} and $C_{w\gamma}$ assuming that the water depth is greater than 1.5 times the footing base plus the embedment depth per **LRFD [Table 10.6.3.1.2a-2]**

$C_{wq} = 1.0$ where $D_w > 1.5B + D_f$

$C_{w\gamma} = 1.0$ where $D_w > 1.5B + D_f$

Compute modified bearing capacity factors
LRFD [Equation 10.6.3.1.2a-2 to 10.6.3.1.2a-4]

$N_{cm} = N_c\, s_c\, i_c$ $\boxed{N_{cm} = 17.0}$

$N_{qm} = N_q\, s_q\, d_q\, i_q$ $\boxed{N_{qm} = 12.5}$

$N_{\gamma m} = N_\gamma\, s_\gamma\, i_\gamma$ $\boxed{N_{\gamma m} = 9.5}$

Compute nominal bearing resistance, q_n, **LRFD [Eq 10.6.3.1.2a-1]**

$q_n = c_{fd}\, N_{cm} + \gamma_{fd}\, D_f\, N_{qm}\, C_{wq} + 0.5\, \gamma_{fd}\, B'\, N_{\gamma m}\, C_{w\gamma}$ $\boxed{q_n = 10.86}$ ksf/ft

Compute factored bearing resistance, q_R, **LRFD [Eq 10.6.3.1.1]**

$\phi_b = 0.55$

$q_R = \phi_b\, q_n$ $\boxed{q_R = 5.97}$ ksf/ft

E14-1.6 Evaluate External Stability of Wall

Three potential external failure mechanisms will be considered in this example. These failures include bearing, limiting eccentricity and sliding. Global (overall) stability requirements are assumed to have been satisfied in prior calculations. Design calculations will be carried out for the governing limit states only.

E14-1.6.1 Bearing Resistance at Base of the Wall

The following calculations are based on **Strength Ib**:

Compute resultant location (distance from Point 'O' Figure E14-1.4-3)

$\Sigma M_R = MV\_Ib$ $\qquad \Sigma M_R = 205.8$ kip-ft/ft

$\Sigma M_O = MH\_Ib$ $\qquad \Sigma M_O = 81.3$ kip-ft/ft

$\Sigma V = V\_Ib$ $\qquad \Sigma V = 29.3$ kip/ft

$$x = \frac{\Sigma M_R - \Sigma M_O}{\Sigma V}$$

Distance from Point "O" the resultant intersects the base

$\boxed{x = 4.25}$ ft

Compute the wall eccentricity

$$e = \frac{B}{2} - x$$

$\boxed{e = 0.75}$ ft

Note: The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e. When the foundation eccentricity is negative the actual bearing width, B, will be used.

Compute the ultimate bearing stress

$$\sigma_V = \frac{\Sigma V}{B - 2\,e}$$

$\boxed{\sigma_V = 3.44}$ ksf/ft

Factored bearing resistance

$\boxed{q_R = 5.97}$ ksf/ft

<u>Capacity:Demand Ratio (CDR)</u>

$$CDR_{Bearing1} = \frac{q_R}{\sigma_V}$$

$\boxed{CDR_{Bearing1} = 1.74}$

Is the $CDR \geq 1.0$?

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

E14-1.6.2 Limiting Eccentricity at Base of the Wall

The location of the resultant of the reaction forces is limited to the middle one-half of the base width for a soil foundation (i.e., e_{max} = B/4). The following calculations are based on **Strength Ia**:

Note: While AASHTO **LRFD [11.6.3.3]** allows B/3, eccentricity is limited to B/4 by WisDOT policy as stated in Chapter 14.

Maximum eccentricity

$$e_{max} = \frac{B}{4} \qquad \boxed{e_{max} = 2.50} \text{ ft}$$

Compute resultant location (distance from Point 'O' Figure E14-1.4.3)

$\Sigma M_R = MV\_Ia \qquad \Sigma M_R = 150.0$ kip-ft/ft

$\Sigma M_O = MH\_Ia \qquad \Sigma M_O = 81.3$ kip-ft/ft

$\Sigma V = V\_Ia \qquad \Sigma V = 20.9$ kip/ft

$$x = \frac{\Sigma M_R - \Sigma M_O}{\Sigma V}$$ Distance from Point "O" the resultant intersects the base

$\boxed{x = 3.29}$ ft

Compute the wall eccentricity

$$e = \frac{B}{2} - x \qquad \boxed{e = 1.71} \text{ ft}$$

<u>Capacity:Demand Ratio (CDR)</u>

$$CDR_{Eccentricity1} = \frac{e_{max}}{e} \qquad \boxed{CDR_{Eccentricity1} = 1.46}$$

Is the $CDR \geq 1.0$? $\boxed{check = \text{"OK"}}$

E14-1.6.3 Sliding Resistance at Base of the Wall

For sliding failure, the horizontal force effects, R_u, is checked against the sliding resistance, R_R, where $R_R=\phi R_n$ **LRFD [10.6.3.4]**. If sliding resistance is not adequate a shear key will be investigated. The following calculations are based on **Strength Ia**:

Factored Sliding Force, R_u

$R_u = H\_Ia$ $\boxed{R_u = 11.7}$ kip/ft

Sliding Resistance, R_R

$$R_R = \phi_s R_n = \phi_\tau R_\tau + \phi_{ep} R_{ep}$$

Compute sliding resistance between soil and foundation, $\phi_\tau\ R_\tau$

$\Sigma V = V\_Ia$ $\boxed{\Sigma V = 20.9}$ kip/ft

$R_\tau = \Sigma V \tan(\phi_{fd})$ $\boxed{R_\tau = 14.1}$ kip/ft

$\phi_\tau = 1.00$ $\boxed{\phi_\tau\ R_\tau = 14.1}$ kip/ft

Compute passive resistance throughout the design life of the wall, $\phi_{ep}\ R_{ep}$

$r_{ep1} = k_p\ \gamma_{fd}\ y_1$ Nominal passive pressure at y_1 $\boxed{r_{ep1} = 1.70}$ kip/ft

$r_{ep2} = k_p\ \gamma_{fd}\ y_2$ Nominal passive pressure at y_2 $\boxed{r_{ep2} = 2.12}$ kip/ft

$R_{ep} = \dfrac{r_{ep1} + r_{ep2}}{2}(y_2 - y_1)$ $\boxed{R_{ep} = 1.9}$ kip/ft

$\phi_{ep} = 0.50$ $\boxed{\phi_{ep}\ R_{ep} = 1.0}$ kip/ft

Compute nominal resistance against failure by sliding, R_n

$R_n = \phi_\tau\ R_\tau + \phi_{ep}\ R_{ep}$ $\boxed{R_n = 15.1}$ kip/ft

Compute factored resistance against failure by sliding, R_R

$\phi_s = 1.00$

$R_R = \phi_s\ R_n$ $\boxed{R_R = 15.1}$ kip/ft

<u>Capacity:Demand Ratio (CDR)</u>

$CDR_{Sliding1} = \dfrac{R_R}{R_u}$ $\boxed{CDR_{Sliding1} = 1.29}$

Is the CDR ≥ 1.0 ? $\boxed{check = \text{"OK"}}$

E14-1.7 Evaluate Wall Structural Design

Note: CIP concrete walls are a non-proprietary wall system and the structural design computations shall be performed by the wall designer.

Wall structural design computations for shear and flexure will be considered in this example. The critical sections for flexure are taken at the front, back and bottom of them stem. For simplicity, critical sections for shear will be taken at the critical sections used for flexsure. In actuality, the toe and stem may be designed for shear at the effective depth away from the face. Crack control and temperature and shrinkage considerations will also be included.

E14-1.7.1 Evaluate Heel Strength

E14-1.7.1.1 Evaluate Heel Shear Strength

For **Strength Ib**:

$$V_u = 1.25\left(\frac{C}{B}V_4 + V_6\right) + 1.35\left(V_7 + V_8 + V_9\right) + 1.75\left(V_{10}\right) + 1.50\left(V_{11}\right)$$

$\boxed{V_u = 21.9}$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

where: $V_c = 0.0316\,\beta\sqrt{f'_c}\,b_v\,d_v$

$V_{n2} = 0.25\,f'_c\,b_v\,d_v$ **LRFD [Eq 5.8.3.3-2]**

Compute the shear resistance due to concrete, V_c :

$cover = 2.0$ in

$s = 7.0$ in (bar spacing)

$Bar_{No} = 6$ (transverse bar size)

$Bar_D = 0.750$ in (transverse bar diameter)

$Bar_A = 0.440$ in^2 (transverse bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$ $\boxed{A_s = 0.75}$ in^2/ft

$$d_s = D\,12 - cover - \frac{Bar_D}{2}$$ $\boxed{d_s = 21.6}$ in

$$a = \frac{A_s\,f_y}{0.85\,f'_c\,b}$$ $\boxed{a = 1.3}$ in

$$d_{v1} = d_s - \frac{a}{2}$$ $\boxed{d_{v1} = 21.0}$ in

$$d_{v2} = 0.9\ d_s$$ $\boxed{d_{v2} = 19.5}$ in

$$d_{v3} = 0.72\ D\ 12$$ $\boxed{d_{v3} = 17.3}$ in

$$d_v = \max(d_{v1}, d_{v2}, d_{v3})$$ $\boxed{d_v = 21.0}$ in

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2}

$$\beta = 2.0$$

$$V_c = 0.0316\ \beta\ \sqrt{f'_c}\ b\ d_v$$

$$V_{n1} = V_c$$ $\boxed{V_{n1} = 29.8}$ kip/ft

$$V_{n2} = 0.25\ f'_c\ b\ d_v$$ $\boxed{V_{n2} = 220.4}$ kip/ft

$$V_n = \min(V_{n1}, V_{n2})$$ $\boxed{V_n = 29.8}$ kip/ft

$$V_r = \phi_v\ V_n$$ $\boxed{V_r = 26.8}$ kip/ft

$\boxed{V_u = 21.9}$ kip/ft

Is V_u less than V_r? $\boxed{check = \text{"OK"}}$

E14-1.7.1.2 Evaluate Heel Flexural Strength

$V_u = 21.9$ kip/ft

$$M_u = V_u\ \frac{C}{2}$$ $\boxed{M_u = 47.9}$ kip-ft/ft

Calculated the capacity of the heel in flexure at the face of the stem:

$$M_n = A_s\ f_y \left(d_s - \frac{a}{2}\right) \frac{1}{12}$$ $\boxed{M_n = 79.2}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$$\beta_1 = 0.85$$

$$c = \frac{a}{\beta_1}$$ $\boxed{c = 1.49}$ in

$$\phi_F = \begin{cases} 0.75 \quad \text{if} \quad \dfrac{d_s}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_s}{c} - 1\right) \quad \text{if} \quad \dfrac{5}{3} \le \dfrac{d_s}{c} \le \dfrac{8}{3} \\ 0.90 \quad \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

Note:	if $\phi_F = 0.75$	Section is compression-controlled
	if $0.75 < \phi_F < 0.90$	Section is in transition
	if $\phi_F = 0.90$	Section is tension-controlled

Calculate the flexural factored resistance, M_r:

$M_r = \phi_F M_n$ $\boxed{M_r = 71.2}$ kip-ft/ft

$\boxed{M_u = 47.9}$ kip-ft/ft

Is M_u less than M_r? $\boxed{\text{check} = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$f_r = 0.37\sqrt{f'_c}$ $\boxed{f_r = 0.692}$ ksi

$I_g = \dfrac{1}{12} b (D\ 12)^3$ $\boxed{I_g = 13824}$ in^4

$y_t = \dfrac{1}{2} D\ 12$ $\boxed{y_t = 12.00}$ in

$S_c = \dfrac{I_g}{y_t}$ $\boxed{S_c = 1152}$ in^3

$M_{cr} = \gamma_3(\gamma_1 f_r) S_c$ therefore, $M_{cr} = 1.1 f_r S_c$

Where:

$\gamma_1 = 1.6$ flexural cracking variability factor

$\gamma_3 = 0.67$ ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$M_{cr} = 1.1 f_r S_c \dfrac{1}{12}$ $\boxed{M_{cr} = 73.1}$ kip-ft/ft

$\boxed{1.33\ M_u = 63.7}$ kip-ft/ft

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

E14-1.7.2 Evaluate Toe Strength

The structural design of the footing toe is calculated using a linear contact stress distribution for bearing for all soil and rock conditions.

E14-1.7.2.1 Evaluate Toe Shear Strength

For **Strength Ib**:

$\Sigma M_R = MV\_Ib$ $\boxed{\Sigma M_R = 205.8}$ kip-ft/ft

$\Sigma M_O = MH\_Ib$ $\boxed{\Sigma M_O = 81.3}$ kip-ft/ft

$\Sigma V = V\_Ib$ $\boxed{\Sigma V = 29.3}$ kip/ft

$$x = \frac{\Sigma M_R - \Sigma M_O}{\Sigma V}$$ $\boxed{x = 4.3}$ ft

$$e = \max\left(0, \frac{B}{2} - x\right)$$ $\boxed{e = 0.75}$ ft

$$\sigma_{max} = \frac{\Sigma V}{B}\left(1 + 6\,\frac{e}{B}\right)$$ $\boxed{\sigma_{max} = 4.24}$ ksf/ft

$$\sigma_{min} = \frac{\Sigma V}{B}\left(1 - 6\,\frac{e}{B}\right)$$ $\boxed{\sigma_{min} = 1.62}$ ksf/ft

Calculate the average stress on the toe

$$\sigma_v = \frac{\sigma_{max} + \left[\sigma_{min} + \frac{B - A}{B}\left(\sigma_{max} - \sigma_{min}\right)\right]}{2}$$ $\boxed{\sigma_v = 3.78}$ ksf/ft

$V_u = \sigma_v\, A$ $\boxed{V_u = 13.2}$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

in which: $V_c = 0.0316\ \beta\sqrt{f'_c}\ b_v\ d_v$

$V_{n2} = 0.25\ f'_c\ b_v\ d_v$ **LRFD [Eq 5.8.3.3-2]**

Design footing toe for shear

$cover = 3.0$ in

$s = 9.0$ in (bar spacing)

$Bar_{No} = 5$ (transverse bar size)

$Bar_D = 0.63$ in (transverse bar diameter)

$Bar_A = 0.31$ in^2 (transverse bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$ $\boxed{A_s = 0.41}$ in^2/ft

$$d_s = D\ 12 - cover - \frac{Bar_D}{2}$$ $\boxed{d_s = 20.7}$ in

$$a = \frac{A_s\ f_y}{0.85\ f'_c\ b}$$ $\boxed{a = 0.7}$ in

$$d_{v1} = d_s - \frac{a}{2}$$ $\boxed{d_{v1} = 20.3}$ in

$$d_{v2} = 0.9\ d_s$$ $\boxed{d_{v2} = 18.6}$ in

$$d_{v3} = 0.72\ D\ 12$$ $\boxed{d_{v3} = 17.3}$ in

$$d_v = \max(d_{v1}, d_{v2}, d_{v3})$$ $\boxed{d_v = 20.3}$ in

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2}

$$\beta = 2.0$$

$$V_c = 0.0316\ \beta\ \sqrt{f'_c}\ b\ d_v$$

$$V_{n1} = V_c$$ $\boxed{V_{n1} = 28.9}$ kip/ft

$$V_{n2} = 0.25\ f'_c\ b\ d_v$$ $\boxed{V_{n2} = 213.6}$ kip/ft

$$V_n = \min(V_{n1}, V_{n2})$$ $\boxed{V_n = 28.9}$ kip/ft

$$V_r = \phi_v\ V_n$$ $\boxed{V_r = 26.0}$ kip/ft

$\boxed{V_u = 13.2}$ kip/ft

Is V_u less than V_r? $\boxed{check = \text{"OK"}}$

E14-1.7.2.2 Evaluate Toe Flexural Strength

$V_u = 13.2$ kip/ft

$$M_u = V_u \frac{A}{2}$$ $\boxed{M_u = 23.2}$ kip-ft/ft

Calculated the capacity of the toe in flexure at the face of the stem:

$$M_n = A_s f_y \left(d_s - \frac{a}{2}\right) \frac{1}{12}$$ $\boxed{M_n = 42.0}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$$c = \frac{a}{\beta_1}$$ $\boxed{c = 0.82}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \frac{d_s}{c} < \frac{5}{3} \\ 0.65 + 0.15\left(\frac{d_s}{c} - 1\right) & \text{if } \frac{5}{3} \le \frac{d_s}{c} \le \frac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$ $\boxed{\phi_F = 0.90}$

Calculate the flexural factored resistance, M_r:

$$M_r = \phi_F M_n$$ $\boxed{M_r = 37.8}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = "OK"}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$$f_r = 0.37 \sqrt{f'_c}$$ $\boxed{f_r = 0.692}$ ksi

$$I_g = \frac{1}{12} b (D\ 12)^3$$ $\boxed{I_g = 13824}$ in^4

$$y_t = \frac{1}{2} D\ 12$$ $\boxed{y_t = 12.00}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1152}$ in^3

$$M_{cr} = 1.1 f_r S_c \frac{1}{12}$$ from E14-1.7.1.2 $\boxed{M_{cr} = 73.1}$ kip-ft/ft

$\boxed{1.33\ M_u = 30.8}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and $1.33*M_u$?

$\boxed{check = "OK"}$

E14-1.7.3 Evaluate Stem Strength

Unfactored Stem Horizontal Loads and Moments:

$H_1 = \gamma_f\, h_{eq}\, (h' - t)\, k_a \cos(90\ deg - \theta + \delta)$ $\boxed{H_1 = 1.2}$ kip/ft

$H_2 = \frac{1}{2}\gamma_f\, (h' - t)^2\, k_a \cos(90\ deg - \theta + \delta)$ $\boxed{H_2 = 5.0}$ kip/ft

$M_1 = H_1 \left(\frac{h' - t}{2}\right)$ $\boxed{M_1 = 10.0}$ kip-ft/ft

$M_2 = H_2 \left(\frac{h' - t}{3}\right)$ $\boxed{M_2 = 28.4}$ kip-ft/ft

Factored Stem Horizontal Loads and Moments:

for **Strength Ib**:

$H_{u1} = 1.75\ H_1 + 1.50\ H_2$ $\boxed{H_{u1} = 9.6}$ kip/ft

$M_{u1} = 1.75\ M_1 + 1.50\ M_2$ $\boxed{M_{u1} = 60.0}$ kip-ft/ft

for **Service I**:

$H_{u3} = 1.00\ H_1 + 1.00\ H_2$ $\boxed{H_{u3} = 6.2}$ kip/ft

$M_{u3} = 1.00\ M_1 + 1.00\ M_2$ $\boxed{M_{u3} = 38.4}$ kip-ft/ft

E14-1.7.3.1 Evaluate Stem Shear Strength at Footing

$V_u = H_{u1}$ $\boxed{V_u = 9.6}$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

where: $V_c = 0.0316\ \beta \sqrt{f'_c}\ b_v\ d_v$

$V_{n2} = 0.25\ f'_c\ b_v\ d_v$ **LRFD [Eq 5.8.3.3-2]**

Compute the shear resistance due to concrete, V_c :

$cover = 2.0$ in

$s = 10.0$ in (bar spacing)

$Bar_{No} = 8$ (transverse bar size)

$Bar_D = 1.00$ in (transverse bar diameter)

$Bar_A = 0.79$ in^2 (transverse bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_s = 0.95} \ in^2/ft$$

$$d_s = T_b\ 12 - cover - \frac{Bar_D}{2} \qquad \boxed{d_s = 23.0} \ in$$

$$a = \frac{A_s\ f_y}{0.85\ f'_c\ b} \qquad \boxed{a = 1.6} \ in$$

$$d_{v1} = d_s - \frac{a}{2} \qquad \boxed{d_{v1} = 22.2} \ in$$

$$d_{v2} = 0.9\ d_s \qquad \boxed{d_{v2} = 20.7} \ in$$

$$d_{v3} = 0.72\ T_b\ 12 \qquad \boxed{d_{v3} = 18.4} \ in$$

$$d_v = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_v = 22.2} \ in$$

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2}

$$\beta = 2.0$$

$$V_c = 0.0316\ \beta\ \sqrt{f'_c}\ b\ d_v$$

$$V_{n1} = V_c \qquad \boxed{V_{n1} = 31.5} \ kip/ft$$

$$V_{n2} = 0.25\ f'_c\ b\ d_v \qquad \boxed{V_{n2} = 233.1} \ kip/ft$$

$$V_n = \min(V_{n1}, V_{n2}) \qquad \boxed{V_n = 31.5} \ kip/ft$$

$$V_r = \phi_v\ V_n \qquad \boxed{V_r = 28.4} \ kip/ft$$

$$\boxed{V_u = 9.6} \ kip/ft$$

Is V_u less than V_r? $\boxed{check = "OK"}$

E14-1.7.3.2 Evaluate Stem Flexural Strength at Footing

$$M_u = M_{u1}$$ $\boxed{M_u = 60.0}$ kip-ft/ft

Calculate the capacity of the stem in flexure at the face of the footing:

$$M_n = A_s\, f_y \left(d_s - \frac{a}{2}\right)\frac{1}{12}$$ $\boxed{M_n = 105.2}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$$\beta_1 = 0.85$$

$$c = \frac{a}{\beta_1}$$ $\boxed{c = 1.87}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \dfrac{d_s}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_s}{c} - 1\right) & \text{if } \dfrac{5}{3} \le \dfrac{d_s}{c} \le \dfrac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$ $\boxed{\phi_F = 0.90}$

Calculate the flexural factored resistance, M_r:

$$M_r = \phi_F\, M_n$$ $\boxed{M_r = 94.7}$ kip-ft/ft

$\boxed{M_u = 60.0}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = "OK"}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$$f_r = 0.37\sqrt{f'_c}$$ $\boxed{f_r = 0.69}$ ksi

$$I_g = \frac{1}{12}\, b\, \left(T_b\, 12\right)^3$$ $\boxed{I_g = 16581}$ in^4

$$y_t = \frac{1}{2}\, T_b\, 12$$ $\boxed{y_t = 12.8}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1301}$ in^3

$$M_{cr\_s} = 1.1\ f_r\ S_c\ \frac{1}{12}$$ from E14-1.7.1.2

$\boxed{M_{cr\_s} = 82.5}$ kip-ft/ft

$\boxed{1.33\ M_u = 79.9}$ kip-ft/ft

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

Check the Service Ib crack control requirements in accordance with **LRFD [5.7.3.4]**

$$\rho = \frac{A_s}{d_s\ b}$$ $\boxed{\rho = 0.00343}$

$$n = \frac{E_s}{E_c}$$ $\boxed{n = 8.09}$

$$k = \sqrt{(\rho\ n)^2 + 2\ \rho\ n} - \rho\ n$$ $\boxed{k = 0.210}$

$$j = 1 - \frac{k}{3}$$ $\boxed{j = 0.930}$

$$d_c = cover + \frac{Bar_D}{2}$$ $\boxed{d_c = 2.5}$ in

$$f_{ss} = \frac{M_{u3}}{A_s\ j\ d_s}\ 12$$ $\boxed{f_{ss} = 22.7}$ ksi

$$h = T_b\ 12$$

$$\beta_s = 1 + \frac{d_c}{0.7\left(h - d_c\right)}$$ $\boxed{\beta_s = 1.2}$

$\gamma_e = 1.0$ for Class 1 exposure

$$s_{max} = \frac{700\ \gamma_e}{\beta_s\ f_{ss}} - 2\ d_c$$ $\boxed{s_{max} = 21.7}$ in

$\boxed{s = 10.0}$ in

Is the bar spacing less than s_{max}? $\boxed{check = \text{"OK"}}$

E14-1.7.3.3 Transfer of Force at Base of Stem

Specification requires that the transfer of lateral forces from the stem to the footing be in accordance with the shear-transfer provisions of **LRFD [5.8.4]**. That calculation will not be presented. Refer to E13-1.9.3 for a similar computation.

E14-1.7.4 Temperature and Shrinkage Steel

E14-1.7.4.1 Temperature and Shrinkage Steel for Footing

The footing will not be exposed to daily temperature changes. Thus temperature and shrinkage steel is not required. However, #4 bars at 18" o.c. (max) are placed longitudinally to serve as spacers.

E14-1.7.4.2 Temperature and Shrinkage Steel of Stem

The stem will be exposed to daily temperature changes. In accordance with **LRFD [5.10.8]** the stem shall provide temperature and shrinkage steel on each face and in each direction as calculated below:

$s = 18.0$ in (bar spacing)

$Bar_{No} = 4$ (bar size)

$Bar_D = 0.50$ in (temperature and shrinkage bar diameter)

$Bar_A = 0.20$ in^2 (temperature and shrinkage bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$ (temperature and shrinkage provided)

$\boxed{A_s = 0.13}$ in^2/ft

$b_s = (H - D)\ 12$ least width of stem $\boxed{b_s = 216.0}$ in

$h_s = T_t\ 12$ least thickness of stem $\boxed{h_s = 12.0}$ in

$$A_{ts} = \frac{1.3\ b_s\ h_s}{2\left(b_s + h_s\right)\ f_y}$$ Area of reinforcement per foot, on each face and in each direction $\boxed{A_{ts} = 0.12}$ in^2/ft

Is $0.11 \leq A_s \leq 0.60$? $\boxed{check = \text{"OK"}}$

Is $A_s > A_{ts}$? $\boxed{check = \text{"OK"}}$

Table of Contents

E14-4 Cast-In-Place Concrete Cantilever Wall on Piles, LRFD

General

This example shows design calculations for a cast-in-place (CIP) concrete wall supported on piles conforming to the LRFD Bridge Design Specifications and the WisDOT Bridge Manual. *(Example is current through LRFD Sixth Edition - 2012)*

Sample design calculations for pile capacities and wall strength design will be presented. The overall stability and settlement calculations will not be shown in this example, but are required.

Design steps presented in 14.5.2.1 are used for the wall design.

E14-4.1 Establish Project Requirements

The CIP concrete wall shown in Figure E14-4.1-1 will be designed appropriately to accommodate a horizontal backslope. External stability, overall stability and wall strength are the designer's (WisDOT/Consultant) responsibility.

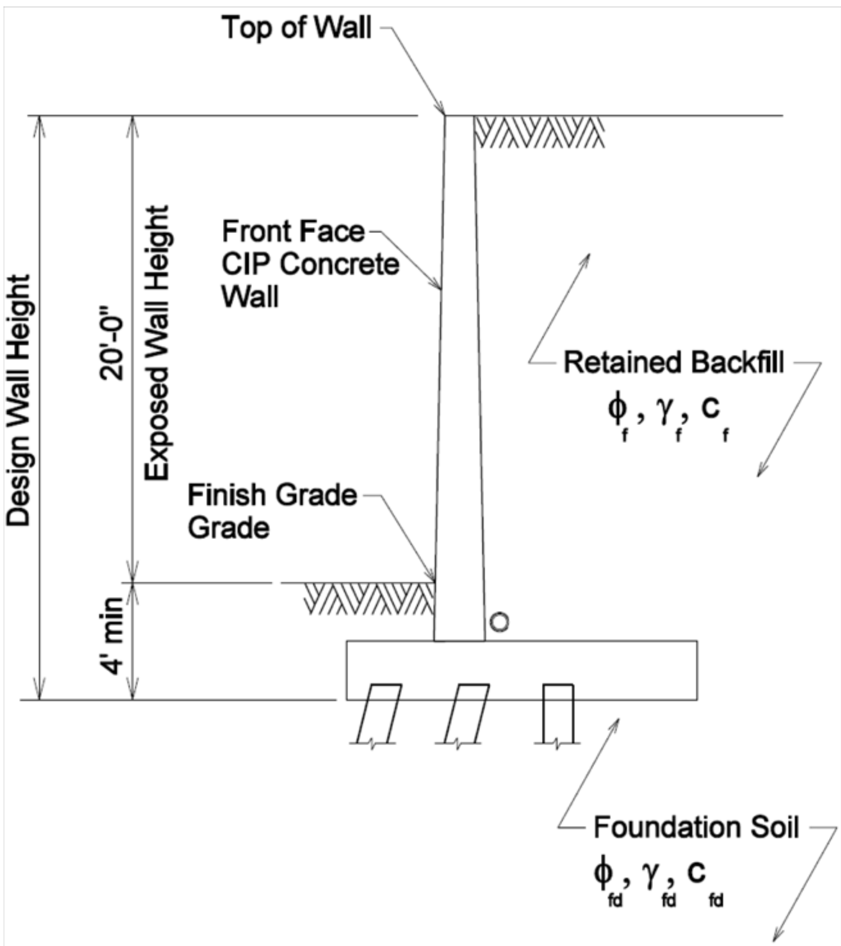


Figure E14-4.1-1
CIP Concrete Wall on Piles

E14-4.2 Design Parameters

Project Parameters

$Design\_Life = 75\ years$ Wall design life (min) **LRFD [11.5.1]**

Soil Properties (From Geotechnical Site Investigation Report)

Designer to determine if long-term or short-term soil strength parameters govern external stability.

Backfill Soil Design Parameters

$\phi_f = 32\ deg$ Angle of internal friction

$\gamma_f = 0.120$ Unit weight, kcf

$c_f = 0$ Cohesion, ksf

$\delta = 17\ deg$ Friction angle between fill and wall

Note: Per WisDOT Bridge Manual and Standard Specifications, structural backfill shall be granular and non-expansive.

ϕ_f= 32 degrees is used for this example, however ϕ_f=30 degrees is the maximum that should be used without testing.

Foundation Soil Design Parameters

$\phi_{fd} = 29\ deg$ Angle of internal friction

$\gamma_{fd} = 0.110$ Unit of weight, kcf

$c_{fd} = 0$ Cohesion, ksf

Reinforced Concrete Parameters

$f'_c = 3.5$ Concrete compressive design strength, ksi (14.5.9)

$\gamma_c = 0.150$ $w_c = \gamma_c$ Unit weight of concrete, ksf

$E_c = 33000\ w_c^{1.5}\sqrt{f'_c}$ Modulus of elasticity of concrete, ksi **LRFD [5.4.2.4]**

$\boxed{E_c = 3587}$ ksi

$f_y = 60$ Yield strength of reinforcing bars, ksi (14.5.9)

$E_s = 29000$ Modulus of elasticity of reinforcing bars, ksi

Live Load Surcharge Parameters

Live load surcharge shall be used when vehicular load is located within H/2 of the backface of the wall **LRFD [3.11.6.4].** The equivalent height of soil for vehicular load, H_{eq}, used for surcharge loads shall be in accordance to **LRFD [Table 3.11.6.4-2]**. However, WisDOT policy for most cases requires an equivalent height of 2.0 feet. The following procedure is used for determining live load surcharge:

$L_{traffic} = 100.00$ Distance from wall backface to edge of traffic, ft

$\frac{H}{2} = 12.00$ Distance from wall backface where live load surcharge shall be considered in the wall design, ft

Note: The wall height used is the exposed height plus an assumed 4 feet embedment ($H=H_e+4$ feet).

Shall live load surcharge be included? $\boxed{check = "NO"}$

$h_{eq} = 0.833$ Equivalent height of soil for surcharge load, ft (14.4.5.4.2)

WisDOT Policy: Wall with live load from traffic use 2.0 feet (240 psf) and walls without traffic use 0.833 feet (100 psf)

E14-4.3 Define Wall Geometry

Wall Geometry

$H_e = 20.00$ Exposed wall height, ft

$D_f = 4.00$ Footing cover, ft (WisDOT policy 4'-0" minimum)

$H = H_e + D_f$ Design wall height, ft

$T_t = 1.00$ Stem thickness at top of wall, ft

$b_1 = 0.25$ Front wall batter, in/ft (b_1H:12V)

$b_2 = 0.50$ Back wall batter, in/ft (b_2H:12V)

$\beta = 0.00\,deg$ Inclination of ground slope behind face of wall, deg (horizontal)

E14-4.6 Evaluate External Stability of Wall

Three potential external failure mechanisms will be considered in this example. These failures include pile bearing resistance, limiting eccentricity and lateral resistance. Global (overall) stability requirements are assumed to have been satisfied in prior calculations. Design calculations will be carried out for the governing limit states only.

E14-4.6.1 Pile Bearing Resistance

Axial and lateral pile capacities from Geotechnical Site Investigation Report:

$Pile\_Axial = 220$ Pile axial capacity, kips

$pile\_batter = 4$ Pile batter (pile_batter V:1H)

$H_{r1} = 11.00$ Battered pile row 1 lateral capacity, kips/pile

$H_{r2} = 11.00$ Battered pile row 2 lateral capacity, kips/pile

$H_{r3} = 14.00$ Vertical pile row 3 lateral capacity, kips/pile

Determine the horizontal and vertical components of the battered pile

$$pile\_angle = \operatorname{atan}\left(\frac{1}{pile\_batter}\right) \qquad \boxed{pile\_angle = 14.0\,deg}$$

$$P_{Rb\_H} = Pile\_Axial \sin(pile\_angle) \qquad \boxed{P_{Rb\_H} = 53.4} \text{ kips/pile}$$

$$P_{Rb\_V} = Pile\_Axial \cos(pile\_angle) \qquad \boxed{P_{Rb\_V} = 213.4} \text{ kips/pile}$$

Calculate axial capacity of battered piles

$$P_R = P_{Rb\_V} \qquad \boxed{P_R = 213.4} \text{ kips/pile}$$

$$P_u = \max(P_{U1a}, P_{U2a}, P_{U1b}, P_{U2b}) \qquad \boxed{P_u = 100.4} \text{ kips/pile}$$

$$CDR_{Brg\_B\_Pile} = \frac{P_R}{P_u} \qquad \boxed{CDR_{Brg\_B\_Pile} = 2.13}$$

Is the $CDR \geq 1.0$? $\boxed{check = \text{"OK"}}$

Calculate axial capacity of vertical piles

$$P_R = Pile\_Axial \qquad \boxed{P_R = 220.0}$$

$$P_u = \max(P_{U3a}, P_{U3b}) \qquad \boxed{P_u = 150.2}$$

$$CDR_{Brg\_V\_Pile} = \frac{P_R}{P_u} \qquad \boxed{CDR_{Brg\_V\_Pile} = 1.46}$$

Is the $CDR \geq 1.0$? $\boxed{check = \text{"OK"}}$

E14-4.6.2 Pile Sliding Resistance

For sliding failure, the horizontal force effects, H_u, is checked against the sliding resistance, H_R, where $H_R=\phi H_n$. The following calculations are based on **Strength Ia**:

Factored Lateral Force, H_u

$H_u = H\_Ia$ $\boxed{H_u = 15.6}$ kip/ft

Sliding Resistance, H_R

It is assumed that the P-y method was used for the pile analysis (LPILE), thus group effects shall be considered. Calculate sliding capacity of the effective pile group per **LRFD [Table-10.7.2.4-1]:**

$B_{yy} = 11.78$ Depth of pile, in

$$\frac{PS1 + PS2}{\frac{B_{yy}}{12}} = 5.86$$ Say:5B

Note: It was assumed that pile row 1 and 3 are aligned throughout the pile group and that pile row 2 will not effect the lateral pile group resistance. Pile row 1 and 3 will then be applied row 1 and 2 "5B" multipliers, respectfully.

"5B" Pile multipliers

$row1 = 1.00$

$row2 = 1.00$

$row3 = 0.80$

Lateral group resistance

$H_{R1} = row1\ H_{r1}\ NP_1 + row2\ H_{r2}\ NP_2 + row3\ H_{r3}\ NP_3$ $\boxed{H_{R1} = 4.15}$ kip/ft

Batter resistance

$H_{R2} = P_{Rb\_H}\left(NP_1 + NP_2\right)$ $\boxed{H_{R2} = 13.34}$ kip/ft

Compute factored resistance against failure by sliding, R_R

$H_R = H_{R1} + H_{R2}$ $\boxed{H_R = 17.49}$ kip/ft

Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} = \frac{H_R}{H_u}$$ $\boxed{CDR_{Sliding} = 1.12}$

Is the $CDR \geq 1.0$? $\boxed{check = \text{"OK"}}$

E14-4.7 Evaluate Wall Structural Design

Note: CIP concrete walls are a non-proprietary wall system and the structural design computations shall be performed by the wall designer.

Wall structural design computations for shear and flexure will be considered in this example. Crack control and temperature and shrinkage considerations will also be included.

E14-4.7.1 Evaluate Wall Footing

E14-4.7.1.1 Evalute One-Way Shear

Design for one-way shear in only the transverse direction.

Compute the effective shear depth,d_v, for the heel:

$cover = 2.0$ in

$s = 9.0$ in (bar spacing)

$Bar_{No} = 7$ (transverse bar size)

$Bar_D = 0.875$ in (transverse bar diameter)

$Bar_A = 0.600$ in^2 (transverse bar area)

$$A_{s\_heel} = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_{s\_heel} = 0.80} \ in^2/ft$$

$$d_{s\_heel} = D \cdot 12 - cover - \frac{Bar_D}{2} \qquad \boxed{d_{s\_heel} = 27.6} \ in$$

$$a\_heel = \frac{A_{s\_heel} f_y}{0.85 f'_c b} \qquad \boxed{a\_heel = 1.3} \ in$$

$$d_{v1} = d_{s\_heel} - \frac{a\_heel}{2} \qquad \boxed{d_{v1} = 26.9} \ in$$

$$d_{v2} = 0.9 d_{s\_heel} \qquad \boxed{d_{v2} = 24.8} \ in$$

$$d_{v3} = 0.72 D \cdot 12 \qquad \boxed{d_{v3} = 21.6} \ in$$

$$d_{v\_heel} = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_{v\_heel} = 26.9} \ in$$

Compute the effective shear depth,d_v, for the toe

cover = 6.0	in
s = 9.0	in (bar spacing)
$Bar_{No} = 7$	(transverse bar size)
$Bar_D = 0.88$	in (transverse bar diameter)
$Bar_A = 0.60$	in^2 (transverse bar area)

$$A_{s\_toe} = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_{s\_toe} = 0.80} \quad in^2/ft$$

$$d_{s\_toe} = D\,12 - cover - \frac{Bar_D}{2} \qquad \boxed{d_{s\_toe} = 23.6} \quad in$$

$$a\_toe = \frac{A_{s\_toe}\, f_y}{0.85\, f'_c\, b} \qquad \boxed{a\_toe = 1.3} \quad in$$

$$d_{v1} = d_{s\_toe} - \frac{a\_toe}{2} \qquad \boxed{d_{v1} = 22.9} \quad in$$

$$d_{v2} = 0.9\, d_{s\_toe} \qquad \boxed{d_{v2} = 21.2} \quad in$$

$$d_{v\_toe} = \max\left(d_{v1}, d_{v2}\right) \qquad \boxed{d_{v\_toe} = 22.9} \quad in$$

Determine the distance from Point 'O' to the critical sections:

$$y\_crit\_toe = A\,12 - d_{v\_toe} \qquad \boxed{y\_crit\_toe = 34.1} \quad in$$

$$y\_crit\_heel = B\,12 - C\,12 + d_{v\_heel} \qquad \boxed{y\_crit\_heel = 112.0} \quad in$$

Determine the distance from Point 'O' to the pile limits:

$$y_{v1\_neg} = y_{p1}\,12 - \frac{B_{yy}}{2} \qquad \boxed{y_{v1\_neg} = 9.1} \quad in$$

$$y_{v1\_pos} = y_{p1}\,12 + \frac{B_{yy}}{2} \qquad \boxed{y_{v1\_pos} = 20.9} \quad in$$

$$y_{v2\_neg} = y_{p2}\,12 - \frac{B_{yy}}{2} \qquad \boxed{y_{v2\_neg} = 42.1} \quad in$$

$$y_{v2\_pos} = y_{p2}\,12 + \frac{B_{yy}}{2} \qquad \boxed{y_{v2\_pos} = 53.9} \quad \text{in}$$

$$y_{v3\_neg} = y_{p3}\,12 - \frac{B_{yy}}{2} \qquad \boxed{y_{v3\_neg} = 78.1} \quad \text{in}$$

$$y_{v3\_pos} = y_{p3}\,12 + \frac{B_{yy}}{2} \qquad \boxed{y_{v3\_pos} = 89.9} \quad \text{in}$$

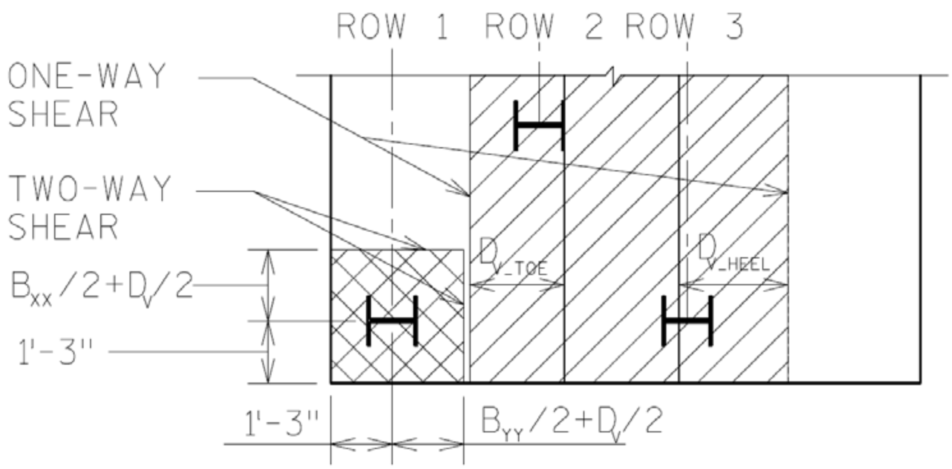


Figure E14-4.7-1
Partial Footing Plan for Critical Shear Sections

Determine if the pile rows are "Outside", "On", or "Inside" the critical sections

Since the pile row 1 falls "Outside" the critical sections, the full row pile reaction will be used for shear

$$P_{U1} = \max\left(P_{U1a}, P_{U1b}\right) \qquad \boxed{P_{U1} = 78.7} \quad \text{kip}$$

$$V_{u\_Pile1} = 1.0\left(P_{U1}\,NP_1\right) \qquad \boxed{V_{u\_Pile1} = 9.8} \quad \text{kip/ft}$$

Since the pile row 2 and 3 falls "Inside" the critical sections, none of the row pile reactions will be used for shear

The load applied to the critical section is based on the proportion of the piles located outside of the critical toe or heel section. In this case, pile row 1 falls outside the toe critical section and the full row pile reaction will be used for shear.

$V_u = V_{u\_Pile1}$ $\boxed{V_u = 9.8}$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

where: $V_c = 0.0316\ \beta \sqrt{f'_c}\ b_v\ d_v$

$V_{n2} = 0.25\ f'_c\ b_v\ d_v$ **LRFD [Eq 5.8.3.3-2]**

Nominal one-way action shear resistance for structures without transverse reinforcement, V_n, is taken as the lesser of V_{n1} and V_{n2}

$\beta = 2.0$

$V_c = 0.0316\ \beta \sqrt{f'_c}\ b\ d_{v\_toe}$

$V_{n1} = V_c$ $\boxed{V_{n1} = 32.5}$ kip/ft

$V_{n2} = 0.25\ f'_c\ b\ d_{v\_toe}$ $\boxed{V_{n2} = 240.3}$ kip/ft

$V_n = \min(V_{n1}, V_{n2})$ $\boxed{V_n = 32.5}$ kip/ft

$\phi_v = 0.90$

$V_r = \phi_v\ V_n$ $\boxed{V_r = 29.2}$ kip/ft

$\boxed{V_u = 9.8}$ kip/ft

Is V_u less than V_r? $\boxed{check = \text{"OK"}}$

E14-4.7.1.2 Evaluate Two-Way Shear

For two-way action around the maximum loaded pile, the pile critical perimeter, b_o, is located a minimum of $0.5d_v$ from the perimeter of the pile. If portions of the critical perimeter are located off the footing, that portion of the critical perimeter is limited by the footing edge.

Two-way action should be checked for the maximum loaded pile.

$V_u = \max(P_{U1a}, P_{U2a}, P_{U3a}, P_{U1b}, P_{U2b}, P_{U3b})$ $\boxed{V_u = 150.2}$ kip

Determine the location of the pile critical perimeter. Assume that the critical section is outside of the footing and only include the portion of the shear perimeter is located within the footing:

$$b_{o\_xx} = 1.25\ 12 + \frac{B_{xx}}{2} + \frac{d_{v\_toe}}{2}$$ $\boxed{b_{o\_xx} = 32.5}$ in

$$b_{o\_yy} = 1.25\ 12 + \frac{B_{yy}}{2} + \frac{d_{v\_toe}}{2}$$ $\boxed{b_{o\_yy} = 32.3}$ in

$$\beta_{c\_pile} = \frac{b_{o\_xx}}{b_{o\_yy}}$$ $\boxed{\beta_{c\_pile} = 1.004}$ in

$$b_{o\_pile} = b_{o\_xx} + b_{o\_yy}$$ $\boxed{b_{o\_pile} = 64.8}$ in

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.13.3.6.3]**

$$V_{n1} = \left(0.063 + \frac{0.126}{\beta_{c\_pile}}\right)\sqrt{f'_c}\, b_{o\_pile}\, d_{v\_toe}$$ $\boxed{V_{n1} = 523.1}$ kip/ft

$$V_{n2} = 0.126\sqrt{f'_c}\, b_{o\_pile}\, d_{v\_toe}$$ $\boxed{V_{n2} = 349.7}$ kip/ft

$$V_n = \min(V_{n1}, V_{n2})$$ $\boxed{V_n = 349.7}$ kip/ft

$$V_r = \phi_v V_n$$ $\boxed{V_r = 314.7}$ kip/ft

$\boxed{V_u = 150.2}$ kip/ft

Is V_u less than V_r? $\boxed{check = \text{"OK"}}$

E14-4.7.1.3 Evaluate Top Transverse Reinforcement Strength

Top transverse reinforcement strength is determined by assuming the heel acts as a cantilever member supporting its own weight and loads acting above it. Pile reactions may be used to decrease this load.

For **Strength Ib**:

$$V_u = 1.25\left(\frac{C}{B} V_4\right) + 1.35\left(V_7 + V_8 + V_9\right) + 1.75\left(V_{10}\right) + 1.50\left(V_{11}\right)$$

$\boxed{V_u = 27.0}$ kip/ft

$$M_u = V_u \frac{C}{2}$$ $\boxed{M_u = 66.3}$ kip-ft/ft

Calculated the capacity of the heel in flexure at the face of the stem:

$$M_n = A_{s\_heel} f_y \left(d_{s\_heel} - \frac{a\_heel}{2}\right)\frac{1}{12}$$ $\boxed{M_n = 107.6}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$c = \frac{a\_heel}{\beta_1}$ $\boxed{c = 1.58}$ in

$$\phi_F = \begin{cases} 0.75 \text{ if } \frac{d_{s\_heel}}{c} < \frac{5}{3} \\ 0.65 + 0.15\left(\frac{d_{s\_heel}}{c} - 1\right) \text{ if } \frac{5}{3} \le \frac{d_{s\_heel}}{c} \le \frac{8}{3} \\ 0.90 \text{ otherwise} \end{cases}$$ $\boxed{\phi_F = 0.90}$

Note: if $\phi_F = 0.75$ Section is compression-controlled
if $0.75 < \phi_F < 0.90$ Section is in transition
if $\phi_F = 0.90$ Section is tension-controlled

Calculate the flexural factored resistance, M_r:

$M_r = \phi_F M_n$ $\boxed{M_r = 96.8}$ kip-ft/ft

$\boxed{M_u = 66.3}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = "OK"}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$f_r = 0.37\sqrt{f'_c}$ $\boxed{f_r = 0.692}$ ksi

$I_g = \frac{1}{12} b (D\ 12)^3$ $\boxed{I_g = 27000}$ in^4

$y_t = \frac{1}{2} D\ 12$ $\boxed{y_t = 15.00}$ in

$S_c = \frac{I_g}{y_t}$ $\boxed{S_c = 1800}$ in^3

$M_{cr} = \gamma_3(\gamma_1 f_r) S_c$ therefore, $M_{cr} = 1.1 f_r S_c$

Where:

$\gamma_1 = 1.6$ flexural cracking variability factor

$\gamma_3 = 0.67$ ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$$M_{cr} = 1.1\ f_r\ S_c\ \frac{1}{12}$$ $\boxed{M_{cr} = 114.2}$ kip-ft/ft

$\boxed{1.33\ M_u = 88.2}$ kip-ft/ft

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

E14-4.7.1.4 Evaluate Bottom Transverse Reinforcement Strength

Bottom transverse reinforcement strength is determined by using the maximum pile reaction.

Determine the moment arms

$$arm\_v1 = A - y_{p1}$$ $\boxed{arm\_v1 = 3.5}$ ft

$$arm\_v2 = A - y_{p2}$$ $\boxed{arm\_v2 = 0.8}$ ft

Determine the moment for **Strength Ia**:

$$V_{u\_1a} = P_{U1a}\ NP_1$$ $\boxed{V_{u\_1a} = 9.8}$ kip/ft

$$V_{u\_2a} = P_{U2a}\ NP_2$$ $\boxed{V_{u\_2a} = 9.5}$ kip/ft

$$M_{u\_Ia} = V_{u\_1a}\ arm\_v1 + V_{u\_2a}\ arm\_v2$$ $\boxed{M_{u\_Ia} = 41.6}$ kip-ft/ft

Determine the moment for **Strength Ib**:

$$V_{u\_1b} = P_{U1b}\ NP_1$$ $\boxed{V_{u\_1b} = 6.8}$ kip/ft

$$V_{u\_2b} = P_{U2b}\ NP_2$$ $\boxed{V_{u\_2b} = 12.5}$ kip/ft

$$M_{u\_Ib} = V_{u\_1b}\ arm\_v1 + V_{u\_2b}\ arm\_v2$$ $\boxed{M_{u\_Ib} = 33.3}$ kip-ft/ft

Determine the design moment:

$$M_u = \max\left(M_{u\_Ia}, M_{u\_Ib}\right)$$ $\boxed{M_u = 41.6}$ kip-ft/ft

Calculated the capacity of the toe in flexure at the face of the stem:

$$M_n = A_{s\_toe}\ f_y\left(d_{s\_toe} - \frac{a\_toe}{2}\right)\frac{1}{12}$$ $\boxed{M_n = 91.6}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$$\beta_1 = 0.85$$

$$c = \frac{a\_toe}{\beta_1}$$ $\boxed{c = 1.58}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \dfrac{d_{s\_toe}}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_{s\_toe}}{c} - 1\right) & \text{if } \dfrac{5}{3} \le \dfrac{d_{s\_toe}}{c} \le \dfrac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

Calculate the flexural factored resistance, M_r:

$M_r = \phi_F\ M_n$ $\boxed{M_r = 82.4}$ kip-ft/ft

$\boxed{M_u = 41.6}$ kip-ft/ft

Is M_u less than M_r? $\boxed{\text{check} = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$f_r = 0.37\sqrt{f'_c}$ $\boxed{f_r = 0.692}$ ksi

$I_g = \frac{1}{12}\ b\ (D\ 12)^3$ $\boxed{I_g = 27000}$ in^4

$y_t = \frac{1}{2}\ D\ 12$ $\boxed{y_t = 15.00}$ in

$S_c = \frac{I_g}{y_t}$ $\boxed{S_c = 1800}$ in^3

$M_{cr} = 1.1\ f_r\ S_c\ \frac{1}{12}$ from E14-4.7.1.3 $\boxed{M_{cr} = 114.2}$ kip-ft/ft

$\boxed{1.33\ M_u = 55.3}$ kip-ft/ft

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

E14-4.7.1.5 Evaluate Longitudinal Reinforcement Strength

The structural design of the longitudinal reinforcement, assuming the footing acts as a continuous beam over pile supports, is calculated using the maximum pile reactions.

Compute the effective shear depth, d_v ,for the longitudinal reinforcement

$cover = 6.0$ in

$s = 12.0$ in (bar spacing)

$Bar_{No} = 5$ (longitudinal bar size)

$Bar_D = 0.625$ in (longitudinal bar diameter)

$Bar_A = 0.310$ in² (longitudinal bar area)

$$A_{s\_long} = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_{s\_long} = 0.31} \text{ in}^2\text{/ft}$$

$$d_s = D\,12 - cover - Bar_{D\_toe} - \frac{Bar_D}{2} \qquad \boxed{d_s = 22.8} \text{ in}$$

$$a\_long = \frac{A_{s\_long}\, f_y}{0.85\, f'_c\, b} \qquad \boxed{a\_long = 0.5} \text{ in}$$

$$d_{v1} = d_s - \frac{a\_long}{2} \qquad \boxed{d_{v1} = 22.6} \text{ in}$$

$$d_{v2} = 0.9\, d_s \qquad \boxed{d_{v2} = 20.5} \text{ in}$$

$$d_{v3} = 0.72\, D\, 12 \qquad \boxed{d_{v3} = 21.6} \text{ in}$$

$$d_{v\_long} = \max\left(d_{v1}, d_{v2}, d_{v3}\right) \qquad \boxed{d_{v\_long} = 22.6} \text{ in}$$

Calculate the design moment using a uniform vertical load:

$$L_{pile} = \max\left(P_1, P_2, P_3\right) \qquad \boxed{L_{pile} = 8.0} \text{ ft}$$

$$w_u = \frac{V\_Ib}{B} \qquad \boxed{w_u = 3.2} \text{ kip/ft/ft}$$

$$M_u = \frac{w_u\, L_{pile}^{\;2}}{10} \qquad \boxed{M_u = 20.3} \text{ kip-ft/ft}$$

Calculated the capacity of the toe in flexure at the face of the stem:

$$M_n = A_{s\_long}\, f_y \left(d_s - \frac{a\_long}{2}\right)\frac{1}{12}$$ $\boxed{M_n = 35.0}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$$\beta_1 = 0.85$$

$$c = \frac{a\_toe}{\beta_1}$$ $\boxed{c = 1.58}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \dfrac{d_s}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_s}{c} - 1\right) & \text{if } \dfrac{5}{3} \le \dfrac{d_s}{c} \le \dfrac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$ $\boxed{\phi_F = 0.90}$

Calculate the flexural factored resistance, M_r:

$$M_r = \phi_F\, M_n$$ $\boxed{M_r = 31.5}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$$f_r = 0.37\sqrt{f'_c}$$ $\boxed{f_r = 0.692}$ ksi

$$I_g = \frac{1}{12}\, b\,(D\,12)^3$$ $\boxed{I_g = 27000}$ in^4

$$y_t = \frac{1}{2}\, D\, 12$$ $\boxed{y_t = 15.00}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1800}$ in^3

$$M_{cr} = 1.1\, f_r\, S_c\, \frac{1}{12}$$ from E14-4.7.1.3 $\boxed{M_{cr} = 114.2}$ kip-ft/ft

$\boxed{1.33\, M_u = 27.1}$ kip-ft/ft

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

E14-4.7.2 Evaluate Stem Strength

Unfactored Stem Horizontal Loads and Moments:

$H_1 = \gamma_f h_{eq} h' k_a \cos(90\ deg - \theta + \delta)$ $\boxed{H_1 = 0.6}$ kip/ft

$H_2 = \frac{1}{2} \gamma_f h'^2 k_a \cos(90\ deg - \theta + \delta)$ $\boxed{H_2 = 7.7}$ kip/ft

$M_1 = H_1 \left(\frac{h'}{2}\right)$ $\boxed{M_1 = 6.4}$ kip-ft/ft

$M_2 = H_2 \left(\frac{h'}{3}\right)$ $\boxed{M_2 = 55.2}$ kip-ft/ft

Factored Stem Horizontal Loads and Moments:

for **Strength Ib**:

$H_{u1} = 1.75\ H_1 + 1.50\ H_2$ $\boxed{H_{u1} = 12.6}$ kip/ft

$M_{u1} = 1.75\ M_1 + 1.50\ M_2$ $\boxed{M_{u1} = 94.0}$ kip-ft/ft

for **Service I**:

$H_{u3} = 1.00\ H_1 + 1.00\ H_2$ $\boxed{H_{u3} = 8.3}$ kip/ft

$M_{u3} = 1.00\ M_1 + 1.00\ M_2$ $\boxed{M_{u3} = 61.6}$ kip-ft/ft

E14-4.7.2.1 Evaluate Stem Shear Strength at Footing

$V_u = H_{u1}$ $\boxed{V_u = 12.6}$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

where: $V_c = 0.0316\ \beta \sqrt{f'_c}\ b_v\ d_v$

$V_{n2} = 0.25\ f'_c\ b_v\ d_v$ **LRFD [Eq 5.8.3.3-2]**

Compute the shear resistance due to concrete, V_c :

$cover = 2.0$ in

$s = 12.0$ in (bar spacing)

$Bar_{No} = 9$ (transverse bar size)

$Bar_D = 1.13$ in (transverse bar diameter)

$Bar_A = 1.00$ in^2 (transverse bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$ $\boxed{A_s = 1.00}$ in^2/ft

$$d_s = T_b\ 12 - cover - \frac{Bar_D}{2}$$ $\boxed{d_s = 25.6}$ in

$$a = \frac{A_s\ f_y}{0.85\ f'_c\ b}$$ $\boxed{a = 1.7}$ in

$$d_{v1} = d_s - \frac{a}{2}$$ $\boxed{d_{v1} = 24.7}$ in

$$d_{v2} = 0.9\ d_s$$ $\boxed{d_{v2} = 23.0}$ in

$$d_{v3} = 0.72\ T_b\ 12$$ $\boxed{d_{v3} = 20.3}$ in

$$d_v = \max(d_{v1}, d_{v2}, d_{v3})$$ $\boxed{d_v = 24.7}$ in

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2}

$$\beta = 2.0$$

$$V_c = 0.0316\ \beta\ \sqrt{f'_c}\ b\ d_v$$

$$V_{n1} = V_c$$ $\boxed{V_{n1} = 35.1}$ kip/ft

$$V_{n2} = 0.25\ f'_c\ b\ d_v$$ $\boxed{V_{n2} = 259.6}$ kip/ft

$$V_n = \min(V_{n1}, V_{n2})$$ $\boxed{V_n = 35.1}$ kip/ft

$$V_r = \phi_v\ V_n$$ $\boxed{V_r = 31.6}$ kip/ft

$\boxed{V_u = 12.6}$ kip/ft

Is V_u less than V_r? $\boxed{check = \text{"OK"}}$

E14-4.7.2.2 Evaluate Stem Flexural Strength at Footing

$$M_u = M_{u1}$$ $\boxed{M_u = 94.0}$ kip-ft/ft

Calculate the capacity of the stem in flexure at the face of the footing:

$$M_n = A_s\ f_y \left(d_s - \frac{a}{2}\right) \frac{1}{12}$$ $\boxed{M_n = 123.6}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$$c = \frac{a}{\beta_1}$$ $\boxed{c = 1.98}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \frac{d_s}{c} < \frac{5}{3} \\ 0.65 + 0.15\left(\frac{d_s}{c} - 1\right) & \text{if } \frac{5}{3} \le \frac{d_s}{c} \le \frac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$ $\boxed{\phi_F = 0.90}$

Calculate the flexural factored resistance, M_r:

$M_r = \phi_F\ M_n$ $\boxed{M_r = 111.2}$ kip-ft/ft

$\boxed{M_u = 94.0}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = "OK"}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$f_r = 0.37\sqrt{f'_c}$ $\boxed{f_r = 0.69}$ ksi

$$I_g = \frac{1}{12}\ b\ (T_b\ 12)^3$$ $\boxed{I_g = 22247}$ in^4

$$y_t = \frac{1}{2}\ T_b\ 12$$ $\boxed{y_t = 14.1}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1582}$ in^3

$$M_{cr\_s} = 1.1\ f_r\ S_c\ \frac{1}{12}$$ from E14-4.7.1.3 $\boxed{M_{cr\_s} = 100.4}$ kip-ft/ft

$\boxed{1.33\ M_u = 125.0}$ kip-ft/ft

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

Check the Service Ib crack control requirements in accordance with **LRFD [5.7.3.4]**

$$\rho = \frac{A_s}{d_s\ b} \qquad \boxed{\rho = 0.00326}$$

$$n = \frac{E_s}{E_c} \qquad \boxed{n = 8.09}$$

$$k = \sqrt{(\rho\ n)^2 + 2\ \rho\ n} - \rho\ n \qquad \boxed{k = 0.205}$$

$$j = 1 - \frac{k}{3} \qquad \boxed{j = 0.932}$$

$$d_c = cover + \frac{Bar_D}{2} \qquad \boxed{d_c = 2.6}\ \text{in}$$

$$f_{ss} = \frac{M_{u3}}{A_s\ j\ d_s}\ 12 \qquad \boxed{f_{ss} = 31.0}\ \text{ksi}$$

$$h = T_b\ 12$$

$$\beta_s = 1 + \frac{d_c}{0.7\left(h - d_c\right)} \qquad \boxed{\beta_s = 1.1}$$

$\gamma_e = 1.00$ for Class 1 exposure

$$s_{max} = \frac{700\ \gamma_e}{\beta_s\ f_{ss}} - 2\ d_c \qquad \boxed{s_{max} = 14.6}\ \text{in}$$

$$\boxed{s = 12.0}\ \text{in}$$

Is the bar spacing less than s_{max}? $\boxed{check = \text{"OK"}}$

E14-4.7.2.3 Transfer of Force at Base of Stem

Specification requires that the transfer of lateral forces from the stem to the footing be in accordance with the shear-transfer provisions of **LRFD [5.8.4]**. That calculation will not be presented. Refer to E13-1.9.3 for a similar computation.

E14-4.7.3 Temperature and Shrinkage Steel

E14-4.7.3.1 Temperature and Shrinkage Steel for Footing

The footing will not be exposed to daily temperature changes. Thus temperature and shrinkage steel is not required.

E14-4.7.3.2 Temperature and Shrinkage Steel of Stem

The stem will be exposed to daily temperature changes. In accordance with AASTHO **LRFD [5.10.8]** the stem shall provide temperature and shrinkage steel on each face and in each direction as calculated below:

$s = 18.0$ in (bar spacing)

$Bar_{No} = 4$ (bar size)

$Bar_A = 0.20$ in^2 (temperature and shrinkage bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$ (temperature and shrinkage provided)

$\boxed{A_s = 0.13}$ in^2/ft

$b_s = (H - D)\,12$ least width of stem $\boxed{b_s = 258.0}$ in

$h_s = T_t\,12$ least thickness of stem $\boxed{h_s = 12.0}$ in

$$A_{ts} = \frac{1.3\,b_s\,h_s}{2\left(b_s + h_s\right)f_y}$$ Area of reinforcement per foot, on each face and in each direction $\boxed{A_{ts} = 0.12}$ in^2/ft

Is $0.11 \le A_s \le 0.60$? $\boxed{check = \text{"OK"}}$

Is $A_s > A_{ts}$? $\boxed{check = \text{"OK"}}$

Check the maximum spacing requirements

$s_1 = \min\left(3\,h_s, 18\right)$ $\boxed{s_1 = 18.0}$ in

$$s_2 = \begin{vmatrix} 12 & \text{if} \quad h_s > 18 \\ s_1 & \text{otherwise} \end{vmatrix}$$ For walls and footings (in) $\boxed{s_2 = 18.0}$ in

$s_{max} = \min\left(s_1, s_2\right)$ $\boxed{s_{max} = 18.0}$ in

Is the bar spacing less than s_{max}? $\boxed{check = \text{"OK"}}$

E14-4.8 Summary of Results

E14-4.8.1 Summary of External Stability

Based on the defined project parameters the following external stability checks have been satisfied:

External Check	CDR
	Strength I
Bearing	1.46
Eccentricity	> 10
Sliding	1.12

Table E14-4.8-1
Summary of External Stability Computations

E14-4.8.2 Summary of Wall Strength Design

The required wall reinforcing from the previous computations are presented in Figure E14-6.9-1.

E14-4.8.3 Drainage Design

Drainage requirements shall be investigated and detailed accordingly. In this example drainage requirements are met by providing granular, free draining backfill materail with a pipe underdrain located at the bottom of the wall (Assumed wall is adjacent to sidewalk) as shown in Figure E14-4.9-1.

E14-4.9 Final Cast-In-Place Concrete Wall Schematic

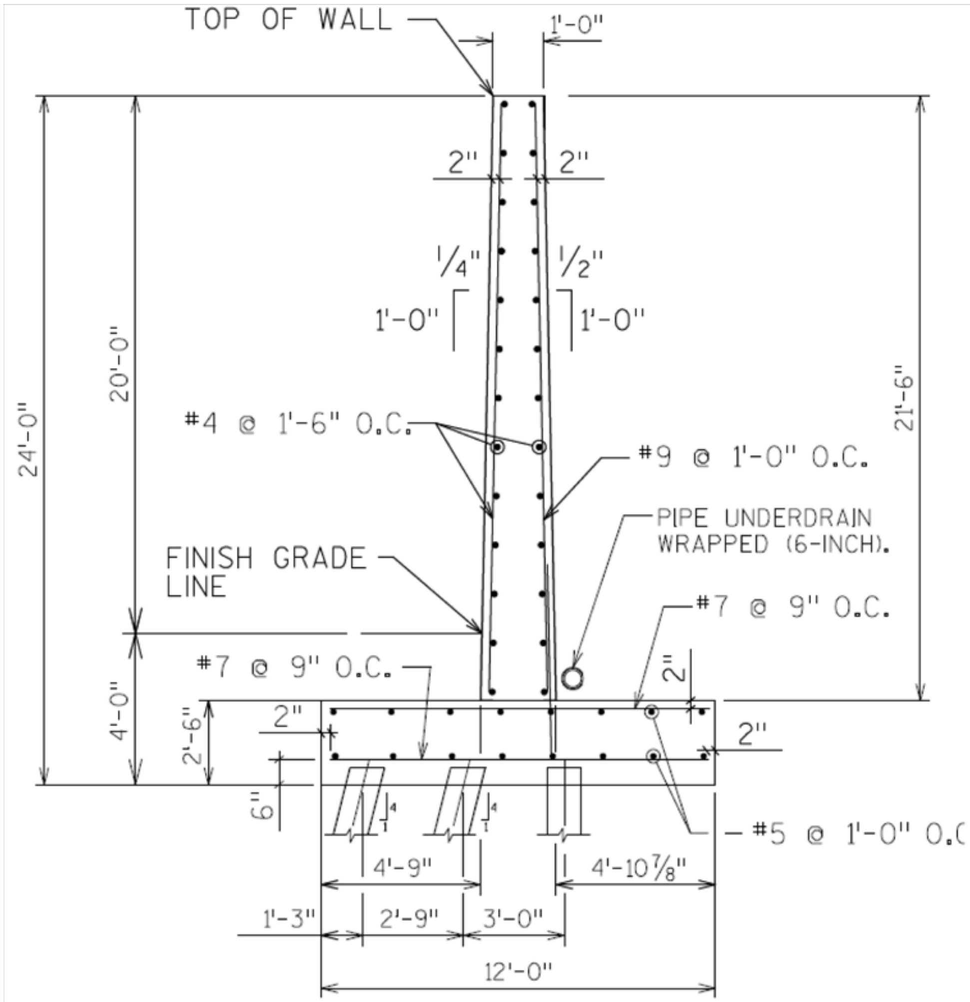


Figure E14-4.9-1
Cast-In-Place Wall Schematic

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Table of Contents

LL#3	Double truck [90% of two design trucks (+ IM) + 90% of design lane load] *	**LRFD [3.6.1.3.1]**
LL#4	Fatigue truck (+ IM)	**LRFD [3.6.1.4.1]**
LL#5	Design truck (+ IM)	**LRFD [3.6.1.3.2]**
LL#6	25% [design truck (+ IM)] + design lane load	**LRFD [3.6.1.3.2]**

* LL#3 is used to calculate negative live load moments between points of contraflexure, as well as reactions at interior supports.

Table 17.2-2
Live Load Combinations

The live load combinations are applied to the limit states as follows:

Strength I – The live load force effect, Qi, shall be taken as the larger of LL#1, LL#2 and LL#3.

Strength V – The live load force effect, Qi, shall be taken as the larger of LL#1, LL#2 and LL#3.

Service I – The live load force effect, Qi, shall be taken as the larger of LL#1, LL#2 and LL#3. However, for live load deflection criteria, the force effects shall be taken as the larger of LL#5 and LL#6.

Service II – The live load force effect, Qi, shall be taken as the larger of LL#1, LL#2 and LL#3.

Service III – The live load force effect, Qi, shall be taken as the larger of LL#1, LL#2 and LL#3.

Fatigue (I or II) – The live load force effect, Qi, shall be from a single fatigue truck, LL#4.

Extreme Event II – The live load force effect, Qi, shall be taken as the larger of LL#1 and LL#2.

17.2.4.3 Multiple Presence Factor

The extreme force effect shall be determined by considering each possible combination of the number of loaded lanes multiplied by a corresponding multiple presence factor. This factor accounts for the probability of simultaneous lane occupation by the full HL93 design live load. Note that the multiple presence factor has been included in the approximate equations for distribution factors in **LRFD [4.6.2.2]** and **[4.6.2.3]**, and in 17.2.8 of this manual.

As described in **LRFD [3.6.1.1.2]**, the multiple presence factors, m, have the values as presented in Table 17.2-3

Number of Loaded Lanes	Multiple Presence Factors "m"
1	1.20
2	1.00
3	0.85
>3	0.65

Table 17.2-3
Multiple Presence Factors

17.2.4.4 Dynamic Load Allowance

The HL-93 loading is based on a static live load applied to the bridge. However, in reality, the live load is not static but is moving across the bridge. Since the roadway surface on a bridge is usually not perfectly smooth and the suspension systems of most trucks react to roadway roughness with oscillations, a dynamic load is applied to the bridge and must also be considered with the live load. This is referred to as dynamic load allowance.

As described in **LRFD [3.6.2]**, the dynamic load allowance has values as presented in Table 17.2-4.

Component	Limit State	Dynamic Load Allowance, IM
Deck joints	All limit states	75%
All other components	Fatigue and fracture limit states	15%
	All other limit states	33%

Table 17.2-4
Dynamic Load Allowance

Applying these specifications to the live load combinations listed in Table 17.2-2:

IM = 15% for fatigue truck (LL#4)

IM = 33% for all other live load combinations (LL#1, LL#2, LL#3, LL#5 and LL#6)

Where IM is required, multiply the loads by (1 + IM/100) to include the dynamic effects of the load.

It is important to note that the dynamic load allowance is applied only to the design truck and design tandem. The dynamic load allowance is not applied to the design lane load or to pedestrian loads.

17.2.4.5 Pedestrian Loads

For bridges designed for both vehicular and pedestrian load, a pedestrian load of 75 psf is used, as specified in **LRFD [3.6.1.6]**. However, for bridges designed exclusively for pedestrian 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)	0.70
	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.

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 [5.7.3.3.2] 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 [5.7.3.3.2]** 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 [5.7.3.3.2]**.

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 [5.7.3.4]** 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 both concrete and mortar or 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 cast-in-place concrete under the panel. The polystyrene is cut to the required haunch height so a constant slab thickness is maintained. High-density expanded polystyrene is available in different strengths, and it is the responsibility of the contractor to determine the strength required based on the vertical load that must be resisted. 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.

When panels are supported on grout, 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 immediately before placement of the panels. It is important that enough grout be placed so that the vertical load from the panels is supported by the grout and not by the polystyrene.

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

Table of Contents

[2] (LL#3) is used to calculate negative live load moments between points of contraflexure and also reactions at interior supports. The points of contraflexure are located by placing a uniform load across the entire structure. For these moments and reactions, the results calculated from (LL#3) are compared with (LL#1) and (LL#2) results, and the critical value is selected.

[3] Used for design of interior strip only.

18.4.3.2 Pedestrian Live Load (PL)

For bridges designed for both vehicular and pedestrian live load, a pedestrian live load, PL, of 75 psf is used. However, for bridges designed exclusively for pedestrian and/or bicycle traffic, see *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges* for live load. The dynamic load allowance, IM, is not applied to pedestrian live loads **LRFD [3.6.2]**.

Pedestrian loads are not applied to an interior strip for its design. For the design of exterior strips (edge beams), any pedestrian loads that are located directly over the exterior strip width and on the cantilevered portion of the sidewalk, shall be applied to the exterior strip. See 17.2.7 for the distribution of pedestrian live loads.

18.4.4 Minimum Slab Thickness Criteria

Check adequacy of chosen slab thickness by looking at live load deflection and dead load deflection (camber) criteria, using Service I Limit State.

18.4.4.1 Live Load Deflection Criteria

All concrete slab structures shall be designed to meet live load deflection limits **LRFD [2.5.2.6.2]**. Live load deflections for concrete slab structures are limited to L/1200, by the Bureau of Structures. The live load deflection, Δ_{LL+IM}, shall be calculated using factored loads described in 18.3.4.1 and 18.4.3.1 for Service I Limit State.

Place live loads in each design lane **LRFD [3.6.1.1.1]** and apply a multiple presence factor **LRFD [3.6.1.1.2]**. Use gross moment of inertia, I_g, based on entire slab width acting as a unit. Use modulus of elasticity E_c = 3800 ksi, see 18.2.2. The factored resistance, R_r, is described in 18.3.4.2.2.

Then check that, $\Delta_{LL+IM} \leq R_r$ is satisfied.

18.4.4.2 Dead Load Deflection (Camber) Criteria

All concrete slab structures shall be designed to meet dead load deflection (camber) limits **LRFD [5.7.3.6.2]**. Dead load deflections for concrete slab structures are computed using the gross moment of inertia, I_g. All dead loads are to be uniformly distributed across the width of the slab. These deflections are increased to provide for the time-dependent deformations of creep and shrinkage. Bureau of Structures currently calculates full camber as three times the dead load deflection. Most of the excess camber is dissipated during the first year of service, which is the time period that the majority of creep and shrinkage deflection occurs.

Noticeable excess deflection or structure sag can normally be attributed to falsework settlement. Use modulus of elasticity E_c = 3800 ksi, see 18.2.2 . The dead load deflection, Δ_{DL} , shall be calculated using factored loads described in 18.3.4.1 and 18.4.2. The factored resistance, R_r , is described in 18.3.4.2.3.

WisDOT exception to AASHTO:

Calculating full camber as three times the dead load deflection, as stated in paragraph above, is an exception to **LRFD [5.7.3.6.2]**. This exception, used by the Bureau of Structures, is based on field observations using this method.

Then check that, $\Delta_{DL} \leq R_r$ is satisfied.

A "Camber Diagram" is shown in the plans on the "Superstructure" sheet. Provide camber values, as well as centerline and edge of slab elevations, at 0.1 points of all spans.

Simple-Span Concrete Slabs:

Maximum allowable camber for simple-span slabs is limited to 2 ½ inches. For simple-span slabs, Bureau of Structures practice indicates that using a minimum slab depth (ft) from the equation 1.1(S + 10) / 30, (where S is span length in feet), and meeting the live load deflection and dead load deflection (camber) limits stated in this section, provides an adequate slab section for most cases.

WisDOT exception to AASHTO:

The equation for calculating minimum slab depth for simple-spans, as stated in paragraph above, is an exception to **LRFD [Table 2.5.2.6.3-1]**. This exception, used by the Bureau of Structures, is based on past performance using this equation.

Continuous-Span Concrete Slabs:

Maximum allowable camber for continuous-span slabs is 1 ¾ inches.

18.4.5 Live Load Distribution

Live loads are distributed over an equivalent width, E, as calculated below. The equivalent distribution width applies for both live load moment and shear.

18.4.5.1 Interior Strip

Equivalent interior strip widths for slab bridges are covered in **LRFD [4.6.2.1.2, 4.6.2.3]**.

The live loads to be placed on these widths are axle loads (i.e., two lines of wheels) and the full lane load.

Single-Lane Loading: $E = 10.0 + 5.0\ (L_1 W_1)^{1/2}$

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 permanent loads, fatigue shall be considered only if this compressive stress is less than 1.5 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 1.5 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, 5.7.1, 5.7.3.4]**.

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:

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

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)

f_{ss} = tensile stress in steel reinforcement (ksi); 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 [5.7.3.3.2]**.

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 = $0.37 (f'c)^{1/2}$ 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

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.

Then check that, $M_L \leq M_r$ is satisfied.

18.4.6.5 Bar Cutoffs

One-half of the bar steel reinforcement required for maximum moment can be cut off at a point, where the remaining one-half has the moment capacity, or factored resistance, M_r , equal to the total factored moment, M_u , at that point. This is called the theoretical cutoff point.

Select tentative cutoff point at theoretical cutoff point or at a distance equal to the development length from the point of maximum moment, whichever is greater. The reinforcement is extended beyond this tentative point for a distance equal to the effective depth of the slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. This cutoff point is acceptable, if it satisfies fatigue and crack control criteria. The continuing bars must be fully developed at this point **LRFD [5.11.1.2.1]**.

18.4.6.5.1 Positive Moment Reinforcement

At least one-third of the maximum positive moment reinforcement in simple-spans and one-fourth of the maximum positive moment reinforcement in continuous-spans is extended along the same face of the slab beyond the centerline of the support **LRFD [5.11.1.2.2]**.

18.4.6.5.2 Negative Moment Reinforcement

For negative moment reinforcement, the second tentative cutoff point is at the point of inflection. At least one-third of the maximum negative moment reinforcement must extend beyond this point for a distance equal to the effective depth of the slab, 12 bar diameters, or 1/16 of the clear span, whichever is greater **LRFD [5.11.1.2.3]**.

18.4.7 Transverse Slab Reinforcement

18.4.7.1 Distribution Reinforcement

Distribution reinforcement is placed transversely in the bottom of the slab, to provide for lateral distribution of concentrated loads **LRFD [5.14.4.1]**. The criteria for main reinforcement parallel to traffic is applied. The amount of distribution reinforcement is to be determined as a percentage of the main reinforcing steel required for positive moment as given by the following formula:

$$\text{Percentage} = \frac{100\%}{\sqrt{L}} \leq \ 50\% \text{ maximum}$$

Where:

L = span length (ft)

The above formula is conservative when applied to slab structures. This specification was primarily drafted for the relatively thin slabs on stringers.

18.4.7.2 Reinforcement in Slab over Piers

If the concrete superstructure rests on a pier cap (with columns) or directly on columns, design of transverse slab reinforcement over the pier is required. A portion of the slab over the pier is designed as a continuous transverse slab member (beam) along the centerline of the substructure. The depth of the assumed section is equal to the depth of the slab or haunch when the superstructure rests directly on columns. When the superstructure rests on a pier cap and the transverse slab member and pier cap act as a unit, the section depth will include the slab or haunch depth plus the cap depth. For a concrete slab, the width of the transverse slab member is equal to one-half the center to center spacing between columns (or 8 foot maximum) for the positive moment zone. The width equals the diameter of the column plus 6 inches for negative moment zone when no pier cap is present. The width equals the cap width for negative moment zone when a pier cap is present. Reference is made to the design example in 18.5 of this chapter for computations relating to transverse reinforcement in slab over the piers.

18.4.8 Shrinkage and Temperature Reinforcement

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperature changes and in structural mass concrete.

The area, A_s , of reinforcement per foot for shrinkage and temperature effects, on each face and in each direction shall satisfy: **LRFD [5.10.8]**

$$A_s \geq 1.30\ (b)\ (h)\ /\ 2\ (b+h)\ (f_y) \quad \text{and} \quad 0.11 \leq A_s \leq 0.60$$

Where:

A_s	=	area of reinforcement in each direction and on each face (in^2/ft)
b	=	least width of component section (in)
h	=	least thickness of component section (in)
f_y	=	specified yield strength of reinforcing bars (ksi) $\leq$ 75 ksi

Shrinkage and temperature reinforcement shall not be spaced farther apart than 3.0 times the component thickness or 18 inches. For components greater than 36 inches thick, the spacing shall not exceed 12 inches.

All longitudinal reinforcement and transverse reinforcement in the slab must exceed required A_s (on each face and in each direction), and not exceed maximum spacing.

18.4.9 Shear Check of Slab

Slab bridges 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 [5.14.4.1]**.

18.4.10 Longitudinal Reinforcement Tension Check

The tensile capacity check of longitudinal reinforcement on the flexural tension side of a member is detailed in **LRFD [5.8.3.5]**.

The area of longitudinal reinforcement (in bottom of slab), A_s , should be checked for tensile capacity at the abutments, for dead load and (HL-93) live load on interior and exterior strips. The reinforcement at these locations shall have the capacity to resist the tension in the reinforcement produced by shear.

The factored shear, V_u , shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. The factored tension force, T_{fact} , from shear, to be resisted is from **LRFD [Eq'n. 5.8.3.5-2]**, where $V_s = V_p = 0$, is:

$$T_{fact} = [\ V_u / \phi_v\ ] \cot \theta$$

Assume a diagonal crack would start at the inside edge of the bearing area. Assume the crack angle, θ, is 35 degrees. Calculate the distance from the bottom of slab to center of tensile reinforcement. Determine the distance D_{crack} from the end of the slab to the point at which the diagonal crack will intersect the bottom longitudinal reinforcement. Find the development length, ℓ_d , from Table 9.9-2, Chapter 9.

The nominal tensile resistance, T_{nom} , of the longitudinal bars at the crack location is:

$$T_{nom} = A_s f_y\ [\ D_{crack} - (\text{end cover})] / \ell_d \ \leq\ A_s f_y$$

Then check that, $T_{fact} \leq T_{nom}$ is satisfied.

If the values for T_{fact} and T_{nom} are close, the procedure for determining the crack angle, θ, as outlined in **LRFD [5.8.3.4.2]** should be used.

18.4.11 Uplift Check

Check for uplift at the abutments for (HL-93) live loads **LRFD [C3.4.1, 5.5.4.3]**. Compare the factored dead load reaction to the factored live load reaction. The reactions shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. Place (HL-93) live loads in each design lane **LRFD [3.6.1.1.1]** and apply a multiple presence factor **LRFD [3.6.1.1.2]**.

18.4.12 Deflection Joints and Construction Joints

The designer should locate deflection joints in sidewalks and parapets on concrete slab structures according to the Standard *Vertical Face Parapet 'A'* in Chapter 30.

Refer to Standards *Continuous Haunched Slab* and *Continuous Flat Slab* in Chapter 18, for recommended construction joint guidelines.

18.4.13 Reinforcement Tables

Table 18.4-3 applies to: Rectangular Sections with Tension Reinforcement only

- Reinforcement Yield Strength (f_y) = 60,000 psi
- Concrete Compressive Strength (f'_c) = 4,000 psi

R_u	ρ	R_u	ρ	R_u	ρ	R_u	ρ	R_u	ρ
117.9	0.0020	335.6	0.0059	537.1	0.0098	722.6	0.0137	892.0	0.0176
123.7	0.0021	340.9	0.0060	542.1	0.0099	727.2	0.0138	896.1	0.0177
129.4	0.0022	346.3	0.0061	547.1	0.0100	731.7	0.0139	900.2	0.0178
135.2	0.0023	351.6	0.0062	552.0	0.0101	736.2	0.0140	904.4	0.0179
141.0	0.0024	357.0	0.0063	556.9	0.0102	740.7	0.0141	908.5	0.0180
146.7	0.0025	362.3	0.0064	561.8	0.0103	745.2	0.0142	912.5	0.0181
152.4	0.0026	367.6	0.0065	566.7	0.0104	749.7	0.0143	916.6	0.0182
158.1	0.0027	372.9	0.0066	571.6	0.0105	754.2	0.0144	920.7	0.0183
163.8	0.0028	378.2	0.0067	576.5	0.0106	758.7	0.0145	924.8	0.0184
169.5	0.0029	383.5	0.0068	581.4	0.0107	763.1	0.0146	928.8	0.0185
175.2	0.0030	388.8	0.0069	586.2	0.0108	767.6	0.0147	932.8	0.0186
180.9	0.0031	394.1	0.0070	591.1	0.0109	772.0	0.0148	936.9	0.0187
186.6	0.0032	399.3	0.0071	595.9	0.0110	776.5	0.0149	940.9	0.0188
192.2	0.0033	404.6	0.0072	600.8	0.0111	780.9	0.0150	944.9	0.0189
197.9	0.0034	409.8	0.0073	605.6	0.0112	785.3	0.0151	948.9	0.0190
203.5	0.0035	415.0	0.0074	610.4	0.0113	789.7	0.0152	952.9	0.0191
209.1	0.0036	420.2	0.0075	615.2	0.0114	794.1	0.0153	956.8	0.0192
214.8	0.0037	425.4	0.0076	620.0	0.0115	798.4	0.0154	960.8	0.0193
220.4	0.0038	430.6	0.0077	624.8	0.0116	802.8	0.0155	964.7	0.0194
225.9	0.0039	435.8	0.0078	629.5	0.0117	807.2	0.0156	968.7	0.0195
231.5	0.0040	441.0	0.0079	634.3	0.0118	811.5	0.0157	972.6	0.0196
237.1	0.0041	446.1	0.0080	639.0	0.0119	815.8	0.0158	976.5	0.0197
242.7	0.0042	451.3	0.0081	643.8	0.0120	820.1	0.0159	980.4	0.0198
248.2	0.0043	456.4	0.0082	648.5	0.0121	824.5	0.0160	984.3	0.0199
253.7	0.0044	461.5	0.0083	653.2	0.0122	828.8	0.0161	988.2	0.0200
259.3	0.0045	466.6	0.0084	657.9	0.0123	833.1	0.0162	992.1	0.0201
264.8	0.0046	471.7	0.0085	662.6	0.0124	837.3	0.0163	996.0	0.0202
270.3	0.0047	476.8	0.0086	667.3	0.0125	841.6	0.0164	999.8	0.0203
275.8	0.0048	481.9	0.0087	671.9	0.0126	845.9	0.0165	1003.7	0.0204
281.3	0.0049	487.0	0.0088	676.6	0.0127	850.1	0.0166	1007.5	0.0205
286.8	0.0050	492.1	0.0089	681.3	0.0128	854.3	0.0167	1011.3	0.0206
292.2	0.0051	497.1	0.0090	685.9	0.0129	858.6	0.0168	1015.1	0.0207
297.7	0.0052	502.2	0.0091	690.5	0.0130	862.8	0.0169	1018.9	0.0208
303.1	0.0053	507.2	0.0092	695.1	0.0131	867.0	0.0170	1022.7	0.0209
308.6	0.0054	512.2	0.0093	699.7	0.0132	871.2	0.0171	1026.5	0.0210
314.0	0.0055	517.2	0.0094	704.3	0.0133	875.4	0.0172	1030.3	0.0211
319.4	0.0056	522.2	0.0095	708.9	0.0134	879.5	0.0173	1034.0	0.0212
324.8	0.0057	527.2	0.0096	713.5	0.0135	883.7	0.0174	1037.8	0.0213
330.2	0.0058	532.2	0.0097	718.1	0.0136	887.9	0.0175	----	----

Table 18.4-3

R_u (psi) vs. ρ

R_u = coefficient of resistance (psi) = $M_u / \phi\, b\, d_s^2$ ρ = reinforcement ratio = $A_s / b\, d_s$

E18-1 Continuous 3-Span Haunched Slab - LRFD

A continuous 3-span haunched slab structure is used for the design example. The same basic procedure is applicable to continuous flat slabs. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. Design using a slab width equal to one foot. ***(Example is current through LRFD Sixth Edition - 2012)***

E18-1.1 Structure Preliminary Data

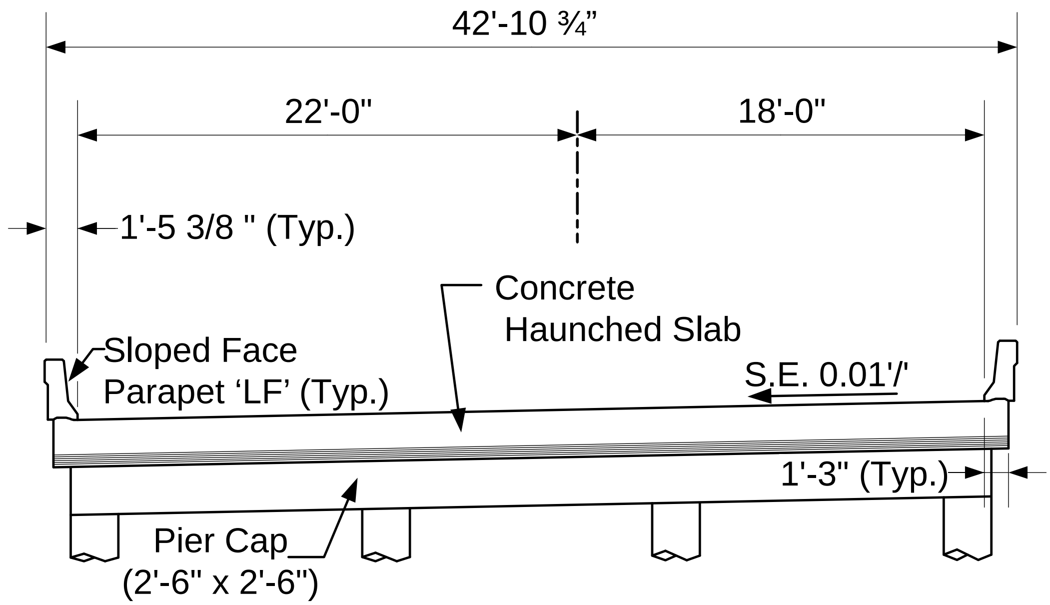


Figure E18.1

Section Perpendicular to Centerline

Live Load: HL-93
(A1) Fixed Abutments at both ends
Parapets placed after falsework is released

Geometry:

$L_1 := 38.0$	ft	Span 1
$L_2 := 51.0$	ft	Span 2
$L_3 := 38.0$	ft	Span 3
$slab_{width} := 42.5$	ft	out to out width of slab
$skew := 6$	deg	skew angle (RHF)
$w_{roadway} := 40.0$	ft	clear roadway width

Material Properties: (See 18.2.2)

$f'_c := 4$	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)

E18-1.2 LRFD Requirements

For concrete slab design, the slab dimensions and the size and spacing of reinforcement shall be selected to satisfy the equation below for all appropriate Limit States: (See 18.3.2.1)

$Q = \Sigma\eta_i \cdot \gamma_i \cdot Q_i \le \phi \cdot R_n = R_r$ (Limit States Equation)

The value of the load modifier is:

$\eta_i := 1.0$ for all Limit States (See 18.3.2.2)

The force effect, Q_i , is the moment, shear, stress range or deformation caused by applied loads.

The applied loads from **LRFD [3.3.2]** are:

DC = dead load of slab (DC_{slab}), ½ inch wearing surface ($DC_{1/2"WS}$) and parapet dead load (DC_{para}) - (See E18-1.3)

DW = dead load of future wearing surface (DW_{FWS}) - (See E18-1.3)

LL+IM = vehicular live load (LL) with dynamic load allowance (IM) - (See E18-1.4)

The Influence of ADTT and skew on force effects, Q_i , are ignored for slab bridges (See 18.3.2.2).

The values for the load factors, γ_i , (for each applied load) and the resistance factors, ϕ , are found in Table E18.1.

The total factored force effect, Q , must not exceed the factored resistance, R_r . The nominal resistance, R_n , is the resistance of a component to the force effects.

$s \le 12.9$ in

Therefore, spacing prov'd. = 7 in < 12.9 in O.K.

Use: #9 at 7" c-c spacing in span 1 (Max. positive reinforcement).

E18-1.7.1.4 Minimum Reinforcement Check

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: **LRFD[5.7.3.3.2]**

M_{cr} (or) $1.33M_u$

$$M_{cr} = \gamma_3(\gamma_1 \cdot f_r) S \qquad \text{where: } S = \frac{I_g}{c} \qquad \text{therefore, } M_{cr} = 1.1(f_r)\frac{I_g}{c}$$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$f_r = 0.37\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$$f_r = 0.37\sqrt{4} \qquad \boxed{f_r = 0.74} \text{ ksi}$$

$$I_g := \frac{1}{12} \cdot b \cdot d_{slab}^{\;3} \qquad \boxed{I_g = 4913} \text{ in}^4 \qquad c := \frac{d_{slab}}{2} \qquad \boxed{c = 8.5} \text{ in}$$

$$M_{cr} = \frac{1.1 f_r \cdot (I_g)}{c} = \frac{1.1 \cdot 0.74 \cdot (4913)}{8.5(12)} \qquad \boxed{M_{cr} = 39.21} \text{ kip-ft}$$

$1.33 \cdot M_u = 138.75$ kip-ft , where M_u was calculated for Strength Design in E18-1.7.1.1 and (M_u = 104.3 kip-ft)

M_{cr} controls because it is less than 1.33 M_u

As shown in E18-1.7.1.1, the reinforcement yields, therefore:

$$M_r = 0.90 \cdot A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) \qquad \boxed{M_r = 105} \text{ kip-ft}$$

Therefore, M_{cr} = 39.21 kip-ft < M_r = 105 kip-ft O.K.

E18-1.7.2 Negative Moment Reinforcement at Piers

Examine at C/L of Pier

E18-1.7.2.1 Design for Strength

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \leq 0.90\ A_s\ f_s\ (d_s - a/2)$$

The negative live load moment shall be the largest caused by live loads (LL#1, LL#2 or LL#3). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

From Table E18.4, the largest live load moment is from (LL#2), therefore at (C/L of Pier):

M_{DC} = -59.2 kip-ft M_{DW} = -4.9 kip-ft M_{LL+IM} = -15.5 + (-39.9) = -55.4 kip-ft

$M_u := 1.25\cdot(-59.2) + 1.50\cdot(-4.9) + 1.75\cdot(-55.4)$ $\boxed{M_u = -178.3}$ kip-ft

$b := 12$ inches (for a one foot design width) and $\boxed{d_s = 25.4}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 307.1}$ psi $\boxed{\rho = 0.0054}$ $\boxed{A_s = 1.65}$ $\frac{in^2}{ft}$

Try: #8 at 5 1/2" c-c spacing (A_s = 1.71 in²/ft) from Table 18.4-4 in 18.4.13

Assume $f_s = f_y$, then the depth of the compressive stress block is: $\boxed{a = 2.51}$ in

Then, $\boxed{c = 2.96}$ in and $\frac{c}{d_s} = 0.12 < 0.6$ therefore, the reinforcement will yield.

The factored resistance is: $\boxed{M_r = 186.6}$ kip-ft

Therefore, M_u = 178.3 kip-ft < M_r = 186.6 kip-ft O.K.

E18-1.7.2.2 Check for Fatigue

Following the procedure in E18-1.7.1.2, using Fatigue I Limit State:

$$1.5\cdot(f_{range}) \leq 24 - 0.33\cdot f_{min}$$

From Table E18.4, the moments at (C/L Pier) are:

M_{DC} = -59.2 kip-ft M_{DW} = -4.9 kip-ft

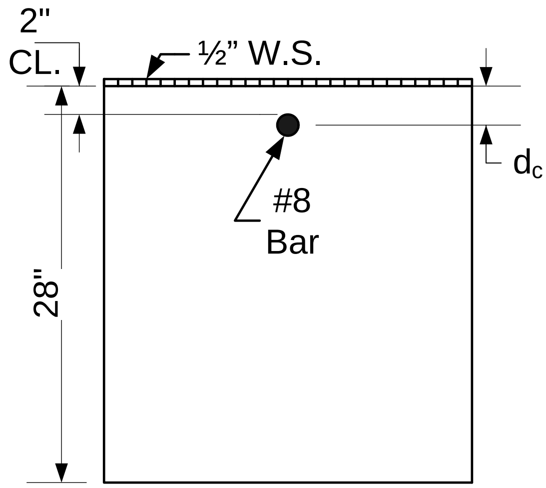


Figure E18.4

Cross Section - (at C/L of Pier)

E18-1.7.2.4 Minimum Reinforcement Check

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: **LRFD [5.7.3.3.2]**

M_{cr} (or) $1.33M_u$

from E18-1.7.1.4, $M_{cr} = 1.1(f_r)\dfrac{I_g}{c}$

Where:

$f_r = 0.37\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.37\sqrt{4}$ $\boxed{f_r = 0.74}$ ksi

$I_g := \dfrac{1}{12}\cdot b\cdot D_{haunch}^{\,3}$ $\boxed{I_g = 21952}$ in^4 $c := \dfrac{D_{haunch}}{2}$ $\boxed{c = 14}$ in

$M_{cr} = \dfrac{1.1f_r\cdot(I_g)}{c} = \dfrac{1.1\cdot 0.74\cdot(21952)}{14(12)}$ $\boxed{M_{cr} = 106.36}$ kip-ft

$1.33\cdot M_u = 237.1$ kip-ft , where M_u was calculated for Strength Design in E18-1.7.2.1 and (M_u = 178.3 kip-ft)

M_{cr} controls because it is less than 1.33 M_u

By examining E18-1.7.2.1, the reinforcement yields, therefore:

$M_r = 0.90 \cdot A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$ $\boxed{M_r = 204.1}$ kip-ft

Therefore, $M_{cr} = 106.36$ kip-ft < $M_r = 204.1$ kip-ft <u>O.K</u>.

E18-1.7.3 Positive Moment Reinforcement for Span 2

<u>Examine the 0.5 point of span 2</u>

E18-1.7.3.1 Design for Strength

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \leq 0.90\ A_s\ f_s\ (d_s - a/2)$

The positive live load moment shall be the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

From Table E18.4, the largest live load moment is from (LL#1), therefore at (0.5 pt.) of span 2:

$M_{DC} = 19.6$ kip-ft $M_{DW} = 1.6$ kip-ft $M_{LL+IM} = 8.2 + 37.4 = 45.6$ kip-ft

$M_u := 1.25 \cdot (19.6) + 1.50 \cdot (1.6) + 1.75 \cdot (45.6)$ $\boxed{M_u = 106.7}$ kip-ft

$b := 12$ inches (for a one foot design width) and $\boxed{d_s = 14.9}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 534}$ psi $\boxed{\rho = 0.0097}$ $\boxed{A_s = 1.73}$ $\frac{in^2}{ft}$

Try: #9 at 6" c-c spacing ($A_s = 2.00$ in²/ft) from Table 18.4-4 in 18.4.13

Assume $f_s = f_y$, then the depth of the compressive stress block is: $\boxed{a = 2.94}$ in

Then, $\boxed{c = 3.46}$ in and $\frac{c}{d_s} = 0.23 < 0.6$ therefore, the reinforcement will yield.

The factored resistance is: $\boxed{M_r = 120.9}$ kip-ft

Therefore, $M_u = 106.7$ kip-ft < $M_r = 120.9$ kip-ft <u>O.K.</u>

$$V_r = \phi_v \cdot V_n = \phi_v \cdot \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \sqrt{f'_c} \cdot (b_o) \cdot (d_v) \leq \phi_v \cdot (0.126) \cdot \sqrt{f'_c} \cdot (b_o) \cdot (d_v)$$

Where:

β_c = ratio of long side to short side of the rectangle through which reaction force is transmitted
$\approx$ 41.71 ft. / 2.5 ft. = 16.7

d_v = effective shear depth = dist. between resultant tensile & compressive forces
$\approx$ 24 in.

b_o = perimeter of the critical section
$\approx$ 1109 in.

Therefore, $V_r := \phi_v \cdot \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \sqrt{f'_c} \cdot (b_o) \cdot d_v$ $\boxed{V_r = 3380}$ kips

but $\leq \phi_v \cdot 0.126 \cdot \sqrt{f'_c} \cdot (b_o) \cdot d_v = 6036$ kips

Therefore, V_u = 1336 kips < V_r = 3380 kips <u>O.K.</u>

Note: Shear check and shear reinforcement design for the pier cap is not shown in this example. Also crack control criteria, minimum reinforcement checks, and shrinkage and temperature reinforcement checks are not shown for the pier cap.

E18-1.16.8 Minimum Reinforcement Check for Transverse Slab Member

Check the negative moment reinforcement (at interior column) for minimum reinforcement criteria.

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: **LRFD [5.7.3.3.2]**

M_{cr} (or) $1.33M_u$

from E18-1.7.1.4, $M_{cr} = 1.1(f_r)\frac{I_g}{c}$

Where:

$f_r = 0.37\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.37\sqrt{4}$ $\boxed{f_r = 0.74}$ ksi

h = pier cap depth + D_{haunch} (section depth) $\boxed{h = 58}$ in

b_{cap} = pier cap width $\boxed{b_{cap} = 30}$ in

$I_g := \frac{1}{12} \cdot b_{cap} \cdot h^3$ (gross moment of inertia) $\boxed{I_g = 487780}$ in^4

$c := \frac{h}{2}$ (section depth/2) $\boxed{c = 29}$ in

$$M_{cr} = \frac{1.1 f_r \cdot (I_g)}{c} = \frac{1.1 \cdot 0.74 \cdot (487780)}{29(12)}$$ $\boxed{M_{cr} = 1141}$ kip-ft

$1.33 \cdot M_u = 605.4$ kip-ft , where M_u was calculated for Strength Design in E18-1.16.6.1 and (M_u = 455.2 kip-ft)

1.33 M_u controls because it is less than M_{cr}

Recalculating requirements for (New moment = $1.33 \cdot M_u$ = 605.4 kip-ft)

$\boxed{b_{neg} = 30}$ in (See E18-1.16.2)

$\boxed{d_{neg} = 54.62}$ in (See E18-1.16.2)

Calculate R_u, coefficient of resistance:

$$R_u = \frac{M_u}{\phi_f \cdot (b_{neg}) \cdot d_{neg}^2} \qquad R_u := \frac{605.4 \cdot (12) \cdot 1000}{0.9(30) \cdot 54.62^2}$$ $\boxed{R_u = 90.2}$ psi

Solve for ρ, reinforcement ratio, using Table 18.4-3 (R_u vs ρ) in 18.4.13;

$\rho := 0.00152$

$A_s = \rho \cdot (b_{neg}) \cdot d_{neg}$ $\quad A_s := 0.00152 \cdot (30) 54.62$ $\quad \boxed{A_s = 2.49}$ in^2

Place this reinforcement in a width, centered over the pier, equal to 1/2 the center to center column spacing or 8 feet, whichever is smaller. Therefore, width equals 6.5 feet.

Therefore, 2.49 in^2/6.5 ft. = 0.38 in^2/ft. Try <u>#5 at 9" c-c spacing</u> for a 6.5 ft. transverse width over the pier. This will provide (A_s = 2.79 in^2) in a 6.5 ft. width.

Calculate the depth of the compressive stress block

Assume $f_s = f_y$ (See 18.3.3.2.1)

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b_{neg}} \qquad a := \frac{2.79 \cdot (60)}{0.85 \cdot (4.0) \cdot 30}$$ $\boxed{a = 1.64}$ in

Table of Contents

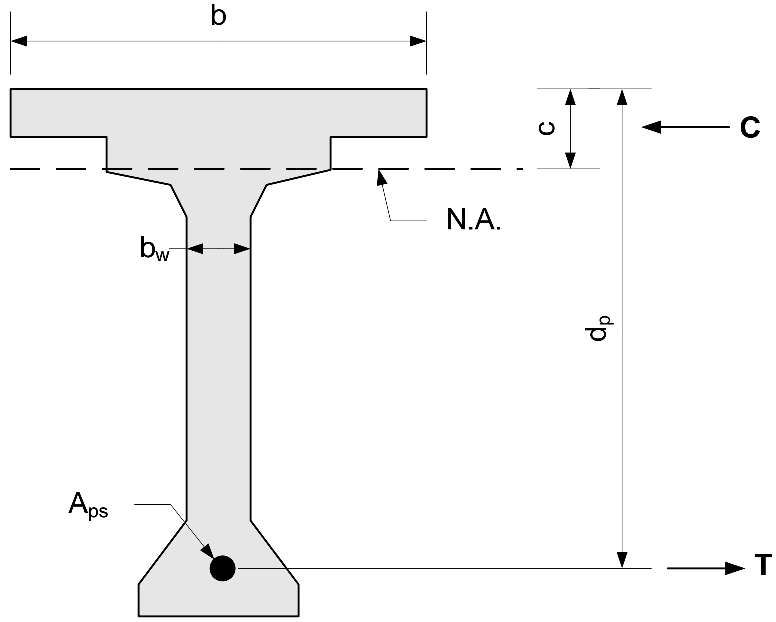


Figure 19.3-3
Depth to Neutral Axis, c

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

$$c = \frac{A_{ps} f_{pu} - 0.85\beta_1 f'_c (b - b_w) h_f}{0.85 f'_c \beta_1 b_w + k A_{ps} \frac{f_{pu}}{d_p}}$$

Where:

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

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

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

Where:

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

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

$$M_r = \phi A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + 0.85\phi f'_c (b - b_w) h_f \left(\frac{a}{2} - \frac{h_f}{2}\right)$$

Where:

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

The engineer must then verify that M_r is greater than or equal to M_u.

WisDOT exception to AASHTO:

WisDOT standard prestressed concrete girders and strand patterns are tension-controlled. The ε_t check, as specified in **LRFD [5.7.2.1]**, is not required when the standard girders and strand patterns are used, and $\phi = 1$.

19.3.3.13.2 Minimum Reinforcement

Per **LRFD [5.7.3.3.2]**, the minimum amount of prestressed reinforcement provided shall be adequate to develop an M_r at least equal to the lesser of M_{cr}, or $1.33M_u$.

M_{cr} is the cracking moment, and is given by:

$$M_{cr} = \gamma_3 [S_c (\gamma_1 f_r + \gamma_2 f_{cpe}) - 12M_{dnc} [(S_c/S_{nc}) - 1]]$$

Where:

S_c	=	Section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in^3)
f_r	=	Modulus of rupture (ksi)
f_{cpe}	=	Compressive stress in concrete due to effective prestress forces only (after losses) at extreme fiber of section where tensile stress

is caused by externally applied loads (ksi)

M_{dnc} = Total unfactored dead load moment acting on the basic beam (k-ft)

S_{nc} = Section modulus for the extreme fiber of the basic beam where tensile stress is caused by externally applied loads (in^3)

γ_1 = 1.6 flexural cracking variability factor

γ_2 = 1.1 prestress variability factor

γ_3 = 1.0 for prestressed concrete structures

Per **LRFD [5.4.2.6]**, the modulus of rupture for normal weight concrete is given by:

$$f_r = 0.37\sqrt{f'_c}$$

19.3.3.14 Non-prestressed Reinforcement

Non-prestressed reinforcement consists of bar steel reinforcement used in the conventional manner. It is placed longitudinally along the top of the member to carry any tension which may develop after transfer of prestress. The designer should completely detail all rebar layouts including stirrups.

The amount of reinforcement is that which is sufficient to resist the total tension force in the concrete based on the assumption of an uncracked section.

For draped designs, the control is at the hold-down point of the girder. At the hold-down point, the initial prestress is acting together with the girder dead load stress. This is where tension due to prestress is still maximum and compression due to girder dead load is decreasing.

For non-draped designs, the control is at the end of the member where prestress tension exists but dead load stress does not.

Note that a minimum amount of reinforcement is specified in the Standards. This is intended to help prevent serious damage due to unforeseeable causes like improper handling or storing.

19.3.3.15 Horizontal Shear Reinforcement

The horizontal shear reinforcement resists the Strength I limit state horizontal shear that develops at the interface of the slab and girder in a composite section. The dead load used to calculate the horizontal shear should only consider the DC and DW dead loads that act on the composite section. See 17.2.4 for further information regarding the treatment of dead loads and load combinations.

$$V_u = 1.25DC + 1.50DW + 1.75(LL + IM)$$

$$V_{ni} \geq V_{ui} / \phi$$

Where:

V_u	=	Maximum strength limit state vertical shear (kips)
V_{ui}	=	Strength limit state horizontal shear at the girder/slab interface (kips)
V_{ni}	=	Nominal interface shear resistance (kips)
ϕ	=	0.90 per **LRFD [5.5.4.2.1]**

The shear stress at the interface between the slab and the girder is given by:

$$v_{ui} = \frac{V_u}{b_{vi} d_v}$$

Where:

v_{ui}	=	Factored shear stress at the slab/girder interface (ksi)
b_{vi}	=	Interface width to be considered in shear transfer (in)
d_v	=	Distance between the centroid of the girder tension steel and the mid-thickness of the slab (in)

The factored horizontal interface shear shall then be determined as:

$$V_{ui} = 12 v_{ui} b_{vi}$$

The nominal interface shear resistance shall be taken as:

$$V_{ni} = cA_{cv} + \mu\left[A_{vf} f_y + P_c\right]$$

Where:

A_{cv}	=	Concrete area considered to be engaged in interface shear transfer. This value shall be set equal to $12b_{vi}$ (ksi)
c	=	Cohesion factor specified in **LRFD [5.8.4.3]**. This value shall be taken as 0.28 ksi for WisDOT standard girders with a cast-in-place deck

μ	=	Friction factor specified in **LRFD [5.8.4.3]**. This value shall be taken as 1.0 for WisDOT standard girders with a cast-in-place deck (dim.)
A_{vf}	=	Area of interface shear reinforcement crossing the shear plan within the area A_{cv} (in^2)
f_y	=	Yield stress of shear interface reinforcement not to exceed 60 (ksi)
P_c	=	Permanent net compressive force normal to the shear plane (kips)

P_c shall include the weight of the deck, haunch, parapets, and future wearing surface. A conservative assumption that may be considered is to set $P_c = 0.0$.

The nominal interface shear resistance, V_{ni}, shall not exceed the lesser of:

$$V_{ni} \leq K_1 f'_c A_{cv} \quad \text{or} \quad V_{ni} \leq K_2 A_{cv}$$

Where:

K_1	=	Fraction of concrete strength available to resist interface shear as specified in **LRFD [5.8.4.3]**. This value shall be taken as 0.3 for WisDOT standard girders with a cast-in-place deck (dim.)
K_2	=	Limiting interface shear resistance as specified in **LRFD [5.8.4.3]**. This value shall be taken as 1.8 ksi for WisDOT standard girders with a cast-in-place deck

WisDOT policy item:

The stirrups that extend into the deck slab presented on the Standards are considered adequate to satisfy the minimum reinforcement requirements of **LRFD [5.8.4.4]**

19.3.3.16 Web Shear Reinforcement

Web shear reinforcement consists of placing conventional reinforcement perpendicular to the axis of the I-girder.

WisDOT policy item:

Web shear reinforcement shall be designed by **LRFD [5.8.3.4.3]** (Simplified Procedure) using the Strength I limit state for WisDOT standard girders.

WisDOT prefers girders with spacing symmetrical about the midspan in order to simplify design and fabrication. The designer is encouraged to simplify the stirrup arrangement as much as possible. For vertical stirrups, the required area of web shear reinforcement is given by the following equation:

$$A_v \geq \frac{(V_n - V_c)s}{f_y d_v \cot\theta} \quad \text{(or } 0.0316\sqrt{f'_c}\frac{b_v s}{f_y} \text{ minimum)}$$

Where:

A_v	=	Area of transverse reinforcement within distance, s (in^2)
V_n	=	Nominal shear resistance (kips)
V_c	=	Nominal shear resistance provided by tensile stress in the concrete (kips)
s	=	Spacing of transverse reinforcement (in)
f_y	=	Specified minimum yield strength of transverse reinforcement (ksi)
d_v	=	Effective shear depth as determined in **LRFD [5.8.2.9]** (in)
b_v	=	Minimum web width within depth, d_v

$\cot\theta$ shall be taken as follows:

- When $V_{ci} < V_{cw}$, $\cot\theta = 1.0$
- When $V_{ci} > V_{cw}$, $\cot\theta = 1.0 + 3\left(\frac{f_{pc}}{\sqrt{f'_c}}\right) \leq 1.8$

$$V_u = 1.25DC + 1.5DW + 1.75(LL + IM)$$
$$V_n = V_u / \phi$$

Where:

V_u	=	Strength I Limit State shear force (kips)
ϕ	=	0.90 per **LRFD [5.5.4.2.1]**

See 17.2 for further information regarding load combinations.

Per **LRFD [5.8.3.4.3]**, determine V_c as the minimum of either V_{ci} or V_{cw} given by:

$$V_{cw} = (0.06\sqrt{f'_c} + 0.30 f_{pc}) b_v d_v + V_p$$

$$V_{ci} = 0.02\sqrt{f'_c}\, b_v d_v + V_d + \frac{V_i M_{cre}}{M_{max}} \geq 0.06\sqrt{f'_c}\, b_v d$$

Where:

f_{pc}	=	Compressive stress in concrete, after all prestress losses, at centroid of cross section resisting externally applied loads or at the web-flange junction when the centroid lies within the flange. (ksi) In a composite member, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the web-flange junction, due to both prestress and moments resisted by the member acting alone.
V_d	=	Shear force at section due to unfactored dead loads (kips)
V_i	=	Factored shear force at section due to externally applied loads occurring simultaneously with M_{max} (kips)
M_{cre}	=	Moment causing flexural cracking at the section due to externally applied loads (k-in)
M_{max}	=	Maximum factored moment at section due to externally applied loads (k-in)

$$V_i = V_u - V_d$$

$$M_{cre} = S_c\left(f_r + f_{cpe} - \frac{12M_{dnc}}{S_{nc}}\right)$$

$$M_{max} = M_u - M_{dnc}$$

Where:

S_c	=	Section modulus for the extreme tensile fiber of the composite section where the stress is caused by externally applied loads (in^3)
S_{nc}	=	Section modulus for the extreme tensile fiber of the noncomposite section where the stress is caused by externally applied loads (in^3)
f_{cpe}	=	Compressive stress in concrete due to effective prestress forces only, after all prestress losses, at the extreme tensile fiber of the section where the stress is caused by externally applied loads (ksi)
M_{dnc}	=	Total unfactored dead load moment acting on the noncomposite section (k-ft)
f_r	=	Modulus of rupture of concrete. Shall be $= 0.24\sqrt{f'_c}$ (ksi)

For a composite section, V_{ci} corresponds to shear at locations of accompanying flexural stress. V_{cw} corresponds to shear at simple supports and points of contraflexure. The critical computation for V_{cw} is at the centroid for composite girders.

Set the vertical component of the draped strands, V_p, equal to 0.0 when calculating V_n, as per **LRFD [5.8.3.3]**. This vertical component helps to reduce the shear on the concrete section. The actual value of V_p should be used when calculating V_{cw}. However, the designer may make the conservative assumption to neglect V_p for all shear resistance calculations.

WisDOT policy item:

Based on past performance, the upper limit for web reinforcement spacing, s_{max}, per **LRFD [5.8.2.7]** will be reduced to 18 inches.

When determining shear reinforcement, spacing requirements as determined by analysis at 1/10th points, for example, should be carried-out to the next 1/10th point. As an illustration, spacing requirements for the 1/10th point should be carried out to very close to the 2/10th point, as the engineer, without a more refined analysis, does not know what the spacing requirements would be at the 0.19 point. For the relatively small price of stirrups, don't shortchange the shear capacity of the prestressed girder.

The web reinforcement spacing shall not exceed the maximum permitted spacing determined as:

- If $\upsilon_u < 0.125f'_c$, then $s_{max} = 0.8d_v \leq 18"$
- If $\upsilon_u \geq 0.125f'_c$, then $s_{max} = 0.4d_v \leq 12"$

Where:

$$\upsilon_u = \frac{V_u - \phi V_p}{\phi b_v d_v}$$ per **LRFD [5.8.2.9]**.

The nominal shear resistance, $V_c + V_s$, is limited by the following:

$$V_c + \frac{A_v f_y d_v \cot\theta}{s} \leq 0.25f'_c b_v d_v$$

Reinforcement in the form of vertical stirrups is required at the extreme ends of the girder. The stirrups are designed to resist 4% of the total prestressing force at transfer at a unit stress of 20 ksi and are placed within h/4 of the girder end, where h is the total girder depth. For a distance of 1.5d from the ends of the beams, reinforcement shall be placed to confine the prestressing steel in the bottom flange. The reinforcement shall be shaped to enclose the strands, shall be a #3 bar or greater and shall be spaced at less than or equal to 6". Note that the reinforcement shown on the Standard Details sheets satisfies these requirements.

Welded wire fabric may be used for the vertical reinforcement. It must be deformed wire with a minimum size of D16.

Per **LRFD [5.8.3.5]**, at the inside edge of the bearing area to the section of critical shear, the longitudinal reinforcement on the flexural tension side of the member shall satisfy:

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{V_u}{\phi} - 0.5 V_s \right) \cot\theta$$

In the above equation, $\cot\theta$ is as defined in the V_c discussion above, and V_s is the shear reinforcement resistance at the section considered. Any lack of full reinforcement development shall be accounted for. Note that the reinforcement shown on the Standard Detail sheets satisfies these requirements.

19.3.3.17 Continuity Reinforcement

The design of non-prestressed reinforcement for negative moment at the support is based on the Strength I limit state requirements of **LRFD [5.7.3]**:

$$M_u = 1.25DC + 1.50DW + 1.75(LL + IM)$$

LRFD [5.5.4.2] allows a ϕ factor equal to 0.9 for tension-controlled reinforced concrete sections such as the bridge deck.

The continuity reinforcement consists of mild steel reinforcement in the deck in the negative moment region over the pier. Consider both the non-composite and the superimposed dead loads and live loads for the Strength I design of the continuity reinforcement in the deck.

Moment resistance is developed in the same manner as shown in 19.3.3.13.1 for positive moments, except that the bottom girder flange is in compression and the deck is in tension. The moment resistance is formed by the couple resulting from the compression force in the bottom flange and the tension force from the longitudinal deck steel. Consider A_s to consist of the longitudinal deck steel present in the deck slab effective flange width as determined in 19.3.3.8. The distance, d_p, is taken from the bottom of the girder flange to the center of the longitudinal deck steel.

WisDOT exception to AASHTO:

Composite sections formed by WisDOT standard prestressed concrete girders shall be considered to be tension-controlled for the design of the continuity reinforcement. The ε_t check, as specified in **LRFD [5.7.2.1]**, is not required, and $\phi = 0.9$.

WisDOT policy item:

New bridge designs shall consider only the top mat of longitudinal deck steel when computing the continuity reinforcement capacity.

WisDOT policy item:

The continuity reinforcement shall be based on the greater of either the interior girder design or exterior girder and detailed as typical reinforcement for the entire width of the bridge deck. However, do not design the continuity steel based on the exterior girder design beneath a raised sidewalk. The continuity steel beneath a raised sidewalk should not be used for rating.

Based on the location of the neutral axis, the bottom flange compressive force may behave as either a rectangle or a T-section. On WisDOT standard prestressed girders, if the depth of the compression block, a, falls within the varying width of the bottom flange, the compression block acts as an idealized T-section. In this case, the width, b, shall be taken as the bottom flange width, and the width, b_w, shall be taken as the bottom flange width at the depth "a". During T-section behavior, the depth, h_f, shall be taken as the depth of the bottom flange of full width, b. See Figure 19.3-4 for details. Ensure that the deck steel is adequate to satisfy $M_r \geq M_u$.

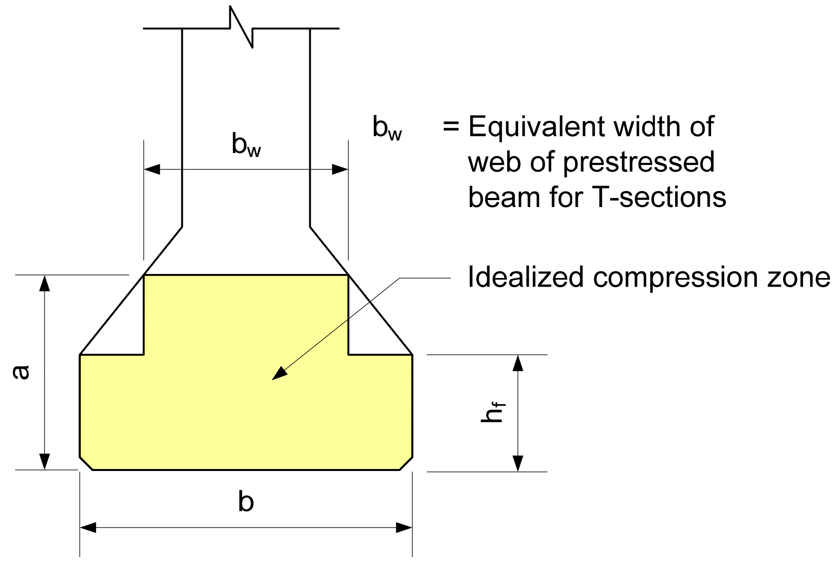


Figure 19.3-4
T-Section Compression Flange Behavior

The continuity reinforcement should also be checked to ensure that it meets the crack control provisions of **LRFD [5.7.3.4]**. This check shall be performed assuming severe exposure conditions. Only the superimposed loads shall be considered for the Service and Fatigue requirements.

The concrete between the abutting girder ends is usually of a much lesser strength than that of the girders. However, tests[1] have shown that, due to lateral confinement of the diaphragm concrete, the girder itself fails in ultimate negative compression rather than failure in the material between its ends. Therefore the ultimate compressive stress, f'_c, of the girder concrete is used in place of that of the diaphragm concrete.

This assumption has only a slight effect on the computed amount of reinforcement, but it has a significant effect on keeping the compression force within the bottom flange.

The continuity reinforcement shall conform to the Fatigue provisions of **LRFD [5.5.3]**.

The transverse spacing of the continuity reinforcement is usually taken as the whole or fractional spacing of the D bars as given in 17.5.3.2. Grade 60 bar steel is used for continuity reinforcement. Required development lengths for deformed bars are given in Chapter 9 – Materials.

WisDOT exception to AASHTO:

The continuity reinforcement is not required to be anchored in regions of the slab that are in compression at the strength limit state as stated in **LRFD [5.14.1.4.8]**. The following locations shall be used as the cut off points for the continuity reinforcement:

1. When ½ the bars satisfy the Strength I moment envelope (considering both the non-composite and composite loads) as well as the Service and Fatigue moment envelopes (considering only the composite moments), terminate ½ of the bars. Extend these bars past this cutoff point a distance not less than the girder depth or 1/16 the clear span for embedment length requirements.

2. Terminate the remaining one-half of the bars an embedment length beyond the point of inflection. The inflection point shall be located by placing a 1 klf load on the composite structure. This cut-off point shall be at least 1/20 of the span length or 4' from point 1, whichever is greater.

Certain secondary features result when spans are made continuous. That is, positive moments develop over piers due to creep[5], shrinkage and the effects of live load and dynamic load allowance in remote spans. The latter only exists for bridges with three or more spans.

These positive moments are somewhat counteracted by negative moments resulting from differential shrinkage[4] between the cast-in-place deck and precast girders along with negative moments due to superimposed dead loads. However, recent field observations cited in **LRFD [C5.14.1.4.2]** suggest that these moments are less than predicted by analysis. Therefore, negative moments caused by differential shrinkage should be ignored in design.

WisDOT exception to AASHTO:

WisDOT requires the use of a negative moment connection only. The details for a positive moment connection per **LRFD [5.14.1.4]** are not compatible with the Standard Details and should not be provided.

19.3.3.18 Camber and Deflection

The prestress camber and dead load deflection are used to establish the vertical position of the deck forms with respect to the girder. The theory presented in the following sections apply to a narrow set of circumstances. The designer is responsible for ensuring that the theoretical camber accounts for the loads applied to the girder. For example, if the diaphragms are configured so there is one at each of the third points instead of one at midspan, the term in the equation for $\Delta_{nc(DL)}$ related to the diaphragms in 19.3.3.18.2 would need to be modified to account for two point loads applied at the third points instead of one point load applied at midspan.

Deflection effects due to individual loads may be calculated separately and superimposed, as shown in this section. The *PCI Design Handbook* provides design aids to assist the designer in the evaluation of camber and deflection, including cambers for prestress forces and loads, and beam design equations and diagrams.

Figure 19.3-5 illustrates a typical girder with a draped strand profile.

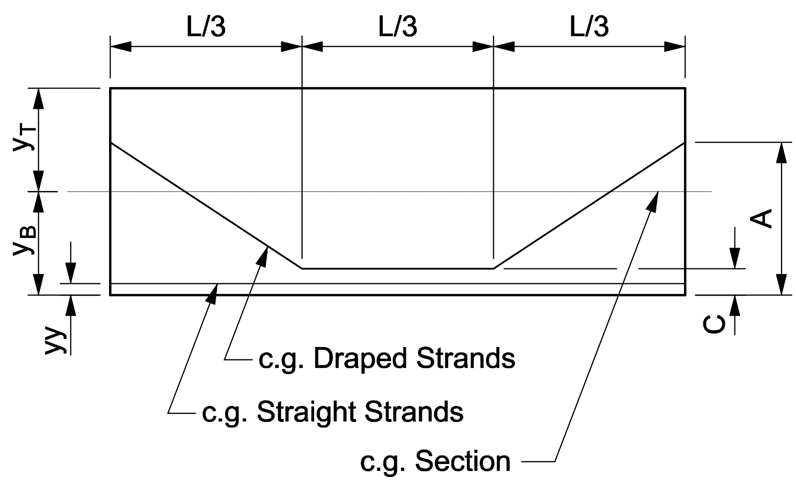


Figure 19.3-5
Typical Draped Strand Profile

19.3.3.18.1 Prestress Camber

The prestressing strands produce moments in the girder as a result of their eccentricity and draped pattern. These moments induce a camber in the girder. The values of the camber are calculated as follows:

Eccentric straight strands induce a constant moment of:

$$M_1 = \frac{1}{12}\left(P_i^s\left(y_B - yy\right)\right)$$

Where:

M_1 = Moment due to initial prestress force in the straight strands minus the elastic shortening loss (k-ft)

P_i^s	=	Initial prestress force in the straight strands minus the elastic shortening loss (kips)
y_B	=	Distance from center of gravity of beam to bottom of beam (in)
yy	=	Distance from center of gravity of straight strands to bottom of beam (in)

This moment produces an upward deflection at midspan which is given by:

$$\Delta_s = \frac{M_1 L^2}{8 E_i I_b} \quad \text{(with all units in inches and kips)}$$

For moments expressed in kip-feet and span lengths expressed in feet, this equation becomes the following:

$$\Delta_s = \frac{M_1 L^2}{8 E_i I_b}\left(\frac{12}{1}\right)\left(\frac{12^2}{1}\right) = \frac{M_1 L^2}{8 E_i I_b}\left(\frac{1728}{1}\right)$$

$$\Delta_s = \frac{216 M_1 L^2}{E_i I_b} \quad \text{(with units as shown below)}$$

Where:

Δ_s	=	Deflection due to force in the straight strands minus elastic shortening loss (in)
L	=	Span length between centerlines of bearing (ft)
E_i	=	Modulus of elasticity at the time of release (see 19.3.3.8) (ksi)
I_b	=	Moment of inertia of basic beam (in^4)

The draped strands induce the following moments at the ends and within the span:

$M_2 = \frac{1}{12}\left(P_i^D\left(A - C\right)\right)$, which produces upward deflection, and

$M_3 = \frac{1}{12}\left(P_i^D\left(A - y_B\right)\right)$, which produces downward deflection when A is greater than y_B

Where:

M_2, M_3	=	Components of moment due to initial prestress force in the draped strands minus the elastic shortening loss (k-ft)
P_i^D	=	Initial prestress force in the draped strands minus the elastic shortening loss (kips)

A = Distance from bottom of beam to center of gravity of draped strands at centerline of bearing (in)

C = Distance from bottom of beam to center of gravity of draped strands between hold-down points (in)

These moments produce a net upward deflection at midspan, which is given by:

$$\Delta_D = \frac{216L^2}{E_i I_b}\left(\frac{23}{27}M_2 - M_3\right)$$

Where:

Δ_D = Deflection due to force in the draped strands minus elastic shortening loss (in)

The combined upward deflection due to prestress is:

$$\Delta_{PS} = \Delta_s + \Delta_D = \frac{216L^2}{E_i I_b}\left(M_1 + \frac{23}{27}M_2 - M_3\right)$$

Where:

Δ_{PS} = Deflection due to straight and draped strands (in)

The downward deflection due to beam self-weight at release is:

$$\Delta_{o(DL)} = \frac{5W_b L^4}{384E_i I_b} \quad \text{(with all units in inches and kips)}$$

Using unit weights in kip per foot, span lengths in feet, E in ksi and I_b in inches^4, this equation becomes the following:

$$\Delta_s = \frac{5W_b L^4}{384E_i I_b}\left(\frac{1}{12}\right)\left(\frac{12^4}{1}\right) = \frac{5W_b L^4}{384E_i I_b}\left(\frac{20736}{12}\right)$$

$$\Delta_{o(DL)} = \frac{22.5W_b L^4}{E_i I_b} \quad \text{(with units as shown below)}$$

Where:

$\Delta_{o(DL)}$ = Deflection due to beam self-weight at release (in)

W_b = Beam weight per unit length (k/ft)

Therefore, the anticipated prestress camber at release is given by:

$$\Delta_i = \Delta_{PS} - \Delta_{o(DL)}$$

Where:

Δ_i = Prestress camber at release (in)

Camber, however, continues to grow after the initial strand release. For determining substructure beam seats, average concrete haunch values (used for both DL and quantity calculations) and the required projection of the vertical reinforcement from the tops of the prestressed girders, **a camber multiplier of 1.4 shall be used**. This value is multiplied by the theoretical camber at release value.

19.3.3.18.2 Dead Load Deflection

The downward deflection due to the dead load of the deck and midspan diaphragm is:

$$\Delta_{nc(DL)} = \frac{5W_{deck}L^4}{384EI_b} + \frac{P_{dia}L^3}{48EI_b} \quad \text{(with all units in inches and kips)}$$

Using span lengths in units of feet, unit weights in kips per foot, E in ksi, and I_b in inches^4, this equation becomes the following:

$$\Delta_s = \frac{5W_{deck}L^4}{384EI_b}\left(\frac{1}{12}\right)\left(\frac{12^4}{1}\right) + \frac{P_{dia}L^3}{48EI_b}\left(\frac{12^3}{1}\right) = \frac{5W_{deck}L^4}{384EI_b}\left(\frac{20736}{12}\right) + \frac{P_{dia}L^3}{48EI_b}\left(\frac{1728}{1}\right)$$

$$\Delta_{o(DL)} = \frac{22.5W_bL^4}{EI_b} + \frac{36P_{dia}L^3}{EI_b} \quad \text{(with units as shown below)}$$

Where:

$\Delta_{nc(DL)}$ = Deflection due to non-composite dead load (deck and midspan diaphragm) (in)

W_{deck} = Deck weight per unit length (k/ft)

P_{dia} = Midspan diaphragm weight (kips)

E = Girder modulus of elasticity at final condition (see 19.3.3.8) (ksi)

A similar calculation is done for parapet and sidewalk loads on the composite section. Provisions for deflections due to future wearing surface shall not be included.

For girder structures with raised sidewalks, loads shall be distributed as specified in Chapter 17, and separate deflection calculations shall be performed for the interior and exterior girders.

19.3.3.18.3 Residual Camber

Residual camber is the camber that remains after the prestress camber has been reduced by the composite and non-composite dead load deflection. Residual camber is computed as follows:

$$RC = \Delta_i - \Delta_{nc(DL)} - \Delta_{c(DL)}$$

19.3.4 Deck Forming

Deck forming requires computing the relationship between the top of girder and bottom of deck necessary to achieve the desired vertical roadway alignment. Current practice for design is to use a minimum haunch of 2" at the edge of the girder flange. This haunch value is also used for calculating composite section properties. This will facilitate current deck forming practices which use 1/2" removable hangers and 3/4" plywood, and it will allow for variations in prestress camber. Also, future deck removal will be less likely to damage the top girder flanges. An average haunch height of 3 inches minimum can be used for determining haunch weight for preliminary design. It should be noted that the actual haunch values should be compared with the estimated values during final design. If there are significant differences in these values, the design should be revised. The actual average haunch height should be used to calculate the concrete quantity reported on the plans as well as the value reported on the prestressed girder details sheet. The actual haunch values at the girder ends shall be used for determining beam seat elevations.

For designs involving vertical curves, Figure 19.3-6 shows two different cases.

19.3.7 Construction Dimensional Tolerances

Refer to the *AASHTO LRFD Bridge Construction Specifications* for the required dimensional tolerances.

19.3.8 Prestressed Girder Sections

WisDOT BOS employs two prestress I girder section families. One I section family follows the AASHTO standard section, while the other I section family follows a wide flange bulb-tee, see Figure 19.3-7. These sections employ draped strand patterns with undraped alternates where feasible. Undraped strand patterns, when practical, should be specified on the designs. For these sections, the cost of draping far exceeds savings in strands. See the Standard Details for the I girder sections' draped and undraped strand patterns. Note, for the 28" prestressed I girder section the 16 and 18 strand patterns require bond breakers.

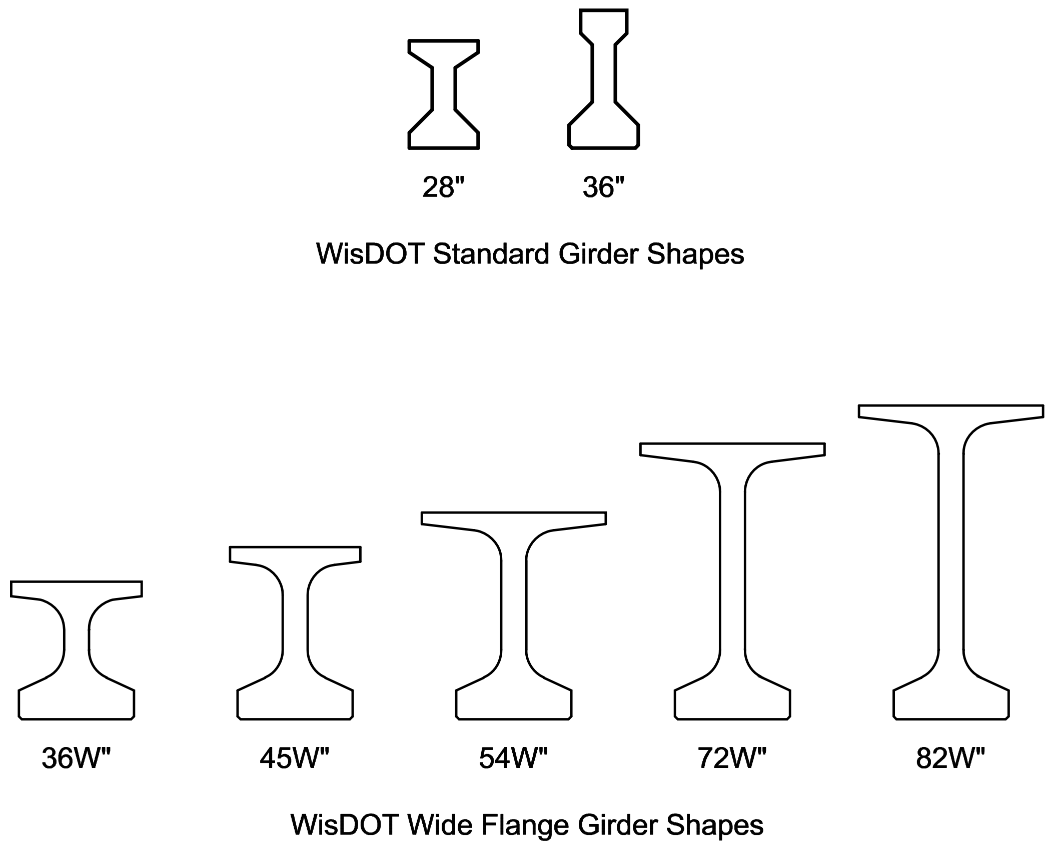


Figure 19.3-7
I Girder Family Details

Table 19.3-1 and Table 19.3-2 provide span lengths versus interior girder spacings for HL-93 live loading on single-span and multiple-span structures for prestressed I-girder sections. Girder spacings are based on using low relaxation strands at $0.75f_{pu}$, a concrete haunch of 2", slab thicknesses from Chapter 17 – Superstructures - General and a future wearing surface. For these tables, a line load of 0.300 klf is applied to the girder to account for superimposed dead loads.

Several girder shapes have been retired from standard use on new structures. These include the following sizes; 45-inch, 54-inch and 70-inch. These girder shapes are used for girder replacements, widening and for curved new structures where the wide flange sections are not practical. See Chapter 40 – Bridge Rehabilitation for additional information on these girder shapes.

Due to the wide flanges on the 54W, 72W and 82W and the variability of residual camber, haunch heights frequently exceed 2”. An average haunch of 2 ½” was used for these girders in the following tables. **The haunch values and parapet weights currently used in all the tables are somewhat unconservative -- do not push the span limits/girder spacing during preliminary design.** See Table 19.3-2 for guidance regarding use of excessively long prestressed girders.

For interior prestressed concrete I-girders, 0.5” or 0.6” dia. strands (in accordance with the Standard Details).

f'_c girder = 8,000 psi

f'_c slab = 4,000 psi

Haunch height = 2” or 2 ½”

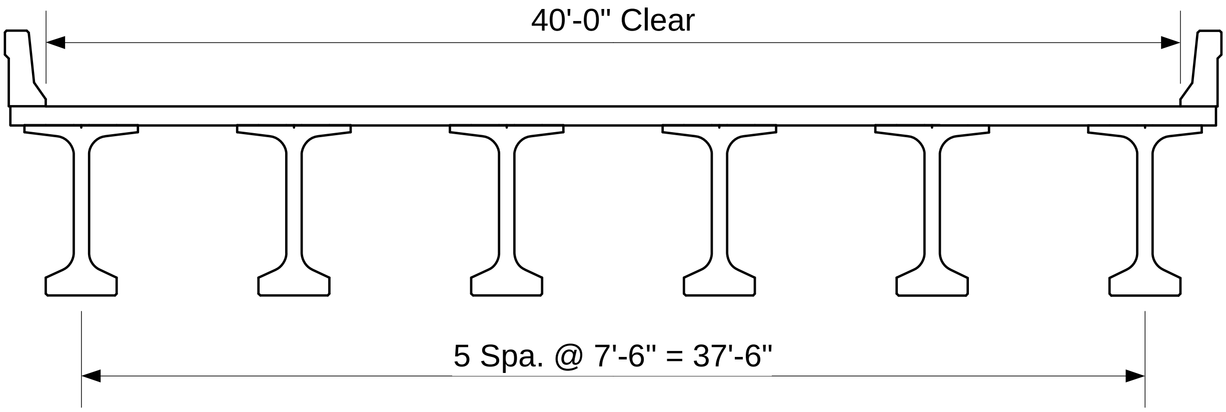
Required f'_c girder at initial prestress < 6,800 psi

Table of Contents

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 Sixth Edition - 2012)***

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
$A_s := 0.217$	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

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

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

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:

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

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 3.4.11]**:

	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			$\gamma f_{LL} := 1.50$

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:

$g_i = 0.636$

$M_{LL} = g_i \cdot 4828$ $\boxed{M_{LL} = 3073}$ kip-ft

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

$M_{LLfat} := g_{if} \cdot 2406$ $\boxed{M_{LLfat} = 871}$ 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

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

$$= 1.0 \cdot \left[1.25 \cdot \left(M_{DLnc} + M_{DLc}\right) + 1.50 \cdot M_{DWc} + 1.75 \cdot M_{LL}\right]$$ $\boxed{M_{str} = 12449}$ kip-ft

Service 1 (for compression checks)

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

$$= 1.0 \cdot \left[1.0 \cdot \left(M_{DLnc} + M_{DLc}\right) + 1.0 \cdot M_{DWc} + 1.0 \cdot M_{LL}\right]$$ $\boxed{M_{s1} = 8659}$ kip-ft

Service 3 (for tension checks)

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

$$= 1.0 \cdot \left[1.0 \cdot \left(M_{DLnc} + M_{DLc}\right) + 1.0 \cdot M_{DWc} + 0.8 \cdot M_{LL}\right]$$ $\boxed{M_{s3} = 8045}$ kip-ft

Service 1 and 3 non-composite DL alone

$M_{nc} := \eta \cdot \gamma s1_{DC} \cdot M_{DLnc}$ $\boxed{M_{nc} = 4887}$ kip-ft

Fatigue 1

$M_{fat} := \eta \cdot \gamma f_{LL} \cdot M_{LLfat}$ $\boxed{M_{fat} = 1307}$ 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

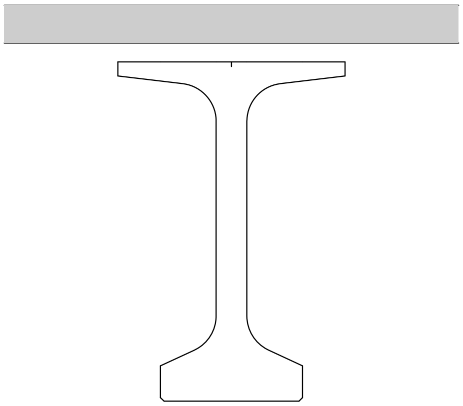
effective slab width; $\boxed{w_{eadj} = 58.46}$ in

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^4

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

$$S_{cgdb} := n \cdot \frac{I_{cg}}{y_{cgt} + hau}$$ $\boxed{S_{cgdb} = 73411}$ in^4

E19-1.9 Preliminary Design Information:

Controlling Design Criteria

A: At transfer, precasting plant:

T is maximum, little loading
Load = $T_{initial}$ (before losses) + M_g (due to girder weight)

Avoid high initial tension or compression with initial concrete strength.

B: At full service load, final loading (say after 50 years):

T is minimum, load is max
Load = $T_{initial}$ (before losses) + M_g (max service moment)

Avoid cracking and limit concrete stress.

At transfer (Interior Girder):

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

$M_g := w_g \cdot \frac{L_g^{\,2}}{8}$ $\boxed{M_g = 2574}$ kip-ft

After 50 Years (Interior Girder):

Service 1 Moment $\boxed{M_{s1} = 8659}$ kip-ft

Service 3 Moment $\boxed{M_{s3} = 8045}$ kip-ft

Service 1 Moment Components:

non-composite moment (girder + deck) $\boxed{M_{nc} = 4887}$ kip-ft

composite moment (parapet, FWS and LL)

$M_{1c} := M_{s1} - M_{nc}$ $\boxed{M_{1c} = 3772}$ kip-ft

Service 3 Moment Components:

non-composite moment (girder + deck) $\boxed{M_{nc} = 4887}$ kip-ft

composite moment (parapet, FWS and LL)

$M_{3c} := M_{s3} - M_{nc}$ $\boxed{M_{3c} = 3157}$ kip-ft

At 50 years the prestress has decreased (due to CR, SH, RE):

The approximate method of estimated time dependent losses is used by WisDOT. The lump sum loss estimate, I-girder loss **LRFD Table [5.9.5.3-1]**

Where PPR is the partial prestressing ratio, $PPR := 1.0$

$F_{delta} := 33 \cdot \left(1 - 0.15 \cdot \frac{f'_c - 6}{6}\right) + 6 \cdot PPR$ $\boxed{F_{delta} = 37.350}$ ksi

but, for low relaxation strand: $F_{Delta} := F_{delta} - 6$ $\boxed{F_{Delta} = 31.350}$ ksi

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.5.9.5.2.3a]** suggests assuming a 10% ES loss.

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

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

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

$$loss := F_{Delta} + \Delta f_{pES\_est} \qquad \boxed{loss = 49.350} \ \text{ksi}$$

$$loss_{\%} := \frac{loss}{f_{tr}} \cdot 100 \qquad \boxed{loss_{\%} = 24.370} \ \%\ \text{(estimated)}$$

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

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

$$ratio := 1 - \frac{loss_{\%}}{100} \qquad \boxed{ratio = 0.756}$$

$$T = ratio \cdot T_o$$

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 50 years.

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 50 years.

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 50 years.

Near center span, after 50 years, 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:

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

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.

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

OR:

where:

$A_{ps} := ns \cdot A_s$ $\boxed{A_{ps} = 9.98}$ in^2

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

LRFD [5.7.2.2]

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

$d_p := y_t + hau + t_{se} - e_s$ $\boxed{d_p = 77.15}$ in

$$c := \frac{A_{ps} \cdot f_{pu}}{0.85 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 9.99}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 8.49}$ 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 $\boxed{h_f = 7.500}$ in

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

$$c := \frac{A_{ps} \cdot f_{pu} - 0.85 \cdot f'_{cd} \cdot \left(b - w_{tf}\right) \cdot h_f}{0.85 \cdot f'_{cd} \cdot \beta_1 \cdot w_{tf} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 10.937}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 9.30}$ 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:

$$f_{ps} := f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right)$$ $\boxed{f_{ps} = 259.283}$ ksi

$T_u := f_{ps} \cdot A_{ps}$ $\boxed{T_u = 2588}$ kips

Calculate the nominal moment capacity of the composite section in accordance with **LRFD [5.7.3.2]**:

$$M_n := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + 0.85 \cdot f'_{cd} \cdot \left(b - w_{tf}\right) \cdot h_f \cdot \left(\frac{a}{2} - \frac{h_f}{2}\right)\right] \cdot \frac{1}{12}$$

$\boxed{M_n = 15717}$ kip-ft

For prestressed concrete, $\phi_f := 1.00$, **LRFD [5.5.4.2.1]**. Therefore the usable capacity is:

$M_r := \phi_f \cdot M_n$ $\boxed{M_r = 15717}$ 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 [5.7.3.3.2]** for the interior girder:

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

$f_r := 0.37 \cdot \sqrt{f'_c}$ **LRFD [5.4.2.6]** $\boxed{f_r = 1.047}$ ksi

$$f_{cpe} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_b}$$ $\boxed{f_{cpe} = 4.348}$ 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

$$M_{cr} := \gamma_3 \cdot \left[S_c \cdot \left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) \cdot \frac{1}{12} - M_{dnc} \cdot \left(\frac{S_c}{S_{nc}} - 1 \right) \right]$$ $\boxed{M_{cr} = 11761}$ 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 exterior girder 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 :

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

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

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

Calculate the neutral axis location for a flanged section:

$$c_x := \frac{A_{ps} \cdot f_{pu} - 0.85 \cdot f'_{cd} \cdot (b_x - w_{tf}) \cdot h_f}{0.85 \cdot f'_{cd} \cdot \beta_1 \cdot w_{tf} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}} \qquad \boxed{c_x = 13.51} \text{ in}$$

$$a_x := \beta_1 \cdot c_x \qquad \boxed{a_x = 11.49} \text{ in}$$

Now calculate the effective tendon stress at ultimate:

$$f_{ps\_x} := f_{pu} \cdot \left(1 - k \cdot \frac{c_x}{d_p}\right) \qquad \boxed{f_{ps\_x} = 256.759} \text{ ksi}$$

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

$$M_{n\_x} := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a_x}{2}\right) + 0.85 \cdot f'_{cd} \cdot (b_x - w_{tf}) \cdot h_f \cdot \left(\frac{a_x}{2} - \frac{h_f}{2}\right)\right] \cdot \frac{1}{12}$$

$$\boxed{M_{n\_x} = 15515} \text{ kip-ft}$$

$$M_{r\_x} := \phi_f \cdot M_{n\_x} \qquad \boxed{M_{r\_x} = 15515} \text{ kip-ft}$$

$\boxed{1.33M_{strx} = 14874}$ 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, as now required by a WisDOT policy item for all girders, is omitted. This example is not yet revised.*

Exterior Beams:

Two or more lanes loaded:

The distribution from the centerline of the exterior beam to the inside edge of the parapet, $d_e = 1.25$ feet.

$$e_v := 0.6 + \frac{d_e}{10} \qquad \boxed{e_v = 0.725}$$

$$g_{vx1} := e_v \cdot g_{vi} \qquad \boxed{g_{vx1} = 0.565}$$

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}..

$$g_{vx2} := g_{x2} \quad = R_x \cdot 1.2 \qquad \boxed{g_{vx2} = 0.600}$$

$g_{vx} := \max(g_{vx1}, g_{vx2})$ $\boxed{g_{vx} = 0.600}$

Apply the shear magnification factor in accordance with **LRFD [4.6.2.2.3c]**.

$$skew_{correction} := 1.0 + 0.2 \cdot \left(\frac{12L \cdot t_s^{\,3}}{K_g}\right)^{0.3} \cdot \tan\left(skew \cdot \frac{\pi}{180}\right)$$

$\boxed{L = 146.00}$

$\boxed{t_s = 8.00}$

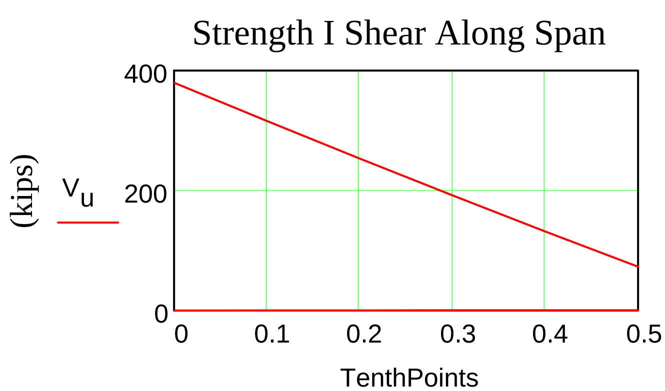
$\boxed{K_g = 3600866}$

$\boxed{skew = 20.000}$ $\boxed{skew_{correction} = 1.048}$

$g_{vx} := g_{vx} \cdot skew_{correction}$ $\boxed{g_{vx} = 0.629}$

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.



$\boxed{Vu_{0.0} = 379.7}$ kips

$\boxed{Vu_{0.5} = 72.9}$ kips

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 [5.8.3.2]**.

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 [5.8.2.9]**

The first estimate of d_v is calculated as follows:

$$d_v := -e_s + y_t + hau + t_{se} - \frac{a}{2}$$ $\boxed{d_v = 72.50}$ 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 $d_v := 65$ inches.

For the standard bearing pad of width, $w_{brg} := 8$ inches, the distance from the end of the girder to the critical section:

$$L_{crit} := \left(\frac{w_{brg}}{2} + d_v\right)\cdot\frac{1}{12} + 0.5$$ $\boxed{L_{crit} = 6.25}$ ft

Calculate the eccentricity of the strand group at the critical section.

$$y_{8t\_crit} := y_{8t} - \frac{slope}{100}\cdot L_{crit}\cdot 12$$ $\boxed{y_{8t\_crit} = 24.22}$ in

$$e_{s\_crit} := \frac{ns_s\cdot y_s + ns_d\cdot y_{8t\_crit}}{ns_s + ns_d}$$ $\boxed{e_{s\_crit} = -21.11}$ in

Calculation of compression stress block based on revised eccentricity:

$$d_{p\_crit} := y_t + hau + t_{se} - e_{s\_crit}$$ $\boxed{d_{p\_crit} = 67.74}$ in

$$A_{ps\_crit} := (ns_d + ns_s)\cdot A_s$$ $\boxed{A_{ps\_crit} = 9.98}$ 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 [5.11.4.2]**:

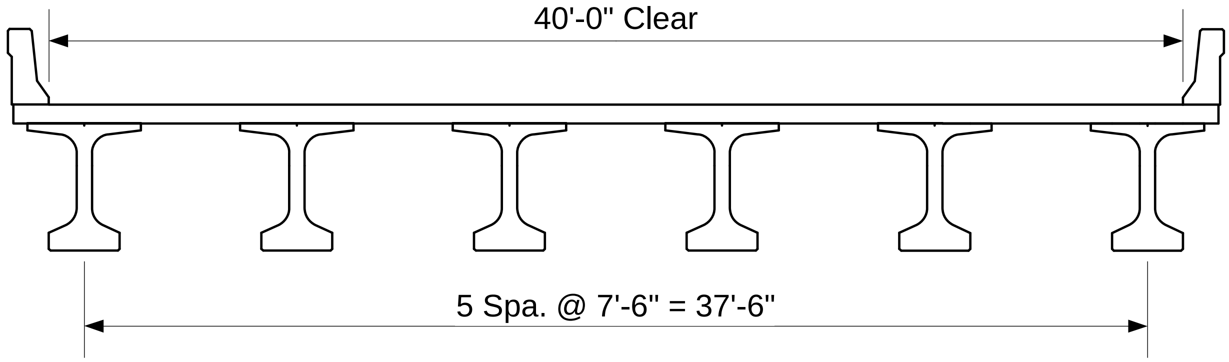
$K := 1.6$ for prestressed members with a depth greater than 24 inches

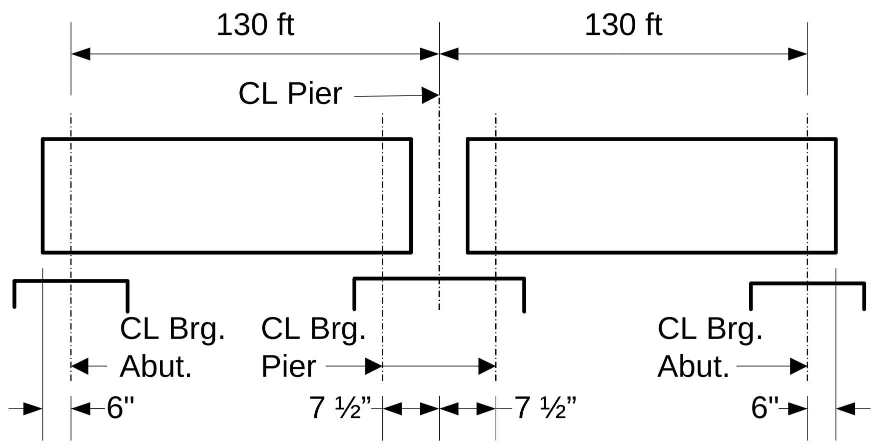
Table of Contents

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 Sixth Edition - 2012)***

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

For flexure in non-prestressed concrete, $\phi_f := 0.9$.

The width of the bottom flange of the girder, $b_w = 30.00$ inches.

$$R_u := \frac{M_u \cdot 12}{\phi_f \cdot b_w \cdot d_e^2} \qquad \boxed{R_u = 0.532} \text{ ksi}$$

$$\rho := 0.85 \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R_u}{0.85 \cdot f'_c}}\right) \qquad \boxed{\rho = 0.00925}$$

$$A_s := \rho \cdot b_w \cdot d_e \qquad \boxed{A_s = 16.74} \text{ in}^2$$

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:

$$As_{req} := \frac{A_s}{\frac{w_e}{12}} \qquad \boxed{As_{req} = 2.232} \text{ in}^2\text{/ft}$$

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:

$$As := As_{prov} \cdot \frac{w_e}{12} \qquad \boxed{As = 21.18} \text{ in}^2$$

Calculate the capacity of the section in flexure at the pier:

Check the depth of the compression block:

$$a := \frac{As \cdot f_y}{0.85 \cdot b_w \cdot f'_c} \qquad \boxed{a = 6.228} \text{ in}$$

This is within the thickness of the bottom flange height of 7.5 inches.

$$M_n := As \cdot f_y \cdot \left(d_e - \frac{a}{2}\right) \cdot \frac{1}{12} \qquad \boxed{M_n = 6056} \text{ kip-ft}$$

$$M_r := \phi_f \cdot M_n \qquad \boxed{M_r = 5451} \text{ kip-ft}$$

$$\boxed{M_u = 4358} \text{ kip-ft}$$

Is M_u less than M_r? $\boxed{\text{check} = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$f_r := 0.37 \cdot \sqrt{f'_{cd}}$ $\boxed{f_r = 0.740}$ 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
for A615, Grade 60 reinforcement

$M_{cr} := 1.1 f_r \cdot S_c \cdot \frac{1}{12}$ $\boxed{M_{cr} = 2635}$ kip-ft

$\boxed{1.33 \cdot M_u = 5796}$ kip-ft

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

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$\rho := \frac{As}{b_w \cdot d_e}$ $\boxed{\rho = 0.01170}$

$n := \frac{E_S}{E_B}$ $\boxed{n = 4.566}$

$k := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n} - \rho \cdot n$ $\boxed{k = 0.278}$

$j := 1 - \frac{k}{3}$ $\boxed{j = 0.907}$

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

$f_s := \frac{M_{s1}}{As \cdot j \cdot d_e} \cdot 12$ $\boxed{f_s = 27.006}$ ksi

The height of the composite section, h, is:

$h := ht + hau + t_{se}$ $\boxed{h = 63.500}$ in

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.

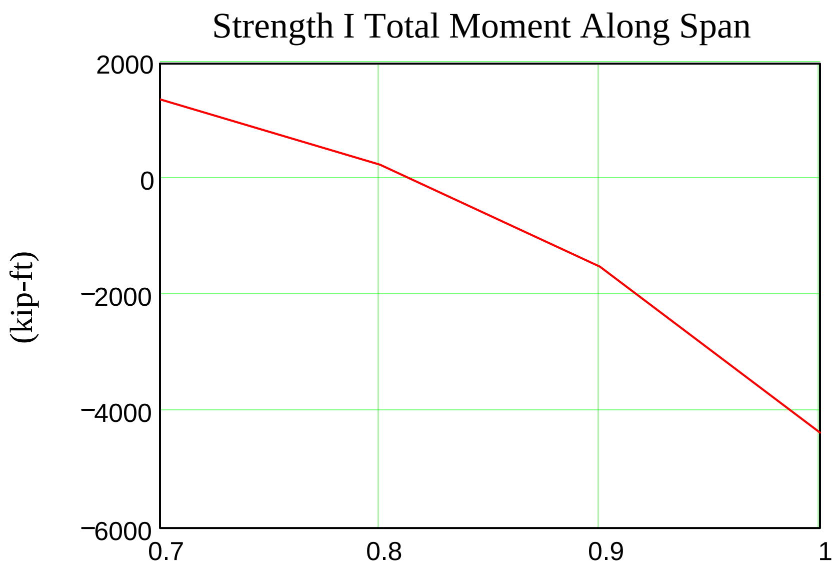
$spa' := spa \cdot 2$ $\boxed{spa' = 8.50}$ in

$As' := \frac{As}{2}$ $\boxed{As' = 10.588}$ in^2

$a' := \frac{As' \cdot f_y}{0.85 \cdot b_w \cdot f'_c}$ $\boxed{a' = 3.11}$ 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.

$M_{r'} = 2799$ kip-ft

$Mu_{cut1} = 1501$ kip-ft

$Ms_{cut1} = 1565$ kip-ft

Is Mu_{cut1} less than $M_{r'}$? check = "OK"

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$M_{cr} = 2635$ kip-ft

$1.33 \cdot Mu_{cut1} = 1996$ kip-ft

Is $M_{r'}$ greater than the lesser value of M_{cr} and 1.33*Mu_{cut1}? check = "OK"

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$$\rho' := \frac{As'}{b_w \cdot d_e} \qquad \rho' = 0.00585$$

$$k' := \sqrt{(\rho' \cdot n)^2 + 2 \cdot \rho' \cdot n} - \rho' \cdot n \qquad k' = 0.206$$

$$j' := 1 - \frac{k'}{3} \qquad j' = 0.931$$

$Ms_{cut1} = 1565$ kip-ft

$$f_{s'} := \frac{Ms_{cut1}}{As' \cdot j' \cdot d_e} \cdot 12 \qquad f_{s'} = 31.582 \text{ ksi}$$

$\beta = 1.076$

$\gamma_e = 0.750$

$$S_{max'} := \frac{700\gamma_e}{\beta \cdot f_{s'}} - 2 \cdot d_c \qquad S_{max'} = 9.08 \text{ in}$$

$spa' = 8.50$ in

Is the bar spacing less than $S_{max'}$? check = "OK"

Table of Contents

E19-3 Box Section Beam

This example shows design calculations for a single span prestressed box multi-beam bridge having a 2" concrete overlay and is designed for a 20 pound per square foot future wearing surface. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Sixth Edition - 2012)***

E19-3.1 Preliminary Structure Data

Design Data
A-1 Abutments at both ends
Skew: 0 degrees
Live Load: HL-93
Roadway Width: 28 ft. minimum clear

$L := 44$	Span Length, single span, ft
$L_g := 44.5$	Girder Length, the girder extends 3" past the CL bearing at each abutment, single span, ft
$N_L := 2$	Number of design lanes
$t_{overlay} := 2$	Minimum overlay thickness, inches
$f_{pu} := 270$	Ultimate tensile strength for low relaxation strands, ksi
$d_s := 0.5$	Strand diameter, inches
$A_s := 0.1531$	Area of prestressing strands, in^2
$E_s := 28500$	Modulus of elasticity of the prestressing strands, ksi
$f'_c := 5$	Concrete strength (prestressed box girder), ksi
$f'_{ci} := 4.25$	Concrete strength at release, ksi
$K_1 := 1.0$	Aggregate correction factor
$w_c := 0.150$	Unit weight of concrete for box girder, overlay, and grout, kcf
$fy := 60$	Bar steel reinforcement, Grade 60, ksi.
$w_{rail} := 0.075$	Weight of Type "M" rail, klf
$Wh_{rail} := 0.42$	Width of horizontal members of Type "M" rail, feet
$\mu := 0.20$	Poisson's ratio for concrete, **LRFD [5.4.2.5]**

Based on past experience, the modulus of elasticity for the precast concrete are given in Chapter 19 as $E_{beam6} := 5500$ ksi for a concrete strength of 6 ksi. The values of E for different concrete strengths are calculated as follows:

E19-3.7 Load Factors

From **LRFD [Table 3.4.11]**:

	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	$\gamma s3_{DC} := 1.0$	$\gamma s3_{DW} := 1.0$	$\gamma s3_{LL} := 0.8$
Fatigue 1			$\gamma f_{LL} := 1.5$

E19-3.8 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 exterior girder:

Strength 1

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

$$= 1.0 \cdot \left(1.25 \cdot M_{DC} + 1.50 \cdot M_{DW} + 1.75 \cdot M_{LL}\right) \qquad \boxed{M_{str} = 862} \quad \text{kip-ft}$$

Service 1 (for compression checks)

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

$$= 1.0 \cdot \left(1.0 \cdot M_{DC} + 1.0 \cdot M_{DW} + 1.0 \cdot M_{LL}\right) \qquad \boxed{M_{s1} = 554} \quad \text{kip-ft}$$

Service 3 (for tension checks)

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

$$= 1.0 \cdot \left(1.0 \cdot M_{DC} + 1.0 \cdot M_{DW} + 0.8 \cdot M_{LL}\right) \qquad \boxed{M_{s3} = 488} \quad \text{kip-ft}$$

Fatigue 1 (for compression checks)

$$M_{f1} := \eta \cdot \left[\frac{1}{2} \cdot \left(M_{DC} + M_{DW}\right) + \gamma f_{LL} \cdot M_{LLf}\right]$$

$$= 1.0 \cdot \left[\frac{1}{2} \cdot \left(M_{DC} + M_{DW}\right) + 1.5 \cdot M_{LLf}\right] \qquad \boxed{M_{f1} = 330} \quad \text{kip-ft}$$

E19-3.9 Allowable Stress

Allowable stresses are determined for 2 sages for prestressed girders. Temporary allowable stresses are set for the loading stage at release of the prestressing strands. Final condition allowable stresses are checked at the end of 50 years of service.

E19-3.9.1 Temporary Allowable Stresses

The temporary allowable stress (compression) **LRFD [5.9.4.1.1]**:

$$f_{ciall} := 0.60 \cdot f'_{ci} \qquad \boxed{f_{ciall} = 2.550} \text{ ksi}$$

In accordance with **LRFD Table [5.9.4.1.2-1]**, the temporary allowable tension stress is calculated as follows (assume there is no bonded reinforcement):

$$f_{tiall} := -\min\left(0.0948 \cdot \sqrt{f'_{ci}}, 0.2\right) \qquad \boxed{f_{tiall} = -0.195} \text{ ksi}$$

If bonded reinforcement is present in the top flange, the temporary allowable tension stress is calculated as follows:

$$f_{tiall\_bond} := -0.24 \cdot \sqrt{f'_{ci}} \qquad \boxed{f_{tiall\_bond} = -0.495} \text{ ksi}$$

E19-3.9.2 Final Condition Allowable Stresses

Allowable Stresses,**LRFD [5.9.4.2]**:

There are three compressive service stress limits:

$$f_{call1} := 0.45 \cdot f'_c \quad \text{PS + DL} \qquad \boxed{f_{call1} = 2.250} \text{ ksi}$$

$$f_{call2} := 0.40 \cdot f'_c \quad \text{LL + 1/2(PS + DL)} \qquad \boxed{f_{call2} = 2.000} \text{ ksi}$$

$$f_{call3} := 0.60 \cdot f'_c \quad \text{LL + PS + DL} \qquad \boxed{f_{call3} = 3.000} \text{ ksi}$$

There us one tension service stress limit:

$$f_{tall} := -0.19 \cdot \sqrt{f'_c} \quad \text{LL + PS + DL} \qquad \boxed{f_{tall} = -0.425} \text{ ksi}$$

There is one compressive fatigue stress limit:

$$f_{call\_f} := 0.40 \cdot f'_c \quad \text{LLf + 1/2(PS + DL)} \qquad \boxed{f_{call2} = 2.000} \text{ ksi}$$

E19-3.10 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 50 years.

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

3) Check resulting stresses at the critical sections of the girder at the time of transfer and after 50 years.

E19-3.10.1 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 50 years.

Near center span, after 50 years, 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 exterior girder has the controlling moments.

Calculate the stress at the bottom of the beam due to the Service 3 loading:

$$f_b := \frac{M_{s3} \cdot 12}{S_b} \qquad \boxed{f_b = -1.867} \text{ ksi}$$

Stress at bottom due to prestressing:

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

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. The required stress due to prestress force at bottom of section to counteract the Service 3 loads:

$$\boxed{f_{bp} = 1.867} \text{ ksi}$$

E19-3.10.1.1 Estimate the Prestress Losses

At 50 years the prestress has decreased (due to CR, SH, RE):

The approximate method of estimated time dependent losses is used by WisDOT. The lump sum loss estimate, I-girder loss **LRFD Table [5.9.5.3-1]**

Where PPR is the partial prestressing ratio, $PPR := 1.0$

$F_{delta} := 26 + 4 \cdot PPR$ $\boxed{F_{delta} = 30}$ ksi

but, for low relaxation strand: $F_{Delta} := F_{delta} - 6$ $\boxed{F_{Delta} = 24}$ ksi

Assume an initial strand stress; $f_{tr} := 0.75 \cdot f_{pu}$ $\boxed{f_{tr} = 202.5}$ ksi

Based on experience, assume $\Delta f_{pES\_est} := 9.1$ ksi loss from elastic shortening. As an alternate initial estimate, **LRFD [C.5.9.5.2.3a]** suggests assuming a 10% ES loss.

$$ES_{loss} := \frac{\Delta f_{pES\_est}}{f_{tr}} \cdot 100$$ $\boxed{ES_{loss} = 4.494}$ %

$f_i := f_{tr} - \Delta f_{pES\_est}$ $\boxed{f_i = 193.4}$ ksi

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

$loss := F_{Delta} + \Delta f_{pES\_est}$ $\boxed{loss = 33.1}$ ksi

$$loss_{\%} := \frac{loss}{f_{tr}} \cdot 100$$ $\boxed{loss_{\%} = 16.346}$ % (estimated)

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

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

$$ratio := 1 - \frac{loss_{\%}}{100}$$ $\boxed{ratio = 0.837}$

$$T = ratio \cdot T_o$$

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

OR:

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

$$f_{bpi} := \frac{f_{bp}}{1 - \frac{loss_{\%}}{100}}$$

$\boxed{f_{bpi} = 2.231}$ ksi

desired bottom initial prestress

E19-3.10.1.2 Determine Number of Strands

$A_s = 0.153$ in^2

$f_{pu} = 270$ ksi

$f_s := 0.75 \cdot f_{pu}$ $\boxed{f_s = 202.5}$ ksi

$P := A_s \cdot f_s$ $\boxed{P = 31.003}$ kips per strand

$$f_{bp} := \frac{P \cdot N}{A} \cdot \left(1 + e \cdot \frac{y_b}{r_{sq}}\right)$$

$y_b = -10.5$ Distance from the centroid of the 21" depth to the bottom of the box section, in.

For the 4'-0 wide box sections, there can be up to 22 strands in the bottom row and 2 rows of strands in the sides of the box. Calculate the eccentricity for the maximum number of strands that can be placed in the bottom row of the box:

$e_b := y_b + 2$ $e_b = -8.5$ Eccentricity to the bottom row of strands, inches

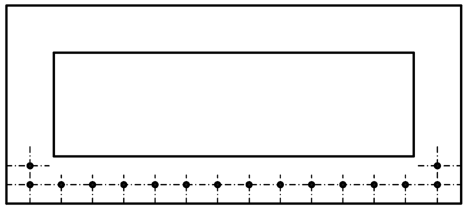
$e_s := e_b$ $\boxed{e_s = -8.5}$ inches

$$N_{req} := \frac{f_{bpi} \cdot A}{P} \cdot \frac{1}{1 + e_s \cdot \frac{y_b}{r_{sq}}}$$

$\boxed{N_{req} = 16.4}$ strands

Therefore, try $N := 16$ strands since some final tension in the bottom of the girder is allowed.

Place 2 of the strands in the second row:



$$e_s := \frac{e_b \cdot 14 + (e_b + 2) \cdot 2}{16}$$

$\boxed{e_s = -8.25}$ inches

E19-3.10.2 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. Can this be compensated for by overstressing?

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

E19-3.10.2.1 Elastic Shortening Loss

at transfer (before ES loss) **LRFD [5.9.5.2]**

$T_{oi} := N \cdot f_{tr} \cdot A_s$ $\boxed{= 16 \cdot 0.75 \cdot 270 \cdot 0.1531 = 496}$ kips

The ES loss estimated above was: $\Delta f_{pES\_est} = 9.1$ ksi, or $ES_{loss} = 4.494$ %. The resulting force in the strands after ES loss:

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

$\boxed{T_o = 474}$ kips

Since all strands are straight, we can calculate the initial elastic shortening loss;

$$f_{cgp} := \frac{T_o}{A} + (T_o \cdot e_s) \cdot \frac{e_s}{I} + M_{gi} \cdot 12 \cdot \frac{e_s}{I}$$

$\boxed{f_{cgp} = 1.264}$ ksi

$\boxed{E_{ct} = 3952}$ ksi

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

$$\Delta f_{pES} := \frac{E_p}{E_{ct}} \cdot f_{cgp} \qquad \boxed{\Delta f_{pES} = 9.118} \text{ ksi}$$

This value of Δf_{pES} is in agreement with the estimated value above; $\Delta f_{pES\_est} = 9.10$ 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:

$$f_i := f_{tr} - \Delta f_{pES} \qquad \boxed{f_i = 193.382} \text{ ksi}$$

The force in the beam after transfer is:

$$T_o := N \cdot A_s \cdot f_i \qquad \boxed{T_o = 474} \text{ kips}$$

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.

$$f_{ttr} := \frac{T_o}{A} + \frac{T_o \cdot e_s}{S_t} + \frac{M_{gi} \cdot 12}{S_t} \qquad \boxed{f_{ttr} = 0.200} \text{ ksi}$$

$$f_{btr} := \frac{T_o}{A} + \frac{T_o \cdot e_s}{S_b} + \frac{M_{gi} \cdot 12}{S_b} \qquad \boxed{f_{btr} = 1.392} \text{ ksi}$$

temporary allowable stress (tension) $\boxed{f_{tiall} = -0.195}$ ksi

temporary allowable stress (compression) $\boxed{f_{ciall} = 2.550}$ ksi

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

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

E19-3.10.2.2 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 [5.9.5.3]**.

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

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

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

$\gamma_{st} := \dfrac{5}{1 + f'_{ci}}$ $\qquad \boxed{\gamma_{st} = 0.952}$

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

$\Delta f_{pCR} := 10.0 \cdot \dfrac{f_{tr} \cdot A_s \cdot N}{A} \cdot \gamma_h \cdot \gamma_{st}$ $\qquad \boxed{\Delta f_{pCR} = 7.781}$ ksi

$\Delta f_{pSR} := 12.0 \cdot \gamma_h \cdot \gamma_{st}$ $\qquad \boxed{\Delta f_{pSR} = 11.200}$ ksi

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

$\Delta f_{pLT} := \Delta f_{pCR} + \Delta f_{pSR} + \Delta f_{pRE}$ $\qquad \boxed{\Delta f_{pLT} = 21.381}$ ksi

The total estimated prestress loss (Approximate Method):

$\Delta f_p := \Delta f_{pES} + \Delta f_{pLT}$ $\qquad \boxed{\Delta f_p = 30.499}$ ksi

$\boxed{\dfrac{\Delta f_p}{f_{tr}} \cdot 100 = 15.061}$ % total prestress loss

This value is less than but in general agreement with the initial estimated $loss_{\%} = 16.3$.

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

$f_{pe} := f_{tr} - \Delta f_p$ $\qquad \boxed{f_{pe} = 172.00}$ ksi

$T := N \cdot A_s \cdot f_{pe}$ $\qquad \boxed{T = 421}$ kips

E19-3.10.3 Check Stresses at Critical Locations

<u>Check the girder stresses at the end of the transfer length of the strands at release:</u>

Minimum moment on section = girder moment at the plant

$$M_{gz} = \frac{w_g}{2} \cdot \left(L_g \cdot z - z^2\right)$$

Stress in the bottom fiber at transfer:

$$f_{bz} = \frac{T_o}{A} + \frac{T_o \cdot e_{sz}}{S_b} + \frac{M_{gz}}{S_b}$$

The transfer length may be taken as:

$$l_{tr} := 60 \cdot d_s \qquad \boxed{l_{tr} = 30.00} \text{ in}$$

$$x := \frac{l_{tr}}{12} \qquad \boxed{x = 2.50} \text{ feet}$$

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

$$M_{gt} := \frac{w_{g\_ext}}{2} \cdot \left(L_g \cdot x - x^2\right) + \left(\frac{w_{diaph} \cdot x}{2} + w_{diaph\_end} \cdot x\right)$$

$$\boxed{M_{gt} = 38} \text{ kip-ft}$$

The girder stresses at the end of the transfer length:

$$f_{tt} := \frac{T_o}{A} + \frac{T_o \cdot e_s}{S_t} + \frac{M_{gt} \cdot 12}{S_t} \qquad \boxed{f_{tt} = -0.303} \text{ ksi}$$

$$\boxed{f_{tiall} = -0.195} \text{ ksi}$$

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

If bonded reinforcement is provided in the top flange, the allowable stress is:

$$f_{tiall\_bond} = -0.495 \text{ ksi}$$

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

$$f_{bt} := \frac{T_o}{A} + \frac{T_o \cdot e_s}{S_b} + \frac{M_{gt} \cdot 12}{S_b} \qquad \boxed{f_{bt} = 1.896} \text{ ksi}$$

$$\boxed{f_{ciall} = 2.55} \text{ ksi}$$

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

Check final stresses after all losses at the mid-span of the girder:

Top of girder stress (Compression - Service 1):

$$f_{t1} := \frac{T}{A} + \frac{T \cdot e_s}{S_t} + \frac{\left(M_{DC} + M_{DW}\right) \cdot 12}{S_t} \quad \text{PS + DL} \qquad \boxed{f_{t1} = 0.459} \text{ ksi}$$

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

$$f_{t2} := \frac{1}{2} \cdot f_{t1} + \frac{M_{LL} \cdot 12}{S_t}$$ LL + 1/2(PS + DL) $\boxed{f_{t2} = 1.489}$ ksi

$\boxed{check = "OK"}$

$$f_{t3} := \frac{T}{A} + \frac{T \cdot e_s}{S_t} + \frac{M_{s1} \cdot 12}{S_t}$$ LL + PS + DL $\boxed{f_{t3} = 1.719}$ ksi

$\boxed{check = "OK"}$

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

$$f_{b1} := \frac{T}{A} + \frac{T \cdot e_s}{S_b} + \frac{(M_{DC} + M_{DW}) \cdot 12}{S_b}$$ PS + DL $\boxed{f_{b1} = 0.958}$ ksi

$\boxed{check = "OK"}$

<u>Bottom of girder stress (Tension - Service 3):</u>

$$f_b := \frac{T}{A} + \frac{T \cdot e_s}{S_b} + \frac{M_{s3} \cdot 12}{S_b}$$ $\boxed{f_b = -0.051}$ ksi

$\boxed{check = "OK"}$

<u>Top of girder stress (Compression - Fatigue 1):</u>

$$f_{tf1} := \frac{1}{2} \cdot \left[\frac{T}{A} + \frac{T \cdot e_s}{S_t} + \frac{(M_{DC} + M_{DW}) \cdot 12}{S_t} \right] + \frac{M_{LLf} \cdot 12}{S_t}$$ 1/2(PS + DL) + LLf

$\boxed{f_{t1} = 0.459}$ ksi

$\boxed{check = "OK"}$

allowable stress (tension) $\boxed{f_{tall} = -0.425}$ ksi

allowable stress (compression) $\boxed{f_{call1} = 2.250}$ ksi

$\boxed{f_{call2} = 2.000}$ ksi

$\boxed{f_{call3} = 3.000}$ ksi

$\boxed{f_{call\_f} = 2.000}$ ksi

E19-3.11 Flexural Capacity at Midspan

Check f_{pe} in accordance with **LRFD [5.7.3.1.1]**:

$\boxed{f_{pe} = 172}$ ksi $\boxed{0.5 \cdot f_{pu} = 135}$ 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 [C5.7.3.1.1-1]**, for low relaxation strands, $k := 0.28$.

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

Assume that the compression block is in the top section of the box. Calculate the capacity as if it is a rectangular section. The neutral axis location,calculated in accordance with **LRFD 5.7.3.1.1** for a rectangular section, is:

$$c = \frac{A_{ps} \cdot f_{pu}}{0.85 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$

where:

$A_{ps} := N \cdot A_s$ $\boxed{A_{ps} = 2.45}$ in^2

$b := W_s \cdot 12$ $\boxed{b = 48.00}$ in

LRFD [5.7.2.2]

$\beta_1 := \max\left[0.85 - \left(f'_c - 4\right) \cdot 0.05, 0.65\right]$ $\boxed{\beta_1 = 0.800}$

$d_p := y_t - e_s$ $\boxed{d_p = 18.75}$ in

$$c := \frac{A_{ps} \cdot f_{pu}}{0.85 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 3.82}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 3.06}$ in

This is within the depth of the top slab (5-inches). Therefore our assumption is OK.

Now calculate the effective tendon stress at ultimate:

$$f_{ps} := f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right)$$ $\boxed{f_{ps} = 254.6}$ ksi

$$T_u := f_{ps} \cdot A_{ps}$$ $\boxed{T_u = 624}$ kips

Calculate the nominal moment capacity of the section in accordance with **LRFD [5.7.3.2]**:

$$M_n := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right)\right] \cdot \frac{1}{12}$$

$\boxed{M_n = 895}$ kip-ft

For prestressed concrete, $\phi_f := 1.00$, **LRFD [5.5.4.2.1]**. Therefore the usable capacity is:

$$M_r := \phi_f \cdot M_n$$ $\boxed{M_r = 895}$ kip-ft

The required capacity:

Exterior Girder Moment

$$M_u := M_{str}$$ $\boxed{M_u = 862}$ kip-ft

Check the section for minimum reinforcement in accordance with **LRFD [5.7.3.3.2]** for the interior girder:

$\boxed{1.33 \cdot M_u = 1147}$ kip-ft

$$f_r := 0.37 \cdot \sqrt{f'_c}$$ **LRFD [5.4.2.6]** $\boxed{f_r = 0.827}$ ksi

$$f_{cpe} := \frac{T}{A} + \frac{T \cdot e_s}{S_b}$$ $\boxed{f_{cpe} = 1.816}$ ksi

$$S_c := -S_b$$ $\boxed{S_c = 3137}$ ksi

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_2 := 1.1$ prestress variability factor

$\gamma_3 := 1.0$ for prestressed concrete structures

$M_{cr} := \gamma_3 \cdot \left[S_c \cdot \left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) \cdot \frac{1}{12} \right]$ $\boxed{M_{cr} = 868}$ kip-ft

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

E19-3.12 Shear Analysis

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

The live load shear distribution factors to the girders are calculated above in E19-3.2.2.

$\boxed{g_{int\_v} = 0.600}$

$\boxed{g_{ext\_v} = 0.744}$

From section E19-3.4, the uniform dead loads on the girders are:

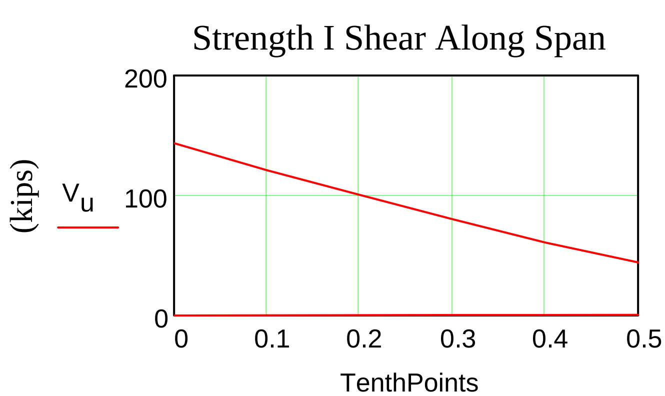
Interior Girder $\boxed{w_{DCint} = 0.792}$ klf

$\boxed{w_{DWint} = 0.082}$ klf

Exterior Girder $\boxed{w_{DCext} = 0.845}$ klf

$\boxed{w_{DWext} = 0.083}$ klf

However, the internal concrete diaphragms were applied as total equivalent uniform loads to determine the maximum mid-span moment. The diaphragm weights should be applied as point loads for the shear calculations.



$\boxed{Vu_{0.0} = 143.5}$ kips

$\boxed{Vu_{0.5} = 44.2}$ kips

Simplified Procedure for Prestressed and Nonprestressed Sections, **LRFD [5.8.3.4.3]**

$b_v := 2t_w$ $\boxed{b_v = 10.00}$ in

The critical section for shear is taken at a distance of d_v from the face of the support, **LRFD [5.8.3.2]**.

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 [5.8.2.9]**

The first estimate of d_v is calculated as follows:

$d_v := -e_s + y_t - \frac{a}{2}$ $\boxed{d_v = 17.22}$ in

For the standard bearing pad of width, $w_{brg} := 8$ inches, the distance from the end of the girder to the critical section:

$L_{crit} := (w_{brg} + d_v) \cdot \frac{1}{12}$ $\boxed{L_{crit} = 2.10}$ ft

The eccentricity of the strand group at the critical section is:

$\boxed{e_s = -8.25}$ in

Calculation of compression stress block:

$\boxed{d_p = 18.75}$ in

$\boxed{A_{ps} = 2.45}$ 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 [5.11.4.2]**:

$K := 1.0$ for prestressed members with a depth less than 24 inches

$\boxed{d_s = 0.5}$ in

$l_d := K \cdot \left(f_{ps} - \frac{2}{3} \cdot f_{pe}\right) \cdot d_s$ $\boxed{l_d = 70.0}$ in

The transfer length may be taken as: $l_{tr} := 60 \cdot d_s$ $\boxed{l_{tr} = 30.00}$ in

Since $L_{crit} = 2.102$ feet is between the transfer length and the development length, the design stress in the prestressing strand is calculated as follows:

$$f_{pu\_crit} := f_{pe} \cdot \frac{L_{crit} \cdot 12}{l_{tr}}$$ $\boxed{f_{pu\_crit} = 145}$ ksi

$$T_{crit} := N \cdot A_s \cdot f_{pu\_crit}$$ $\boxed{T_{crit} = 354}$ kips

For rectangular section behavior:

$$c_{crit} := \frac{A_{ps} \cdot f_{pu\_crit}}{0.85 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu\_crit}}{d_p}}$$ $\boxed{c_{crit} = 2.102}$ in

$$a_{crit} := \beta_1 \cdot c_{crit}$$ $\boxed{a_{crit} = 1.682}$ in

Calculation of shear depth based on refined calculations of a:

$$d_{v\_crit} := -e_s + y_t - \frac{a_{crit}}{2}$$ $\boxed{d_{v\_crit} = 17.91}$ in

This value matches the assumed value of d_v above. OK!

$$d_v := d_{v\_crit}$$

The location of the critical section from the end of the girder is:

$$L_{crit} := \left(w_{brg} + d_v\right) \cdot \frac{1}{12}$$ $\boxed{L_{crit} = 2.159}$ ft

The location of the critical section from the center line of bearing at the abutment is:

$$crit := L_{crit} - 0.25$$ $\boxed{crit = 1.909}$ ft

The nominal shear resistance of the section is calculated as follows, **LRFD [5.8.3.3]**:

$$V_n = \min\left(V_c + V_s + V_p, 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p\right)$$

where $V_p := 0$ in the calculation of V_n, if the simplified procedure is used (**LRFD [5.8.3.4.3]**).

Note, the value of V_p does not equal zero in the calculation of V_{cw}.

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 (Live 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 (Live 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, $L_{crit} = 2.16$ feet from the end of the girder at the abutment.

$\boxed{V_d = 18.3}$ kips

$\boxed{V_i = 109.5}$ kips

$\boxed{M_{dnc} = 37.3}$ kip-ft

$\boxed{M_{max} = 111.7}$ kip-ft

However, the equations below require the value of M_{max} to be in kip-in:

$\boxed{M_{max} = 1340}$ kip-in

$f_r := -0.20 \cdot \sqrt{f'_c}$ **LRFD [5.4.2.6]** $\boxed{f_r = -0.447}$ ksi

$\boxed{T = 421}$ kips

$$f_{cpe} := \frac{T_{crit}}{A} + \frac{T_{crit} \cdot e_s}{S_b}$$ $\boxed{f_{cpe} = 1.527}$ ksi

$\boxed{M_{dnc} = 37}$ kip-ft

$\boxed{M_{max} = 1340}$ kip-in

$S_c := S_b$ $\boxed{S_c = -3137}$ in^3

$S_{nc} := S_b$ $\boxed{S_{nc} = -3137}$ in^3

$M_{cre} := S_c \cdot \left(f_r - f_{cpe} - \frac{12M_{dnc}}{S_{nc}}\right)$ $\boxed{M_{cre} = 5746}$ kip-in

$V_{ci1} := 0.06 \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$ $\boxed{V_{ci1} = 24.0}$ kips

$V_{ci2} := 0.02 \cdot \sqrt{f'_c} \cdot b_v \cdot d_v + V_d + \frac{V_i \cdot M_{cre}}{M_{max}}$ $\boxed{V_{ci2} = 495.9}$ kips

$V_{ci} := \max(V_{ci1}, V_{ci2})$ $\boxed{V_{ci} = 495.9}$ kips

$f_t := \frac{T_{crit}}{A} + \frac{T_{crit} \cdot e_s}{S_t} + \frac{M_{dnc} \cdot 12}{S_t}$ $\boxed{f_t = -0.194}$ ksi

$f_b := \frac{T_{crit}}{A} + \frac{T_{crit} \cdot e_s}{S_b} + \frac{M_{dnc} \cdot 12}{S_b}$ $\boxed{f_b = 1.384}$ ksi

$\boxed{y_b = -10.50}$ in

$\boxed{h = 21.00}$ in

$f_{pc} := f_b - y_b \cdot \frac{f_t - f_b}{h}$ $\boxed{f_{pc} = 0.595}$ ksi

$V_{p\_cw} := 0$ (no strands are draped) $\boxed{V_{p\_cw} = 0.0}$ kips

$V_{cw} := \left(0.06 \cdot \sqrt{f'_c} + 0.30 \cdot f_{pc}\right) \cdot b_v \cdot d_v + V_{p\_cw}$ $\boxed{V_{cw} = 56.0}$ kips

$V_c := \min(V_{ci}, V_{cw})$ $\boxed{V_c = 56.0}$ kips

Calculate the required shear resistance:

$\phi_v := 0.9$ **LRFD [5.5.4.2]**

$V_{u\_crit} = \gamma st_{DC} \cdot V_{DCnc} + \gamma st_{DW} \cdot V_{DWnc} + \gamma st_{LL} \cdot Vu_{LL}$

$V_n := \frac{V_{u\_crit}}{\phi_v}$ $\boxed{V_n = 147.6}$ kips

Transverse Reinforcing Design at Critical Section:

The required steel capacity:

$V_s := V_n - V_c - V_p$ $\boxed{V_s = 91.6}$ kips

$A_v := 0.40$ in^2 for 2 - #4 rebar

$f_y := 60$ ksi

$\boxed{d_v = 17.91}$ in

$$\cot\theta := \begin{cases} 1 & \text{if } V_{ci} < V_{cw} \\ \min\left(1.0 + 3 \cdot \dfrac{f_{pc}}{\sqrt{f'_c}}, 1.8\right) & \text{otherwise} \end{cases}$$ $\boxed{\cot\theta = 1.799}$

$$V_s = A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s}$$

LRFD Eq 5.8.3.3-4 reduced per **C5.8.3.3-1** when α = 90 degrees.

$$s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{V_s}$$ $\boxed{s = 8.441}$ in

Check Maximum Spacing, **LRFD [5.8.2.7]**:

$$v_u := \frac{V_{u\_crit}}{\phi_v \cdot b_v \cdot d_v}$$ $\boxed{v_u = 0.824}$ ksi

$\boxed{0.125 \cdot f'_c = 0.625}$

$$s_{max1} := \begin{cases} \min(0.8 \cdot d_v, 24) & \text{if } v_u < 0.125 \cdot f'_c \\ \min(0.4 \cdot d_v, 12) & \text{if } v_u \geq 0.125 \cdot f'_c \end{cases}$$ $\boxed{s_{max1} = 7.16}$ in

Check Minimum Reinforcing, **LRFD [5.8.2.5]**:

$$s_{max2} := \frac{A_v \cdot f_y}{0.0316 \cdot \sqrt{f'_c} \cdot b_v}$$ $\boxed{s_{max2} = 33.97}$ in

$$s_{max} := \min(s_{max1}, s_{max2})$$ $\boxed{s_{max} = 7.16}$ in

Therefore use a maximum spacing of $s := 7$ inches.

$$V_s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s}$$ $\boxed{V_s = 110.4}$ kips

Check V_n requirements:

$Vn_1 := V_c + V_s + V_p$ $\boxed{Vn_1 = 166}$ kips

$Vn_2 := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$ $\boxed{Vn_2 = 224}$ kips

$V_n := \min(Vn_1, Vn_2)$ $\boxed{V_n = 166}$ kips

$V_r := \phi_v \cdot V_n$ $\boxed{V_r = 149.81}$ kips

$\boxed{V_{u\_crit} = 132.85}$ kips

Is V_{u_crit} less than V_r? $\boxed{check = \text{"OK"}}$

Web reinforcing is required in accordance with **LRFD [5.8.2.4]** 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: $\boxed{V_{u\_crit} = 133}$ $\boxed{0.5 \cdot \phi_v \cdot (V_c + V_p) = 25}$

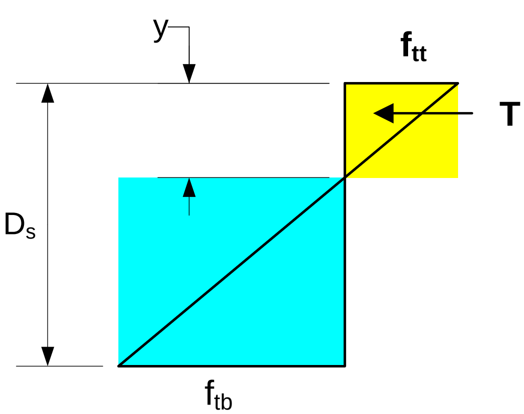
Therefore, use web reinforcing over the entire beam.

<u>Resulting Shear Design:</u>

Use #4 U shaped stirrups at 7-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.

E19-3.13 Non-Prestressed Reinforcement (Required near top of girder)

The following method is used to calculate the non-prestressed reinforcement in the top flange at the end of the girder. **LRFD [T-5.9.4.1.2-1]**



$\boxed{f_{tt} = -0.303}$ ksi

$\boxed{f_{bt} = 1.896}$ ksi

$\boxed{D_s = 1.75}$ feet

$\boxed{b = 48}$ inches

$Y := \frac{f_{tt} \cdot D_s \cdot 12}{f_{tt} - f_{bt}}$ $\boxed{Y = 2.898}$ inches

$T := \frac{|f_{tt}| \cdot b \cdot Y}{2}$ $\boxed{T = 21.101}$ kips

$\boxed{f_y = 60}$

$A_{reqd} := \frac{T}{0.5 \cdot f_y}$ $\boxed{A_{reqd} = 0.703}$ in^2

Therefore, use standard reinforcement; 5 #4 bars, As = 5*0.20 = 1.00 in^2

E19-3.14 Longitudinal Tension Flange Capacity:

The total capacity of the tension reinforcing must meet the requirements of **LRFD [5.8.3.5].** The capacity is checked at the critical section for shear:

$T_{ps} := \frac{M_{max}}{d_v \cdot \phi_f} + \left(\left| \frac{V_{u\_crit}}{\phi_v} - V_{p\_cw} \right| - 0.5 \cdot V_s \right) \cdot \cot\theta$ $\boxed{T_{ps} = 241}$ kips

actual capacity of the straight strands:

$\boxed{N \cdot A_s \cdot f_{pu\_crit} = 354}$ kips

Is the capacity of the straight strands greater than T_{ps}? $\boxed{check = "OK"}$

Check the tension capacity at the edge of the bearing:

The strand is anchored $l_{px} := 8$ inches. The transfer and development lengths for a prestressing strand are calculated in accordance with **LRFD [5.11.4.2]**:

$\boxed{l_{tr} = 30.00}$ in

$\boxed{l_d = 70.0}$ 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

$Y_s := |y_b - e_s|$ $\boxed{Y_s = 2.25}$ in

$l_{px'} := l_{px} + Y_s \cdot \cot\theta$ $\boxed{l_{px'} = 12.05}$ in

$$f_{pb} := \frac{f_{pe} \cdot l_{px'}}{60 \cdot d_s}$$ $\boxed{f_{pb} = 69.07}$ ksi

Tendon capacity of the straight strands: $\boxed{N \cdot A_s \cdot f_{pb} = 169}$ 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:

$$s_{ave} := s$$ $\boxed{s_{ave} = 7.00}$ in

$$V_s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s_{ave}}$$ $\boxed{V_s = 110}$ kips

The vertical component of the draped strands is: $\boxed{V_{p\_cw} = 0}$ kips

The factored shear force at the critical section is: $\boxed{V_{u\_crit} = 133}$ kips

Minimum capacity required at the front of the bearing:

$$T_{breqd} := \left(\frac{V_{u\_crit}}{\phi_v} - 0.5 \cdot V_s - V_{p\_cw}\right) \cdot \cot\theta$$ $\boxed{T_{breqd} = 166}$ kips

Is the capacity of the straight strands greater than T_{breqd}? $\boxed{check = \text{"OK"}}$

E19-3.15 Live Load 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.

$$\Delta_{limit} := \frac{L \cdot 12}{800}$$ $\boxed{\Delta_{limit} = 0.660}$ inches

$\boxed{I = 32942}$ in^4

$\boxed{n_{beams} = 8}$

$$I_{bridge} := I \cdot n_{beams}$$ $\boxed{I_{bridge} = 263536}$ in^4

From CBA analysis with 2 lanes loaded, the truck deflection controlled:

$\Delta_{truck} := 0.347$ in

Applying the multiple presence factor from **LRFD Table [3.6.1.1.2-1]** for 2 lanes loaded:

$\Delta := 1.0 \cdot \Delta_{truck}$ $\boxed{\Delta = 0.347}$ in

Is the actual deflection less than the allowable limit, $\Delta < \Delta$ limit? $\boxed{\text{check} = \text{"OK"}}$

E19-3.16 Camber Calculations

Moment due to straight strands:

Number of straight strands: $\boxed{N = 16}$

Eccentricity of the straight strands: $\boxed{e_s = -8.25}$ in

$P_{i\_s} := N \cdot A_s \cdot \left(f_{tr} - \Delta f_{pES}\right)$ $\boxed{P_{i\_s} = 474}$ kips

$M_1 := P_{i\_s} \cdot \left|e_s\right|$ $\boxed{M_1 = 3908}$ kip-in

Upward deflection due to straight strands:

Length of the girder: $\boxed{L_g = 45}$ ft

Modulus of Elasticity of the girder at release: $\boxed{E_{ct} = 3952}$ ksi

Moment of inertia of the girder: $\boxed{I = 32942}$ in^4

$$\Delta_s := \frac{M_1 \cdot L_g^{\,2}}{8 \cdot E_{ct} \cdot I} \cdot 12^2 \qquad \boxed{\Delta_s = 1.07} \text{ in}$$

Total upward deflection due to prestress:

$\Delta_{PS} := \Delta_s$ $\boxed{\Delta_{PS} = 1.07}$ in

Downward deflection due to beam self weight at release:

$$\Delta_{gi} := \frac{5 \cdot \left(w_g + w_d\right) \cdot L_g^{\,4}}{384 \cdot E_{ct} \cdot I} \cdot 12^3 \qquad \boxed{\Delta_{gi} = 0.44} \text{ in}$$

Anticipated prestress camber at release:

$\Delta_i := \Delta_{PS} - \Delta_{gi}$ $\boxed{\Delta_i = 0.63}$ in

The downward deflection due to the dead load of the joint grout, overlay, railing and future wearing surface:

Calculate the additional non-composite dead loads for an exterior girder:

$w_{nc} := w_{j\_ext} + w_{o\_ext} + w_{r\_ext} + w_{fws\_ext}$ $\boxed{w_{nc} = 0.241}$ klf

Modulus of Elasticity of the beam at final strength $\boxed{E_B = 5021}$ ksi

$\Delta_{nc} := \frac{5 \cdot w_{nc} \cdot L^4}{384 \cdot E_B \cdot I} \cdot 12^3$ $\boxed{\Delta_{nc} = 0.123}$ in

The residual camber for an exterior girder:

$RC := \Delta_i - \Delta_{nc}$ $\boxed{RC = 0.507}$ in

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Table of Contents

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:

WisDOT uses an installation temperature of 60°F for designing bearings. The temperature range considered for prestressed concrete girder superstructures is 5°F to 85°F, resulting in a range of 60° - 5° = 55° for bearing design. For prestressed girders an additional shrinkage factor of 0.0003 ft/ft should also be accounted for. The temperature range considered for steel girder superstructures is -30°F to 120°F, resulting in a range of 60° - (-30°) = 90° for bearing design.

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.

27.2 Bearing Types

Bridge bearings are of two general types: expansion and fixed. Bearings can be fixed in both the longitudinal and transverse directions, fixed in one direction and expansion in the other, or expansion in both directions. Expansion bearings provide for rotational movements of the girders, as well as longitudinal movement for the expansion and contraction of the bridge spans. If an expansion bearing develops a large resistance to longitudinal movement due to corrosion or other causes, this frictional force opposes the natural expansion or contraction of the span, creating a force within the span that could lead to a maintenance problem in the future. Fixed bearings act as hinges by permitting rotational movement, while at the same time preventing longitudinal movement. The function of the fixed bearing is to prevent the superstructure from moving longitudinally off of the substructure units. Both expansion and fixed bearings transfer lateral forces, as described in **LRFD [Section 3]**, from the superstructure to the substructure units. Both bearing types are set parallel to the direction of structural movement; bearings are not set parallel to flared girders.

When deciding which bearings will be fixed and which will be expansion on a bridge, several guidelines are commonly considered:

- The bearing layout for a bridge must be developed as a consistent system. Vertical movements are resisted by all bearings, longitudinal horizontal movements are resisted by fixed bearings and facilitated in expansion bearings, and rotations are generally allowed to occur as freely as possible.

- For maintenance purposes, it is generally desirable to minimize the number of deck joints on a bridge, which can in turn affect the bearing layout.

- The bearing layout must facilitate the anticipated thermal movements, primarily in the longitudinal direction, but also in the transverse direction for wide bridges.

- It is generally desirable for the superstructure to expand in the uphill direction, wherever possible.

- If more than one substructure unit is fixed within a single superstructure unit, then forces will be induced into the fixed substructure units and must be considered during design. If only one pier is fixed, unbalanced friction forces from expansion bearings will induce force into the fixed pier.

- For curved bridges, the bearing layout can induce additional stresses into the superstructure, which must be considered during design.

- Forces are distributed to the bearings based on the superstructure analysis.

A valuable tool for selecting bearing types is presented in **LRFD [Table 14.6.2-1]**, in which the suitability of various bearing types is presented in terms of movement, rotation and resistance to loads. In general, it is best to use a fixed or semi-expansion bearing utilizing an unreinforced elastomeric bearing pad whenever possible, provided adverse effects such as excessive force transfer to the substructure does not occur. Where a fixed bearing is required with greater rotational capacity, steel fixed bearings can be utilized. Laminated elastomeric bearings are the preferred choice for expansion bearings. When such expansion bearings fail to meet project requirements, steel Type “A-T” expansion bearings should be used. For curved and/or highly skewed bridges, consideration should be given to the use of pot bearings.

27.2.1 Elastomeric Bearings

Elastomeric bearings are commonly used on small to moderate sized bridges. Elastomeric bearings are either fabricated as plain bearing pads (consisting of elastomer only) or as laminated (steel reinforced) bearings (consisting of alternate layers of steel reinforcement and elastomer bonded together during vulcanization). A sample plain elastomeric bearing pad is illustrated in Figure 27.2-1, and a sample laminated (steel reinforced) elastomeric bearing is illustrated in Figure 27.2-2.

These bearings are designed to transmit loads and accommodate movements between a bridge and its supporting structure. Plain elastomeric bearing pads can be used for small bridges, in which the vertical loads, translations and rotations are relatively small. Laminated (steel reinforced) elastomeric bearing pads are often used for larger bridges with more sizable vertical loads, translations and rotations. Performance information indicates that elastomeric bearings are functional and reliable when designed within the structural limits of the material. See **LRFD [Section 14]** and *AASHTO LRFD Bridge Construction Specifications*, 3rd Edition, 2010, Section 18 and *AASHTO M251* for design and construction requirements of elastomeric bearings.

WisDOT policy item:

WisDOT currently uses plain or laminated (steel reinforced) elastomeric bearings which are rectangular in shape. No other shapes or configurations are used for elastomeric bearings in Wisconsin.

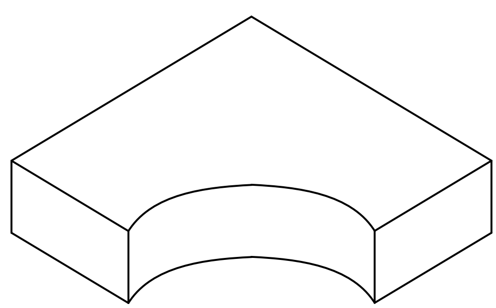

Figure 27.2-1
Plain Elastomeric Bearing

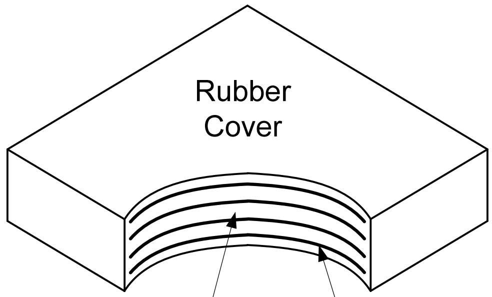

Figure 27.2-2
Laminated (Steel Reinforced) Elastomeric Bearing

AASHTO LRFD does not permit tapered elastomer layers in reinforced bearings. Laminated (steel reinforced) bearings must be placed on a level surface; otherwise gravity loads will produce shear strain in the bearing due to inclined forces. The angle between the alignment of the underside of the girder (due to the slope of the grade line, camber and dead load rotation) and a horizontal line must not exceed 0.01 radians, as per **LRFD [14.8.2]**. If the angle is greater than 0.01 radians or if the rotation multiplied by the top plate length is 1/8" or more, the 1 1/2" top steel plate must be tapered to provide a level load surface along the bottom of this plate under these conditions. The tapered plate will have a minimum thickness of 1 1/2" per *AASHTO Construction Specifications, Section 18*.

Plain and laminated (steel reinforced) elastomeric bearings can be designed by Method A as outlined in **LRFD [14.7.6]** and NCHRP-248 or by Method B as shown in **LRFD [14.7.5]** and NCHRP-298.

WisDOT policy item:

WisDOT uses Method A, as described in **LRFD [14.7.6]**, for elastomeric bearing design.

Method A results in a bearing with a lower capacity than a bearing designed using Method B. However, the increased capacity resulting from the use of Method B requires additional testing and quality control.

A preliminary value for bearing height, H, based on expansion length can be found in the Standard for Elastomeric Bearings for Prestressed Concrete Girders. The corresponding bearing length, L, based on stability requirements can also be found there. The bearing width, W, is then chosen (bottom flange width minus 2" for non-wide flanged prestressed concrete girders; bottom flange width minus 6" for wide flanged prestressed concrete girders; bottom flange width for steel girders) and is checked against stability requirements. Using these values for H, L and W, the *AASHTO LRFD* requirements for compressive stress, compressive deflection, shear deformation, steel reinforcement thickness and anchorage can be checked and the preliminary values adjusted as required.

The design of an elastomeric bearing generally involves the following steps:

1. Obtain required design input **LRFD [14.4 & 14.6]**
2. Select a feasible bearing type – plain or laminated (steel reinforced)
3. Select preliminary bearing properties **LRFD [14.7.6.2]**
4. Check shear deformation **LRFD [14.7.6.3.4]**
5. Check compressive stress **LRFD [14.7.6.3.2]**
6. Check stability **LRFD [14.7.6.3.6]**
7. Check compressive deflection **LRFD [14.7.5.3.6, 14.7.6.3.3]**
8. Check anchorage

WisDOT exception to AASHTO:

Design anchorage for laminated elastomeric bearings if the unfactored dead load stress is less than 200 psi. This is an exception to **LRFD [14.8.3]** based on past practice and good performance of existing bearings.

9. Check reinforcement **LRFD [14.7.5.3.5, 14.7.6.3.7]**
10. Rotation **LRFD [14.7.6.3.5]**

The required design input for the design of an elastomeric bearing at the service limit state is dead load, live load plus dynamic load allowance, minimum vertical force due to permanent load, and design translation. The required design input at the strength limit state is shear force. Other required design input is expansion length, girder or beam bottom flange width, minimum grade of elastomer and temperature zone.

The preliminary bearing properties can be obtained from **LRFD [14.7.6.2]** or from past experience. The preliminary bearing properties include elastomer cover thickness, elastomer internal layer thickness, elastomer hardness, elastomer shear modulus, elastomer creep deflection, pad length, pad width, number of steel reinforcement layers, steel reinforcement thickness, steel reinforcement yield strength and steel reinforcement constant-amplitude fatigue threshold. WisDOT uses the following properties:

- Elastomer cover thickness = 1/4"
- Elastomer internal layer thickness = 1/2"
- Elastomer hardness: Durometer 60 +/- 5
- Elastomer shear modulus (G): 0.1125 ksi < G < 0.165 ksi

- Elastomer creep deflection @ 25 years divided by instantaneous deflection = 0.30
- Steel reinforcement thickness = 1/8"
- Steel reinforcement yield strength = 36 ksi or 50 ksi
- Steel reinforcement constant-amplitude fatigue threshold = 24 ksi

However, not all of these properties are needed for a plain elastomeric bearing design.

Shear deformation, Δ_S, is the sum of deformation from thermal effects, Δ_{ST}, as well as creep and shrinkage effects, $\Delta_{Scr/sh}$. ($\Delta_S = \Delta_{ST} + \Delta_{Scr/sh}$)

$$\Delta_{ST} = (\text{Expansion length})(\Delta_T)(\alpha)$$

Where:

Δ_T	=	Change in temperature (see 27.1) (degrees)
α	=	Coefficient of thermal expansion (6 x 10^{-6} / °F for concrete and 6.5 x 10^{-6} / °F for steel)

Shear deformation due to creep and shrinkage effects, $\Delta_{Scr/sh}$, should be added to Δ_{ST} for prestressed concrete girder structures. The value of $\Delta_{Scr/sh}$ is computed as follows:

$$\Delta_{Scr/sh} = (\text{Expansion length})(0.0003\ \text{ft/ft})$$

LRFD [14.7.6.3.4] provides shear deformation limits to help prevent rollover at the edges and delamination. The shear deformation, Δ_S, can be checked as specified in **LRFD [14.7.6.3.4]** and by the following equation:

$$h_{rt} \geq 2\,\Delta_S$$

Where:

h_{rt}	=	Smaller of total elastomer or bearing thickness (inches)
Δ_S	=	Maximum total shear deformation of the bearing at the service limit state (inches)

The compressive stress, σ_S, at the service limit state can be checked as specified in **LRFD [14.7.6.3.2]** and by the following equations:

$\sigma_s \leq 0.80\ ksi$ and $\sigma_s \leq 1.00GS$ for plain elastomeric pads

$\sigma_s \leq 1.25\,\text{ksi}$ and $\sigma_s \leq 1.25GS$ for laminated (steel reinforced) elastomeric pads

Where:

σ_s	=	Service average compressive stress due to total load (ksi)
G	=	Shear modulus of the elastomer (ksi)
S	=	Shape factor for the thickest layer of the bearing

LRFD [14.7.6.3.2] states that the stress limits may be increased by 10 percent where shear deformation is prevented, but this is not considered applicable to WisDOT bearings.

The shape factor for individual elastomer layers is the plan area divided by the area of the perimeter free to bulge. For laminated (steel reinforced) elastomeric bearings, the following requirements must be satisfied before calculating the shape factor:

- All internal layers of elastomer must be the same thickness.
- The thickness of the cover layers cannot exceed 70 percent of the thickness of the internal layers.

The shape factor, S_i, for rectangular bearings without holes can be determined as specified in **LRFD [14.7.5.1]** and by the following equation:

$$S_i = \frac{LW}{2h_{ri}(L+W)}$$

Where:

S_i	=	Shape factor for the i^{th} layer
h_{ri}	=	Thickness of i^{th} elastomeric layer in elastomeric bearing (inches)
L	=	Length of a rectangular elastomeric bearing (parallel to longitudinal bridge axis) (inches)
W	=	Width of the bearing in the transverse direction (inches)

For stability, the total thickness of the rectangular pad must not exceed one-third of the pad length or one-third of the pad width as specified in **LRFD [14.7.6.3.6]**, or expressed mathematically:

$$H \leq \frac{L}{3} \text{ and } H \leq \frac{W}{3}$$

Where:

H	=	Total thickness of the elastomeric bearing (excluding top plate) (inches)
L	=	Length of a rectangular elastomeric bearing (parallel to longitudinal bridge axis) (inches)
W	=	Width of the bearing in the transverse direction (inches)

The compressive deflection, δ, of the bearing shall be limited to ensure the serviceability of the deck joints, seals and other components of the bridge. Deflections of elastomeric bearings due to total load and to live load alone should be considered separately. Relative deflections across joints must be restricted so that a step doesn't occur at a deck joint. **LRFD [C14.7.5.3.6]** recommends that a maximum relative live load deflection across a joint be limited to 1/8".

WisDOT policy item:

WisDOT uses a live load + creep deflection limit of 1/8" for elastomeric bearing design.

Laminated (steel reinforced) elastomeric bearings have a nonlinear load deflection curve in compression. In the absence of information specific to the particular elastomer to be used, **LRFD [Figure C14.7.6.3.3-1]** may be used as a guide. Creep effects should be determined from information specific to the elastomeric compound used. Use the material properties given in this section. The compressive deflection, δ, can be determined as specified in **LRFD [14.7.5.3.6, 14.7.6.3.3]** and by the following equation:

$$\delta = \sum \varepsilon_i \, h_{ri}$$

Where:

δ	=	Instantaneous deflection (inches)
ε_i	=	Instantaneous compressive strain in i^{th} elastomer layer of a laminated (steel reinforced) bearing
h_{ri}	=	Thickness of i^{th} elastomeric layer in a laminated (steel reinforced) bearing (inches)

Based on **LRFD [14.7.6.3.3]**, the initial compressive deflection of a plain elastomeric pad or in any layer of a laminated (steel reinforced) elastomeric bearing at the service limit state without dynamic load allowance shall not exceed $0.09h_{ri}$.

The bearing pad must be secured against horizontal movement if the service dead load stress is less than 200 psi.

The factored force due to the deformation of an elastomeric element shall be taken as specified in **LRFD [14.6.3.1]** by the following equation:

$$H_u > GA\frac{\Delta_u}{h_{rt}}$$

Where:

H_u	=	Lateral load from applicable strength load combinations in **LRFD [Table 3.4.1-1]** (kips)
G	=	Shear modulus of the elastomer (ksi)
A	=	Plan area of elastomeric element or bearing (inches^2)
Δ_u	=	Factored shear deformation (inches)
h_{rt}	=	Total elastomer thickness (inches)

Reinforcing steel plates increase compressive and rotational stiffness, while maintaining flexibility in shear. The reinforcement must have adequate capacity to handle the tensile stresses produced in the plates as they counter the lateral bulging of the elastomer layers due to compression. These tensile stresses increase with compressive load. The reinforcement thickness must also satisfy the requirements of the *AASHTO LRFD Bridge Construction Specifications*, 3rd Edition, 2010. The reinforcing steel plates can be checked as specified in **LRFD [Equation 14.7.5.3.5-1,2]**:

$$h_s \geq \frac{3\, h_{max}\, \sigma_s}{F_y} \text{ for service limit state}$$

$$h_s \geq \frac{2.0\, h_{max}\, \sigma_L}{\Delta F_{TH}} \text{ for fatigue limit state}$$

Where:

h_s	=	Thickness of the steel reinforcement (inches)
h_{max}	=	Thickness of the thickest elastomeric layer in elastomeric bearing (inches)
σ_s	=	Service average compressive stress due to total load (ksi)
F_y	=	Yield strength of steel reinforcement (ksi)
σ_L	=	Service average compressive stress due to live load (ksi)

ΔF_{TH} = Constant amplitude fatigue threshold for Category A as specified in **LRFD [6.6]** (ksi)

If holes exist in the reinforcement, the minimum thickness shall be increased by a factor equal to twice the gross width divided by the net width.

WisDOT exception to AASHTO:

Lateral rotation about the longitudinal axis of the bearing shall not be considered for straight girders.

WisDOT policy item:

Per **LRFD [14.8.2]**, a tapered plate shall be used if the inclination of the underside of the girder to the horizontal exceeds 0.01 radians. Additionally, if the rotation multiplied by the plate length is 1/8 inch or more, taper the plate.

For several years, plain elastomeric bearing pads have performed well on prestressed concrete girder structures. Refer to the Standard for Bearing Pad Details for Prestressed Concrete Girders for details. Prestressed concrete girders using this detail are fixed into the concrete diaphragms at the supports, and the girders are set on 1/2" thick plain elastomeric bearing pads. Laminated (steel reinforced) bearing details and steel plate and elastomer thicknesses are given on the Standard for Elastomeric Bearings for Prestressed Concrete Girders.

27.2.2 Steel Bearings

For fixed bearings, a rocker plate attached to the girder is set on a masonry plate which transfers the girder reaction to the substructure unit. The masonry plate is attached to the substructure unit with anchor bolts. Pintles set into the masonry plate prevent the rocker from sliding off the masonry plate while allowing rotation to occur. This bearing is represented on the Standard for Fixed Bearing Details Type "A" - Steel Girders.

For expansion bearings, two additional plates are utilized, a stainless steel top plate and a Teflon plate allowing expansion and contraction to occur, but not in the transverse direction. This bearing is shown on the Standard for Stainless Steel - TFE Expansion Bearing Details Type "A-T".

Type "B" rocker bearings have been used for reactions greater than 400 kips and having a requirement for smaller longitudinal forces on the substructure unit. However, in the future, WisDOT plans to eliminate rocker bearings for new bridges and utilize pot bearings.

Pot bearings are commonly used for moderate to large bridges. They are generally used for applications requiring a multi-directional rotational capacity and a medium to large range of load.

Hold down devices are additional details added to the Type "A-T" bearings for situations where live load can cause uplift at the abutment end of a girder. Ideally, proper span configurations would eliminate the need for hold down devices as they have proven to be a maintenance problem.

Since strength is not the governing criteria, anchor bolts are designed with Grade 36 steel for all steel bearings.

27.2.2.1 Type "A" Fixed Bearings

Type "A" Fixed Bearings prevent translation both transversely and longitudinally while allowing rotation in the longitudinal direction. This bearing is represented on the Standard for Fixed Bearing Details Type "A" - Steel Girders. An advantage of this bearing type is that it is very low maintenance. See 27.2.2.2 Type "A-T" Expansion Bearings for design information.

27.2.2.2 Type "A-T" Expansion Bearings

Type "A-T" Expansion bearings are designed to translate by sliding an unfilled polytetrafluoroethylene (PTFE or TFE) surface across a smooth, hard mating surface of stainless steel. Expansion bearings of Teflon are not used without provision for rotation. A rocker plate is provided to facilitate rotation due to live load deflection or change of camber. The Teflon sliding surface is bonded to a rigid back-up material capable of resisting horizontal shear and bending stresses to which the sliding surfaces may be subjected.

Design requirements for TFE bearing surfaces are given in **LRFD [14.7.2]**. Stainless steel-TFE expansion bearing details are given on the Standard for Stainless Steel – TFE Expansion Bearing Details Type "A-T."

Friction values are given in the **LRFD [14.7.2.5]**; they vary with loading and temperature. It is permissible to use 0.10 for a maximum friction value and 0.06 for a minimum value when determining unbalanced friction forces.

The design of type "A-T" bearings is relatively simple. The first consideration is the rocker plate length which is proportional to the contact stress based on a radius of 24" using Grade 50W steel. The rocker plate thickness is determined from a minimum of 1 1/2" to a maximum computed from the moment by assuming one-half the bearing reaction value (N/2) acting at a lever arm of one-fourth the width of the Teflon coated plate (W/4) over the length of the rocker plate. The Teflon coated plate is designed with a minimum width of 7" and the allowable stress as specified in **LRFD [14.7.2.4]** on the gross area; in many cases this controls the capacity of the expansion bearings as given in the Standard for Stainless Steel – TFE Expansion Bearing Details Type "A-T."

The design of the masonry plate is based on a maximum allowable bearing stress as specified in **LRFD [14.8.1]**. The masonry plate thickness is determined from the maximum bending moments about the x-or y-axis using a uniform pressure distribution.

In lieu of designing specific bearings, the designer may use Service I limit state loading, including dynamic load allowance, and Standards for Fixed Bearing Details Type "A" – Steel Girders, Stainless Steel – TFE Expansion Bearing Details Type "A-T" and Steel Bearings for Prestressed Concrete Girders to select the appropriate bearing.

27.2.2.3 Pot Bearings

Pot bearings are commonly used for moderate to large bridges. They are generally used for applications requiring a multi-directional rotational capacity (curved and/or highly skewed bridges) and a medium to large range of load. The bearing consists of a circular non-reinforced neoprene or rubber pad, of relatively thin section, which is totally enclosed by a steel pot. The rubber is prevented from bulging by the pot containing it and acts similar to a fluid under high pressure. The result is a bearing providing suitable rotation and at the same time giving the effect of a point-contact rocker bearing since the center of pressure does not vary more than 4 percent. As specified in **LRFD [14.7.4.1]**, the minimum vertical load on a pot bearing should not be less than 20 percent of the vertical design load.

Pot bearings resist vertical load primarily through compressive stress in the elastomeric pad. The pad can deform and it has some shear stiffness, but it has very limited compressibility. Pot bearings generally have a large reserve of strength against vertical load. Pot bearings facilitate rotation through deformation of the elastomeric pad. During rotation, one side of the pad compresses and the other side expands. Pot bearings can sustain many cycles of small rotations with little or no damage. However, they can experience significant damage when subjected to relatively few cycles of large rotations.

Pot bearings can also resist horizontal loads. They can either be fixed, guided or non-guided. Fixed pot bearings (see Figure 27.2-3) can not translate in any direction, and they resist horizontal load primarily through contact between the rim of the piston and the wall of the pot. Guided pot bearings (see Figure 27.2-4) can translate in only one direction, and they resist horizontal load in the other direction through the use of guide bars. Non-guided pot bearings (see Figure 27.2-5) can translate in any direction, and they do not resist horizontal loads in any direction.

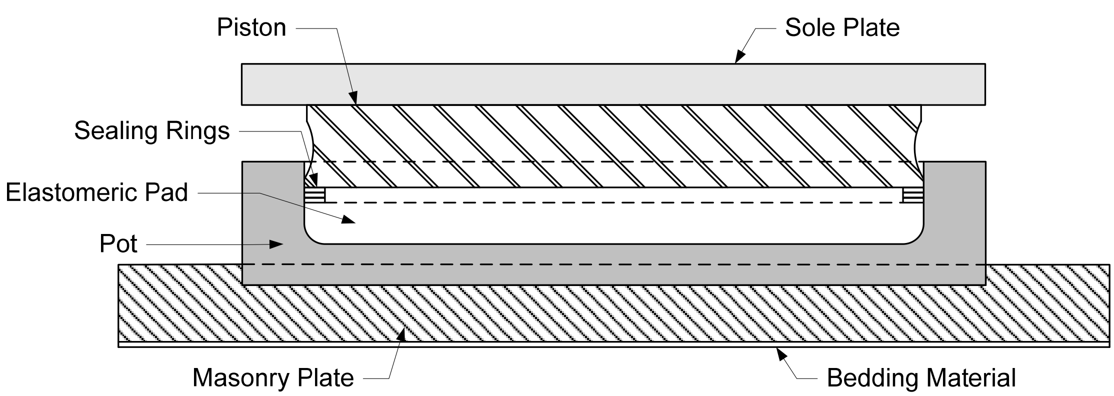

Figure 27.2-3
Fixed Pot Bearing

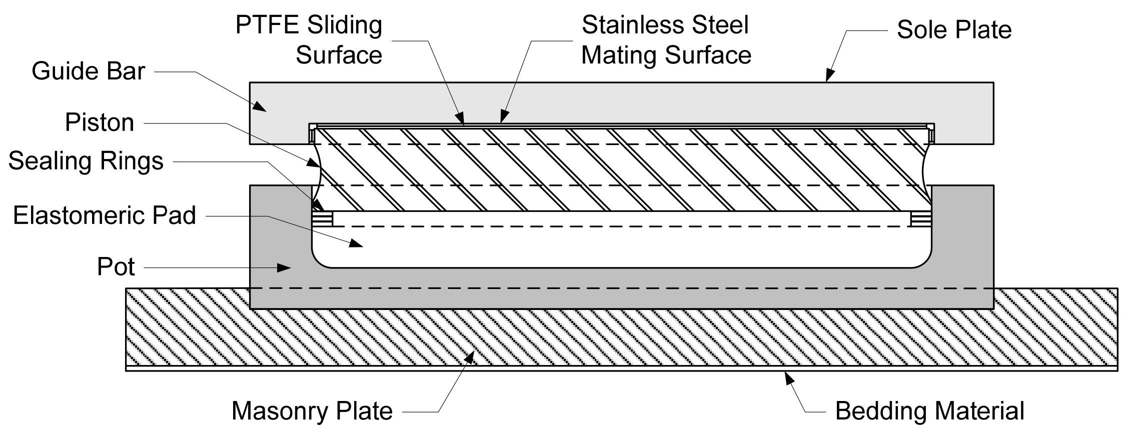

Figure 27.2-4
Guided Pot Bearing

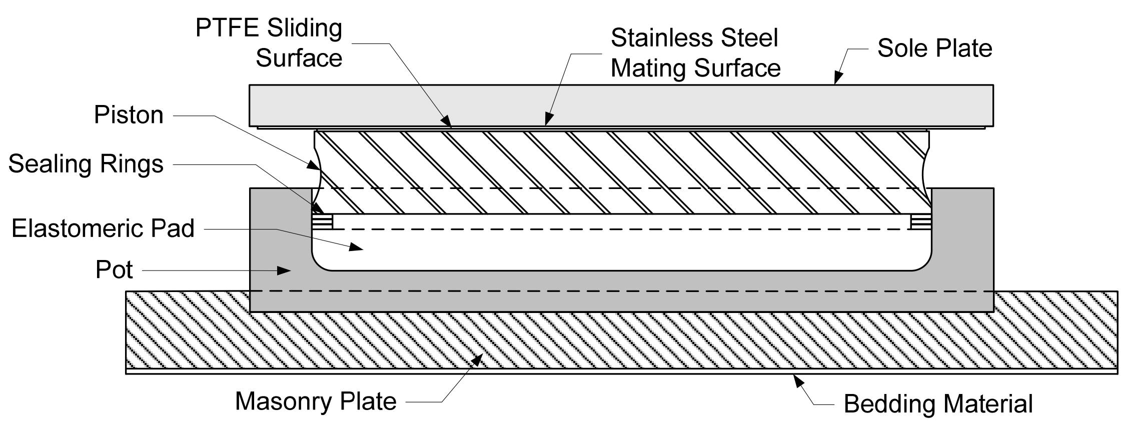

Figure 27.2-5
Non-Guided Pot Bearing

The design of a pot bearing generally involves the following steps:

1. Obtain required design input **LRFD [14.4 & 14.6]**
2. Select a feasible bearing type: fixed, guided or non-guided
3. Select preliminary bearing properties **LRFD [14.7.4.2]**
4. Design the elastomeric disc **LRFD [14.7.4.3 and 14.7.4.4]**
5. Design the sealing rings **LRFD [14.7.4.5]**

6. Design the pot **LRFD [C14.7.4.3, 14.7.4.6 and 14.7.4.7]**

7. Design the piston **LRFD [14.7.4.7]**

8. Design the guides and restraints, if applicable **LRFD [14.7.9]**

9. Design the PTFE sliding surface, if applicable **LRFD [14.7.2]**

10. Design the sole plate, masonry plate (or bearing plate), anchorage and connections **LRFD [6 and 14.8]**

11. Check the concrete or steel support **LRFD [5.7.5 and 6]**

Although the steps for pot bearing design are given above, typically the actual bearing design is done by the manufacturer. The design of the masonry plate is done either by the design engineer or by the bearing manufacturer.

When using pot bearings, the design plans need to specify the following: degree of fixity (fixed, guided in one direction or non-guided), maximum vertical load, minimum vertical load, maximum horizontal load (fixed and guided, only) and an assumed height. The loads specified are Service I limit state loads, including dynamic load allowance.

Field adjustments to the given beam seat elevations will be required if the actual bearing height differs from the assumed bearing height stated on the plan. To facilitate such an adjustment without affecting the structural integrity of the substructure unit, a concrete pedestal (plinth) is detailed at each bearing location. Detailing a pedestal height of 10” based on the assumed bearing height will give sufficient room for adjustment should the actual bearing height differ from the assumed bearing height.

27.3 Hold Down Devices

Hold down devices are additional elements added to the Type "A-T" bearings for situations where live load can cause uplift at the abutment end of a girder. Ideally, proper span configurations would eliminate the need for hold down devices as they have proven to be a maintenance problem. Details for hold down devices are given in the Standard for Hold Down Devices.

27.4 Design Examples

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Table of Contents

E27-1 DESIGN EXAMPLE - STEEL REINFORCED ELASTOMERIC BEARING

ThIs design example is for a 3-span prestressed girder structure. The piers are fixed supports and the abutments accommodate expansion.
(Example is current through LRFD Sixth Edition - 2012)

E27-1.1 Design Data

Bearing location: Abutment (Type A3)

Girder type: 72W

$L_{exp} := 220$ Expansion length, ft

$b_f := 2.5$ Bottom flange width, ft

$DL_{serv} := 167$ Service I limit state dead load, kips

$DL_{ws} := 23$ Service I limit state future wearing surface dead load, kips

$LL_{serv} := 62$ Service I limit state live load, kips

$h_{rcover} := 0.25$ Elastomer cover thickness, in

$h_s := 0.125$ Steel reinforcement thickness, in

$F_y := 36$ Minimum yield strength of the steel reinforcement, ksi

Temperature Zone:	C (Southern Wisconsin)	**LRFD [Fig. 14.7.5.2-1]**
Minimum Grade of Elastomer:	3	**LRFD [Table 14.7.5.2-1]**
Elastic Hardness:	Durometer 60 +/- 5	(used 55 for design)
Shear Modulus (G):	0.1125 ksi < G <0.165 ksi	**LRFD [Table 14.7.6.2-1]**
Creep Deflection @ 25 Years		
divided by instantaneous deflection:	0.3	**LRFD [Table 14.7.6.2-1]**

E27-1.2 Design Method

Use Design Method A **LRFD [14.7.6]**
Method A results in a bearing with a lower capacity than a bearing designed using Method B. However the increased capacity resulting from the use of Method B requires additional testing and quality control.

E27-1.3 Dynamic Load Allowance

The influence of impact need not be included for bearings **LRFD [14.4.1]**; however, dynamic load allowance will be included to follow a **WisDOT policy item**.

E27-1.4 Shear

The maximum shear deformation of the pad shall be taken as the maximum horizontal superstructure displacement, reduced to account for the pier flexibility. **LRFD [14.7.6.3.4]**

$h_{rt} \geq 2\Delta_S$ **LRFD [Equation 14.7.6.3.4-1]**

Temperature range: T_{low} and T_{high} values below are from WisDOT policy item in 27.1

$T_{low} := 5$ Minimum temperature, °F

$T_{high} := 85$ Maximum temperature, °F

$\gamma_{TU} := 1.2$ Service I Load factor for deformation **LRFD [Table 3.4.1-1]**

$T_{install} := 60$ Installation temperature, °F

$\alpha_c := 0.000006$ Coefficient of thermal expansion of concrete, ft/ft/°F

$S_{crsh} := 0.0003$ Coefficient of creep and shrinkage of concrete, ft/ft

$\Delta_T := T_{install} - T_{low}$ $\boxed{\Delta_T = 55}$ °F

Maximum total shear deformation of the elastomer

$\Delta_S := L_{exp} \cdot \alpha_c \cdot \Delta_T \cdot 12 + L_{exp} \cdot S_{crsh} \cdot 12$ $\boxed{\Delta_S = 1.663}$ in

Required total elastomer thickness

$H_{rt} \geq 2 \cdot \gamma_{TU} \cdot \Delta_S$ $\boxed{H_{rt} = 3.992}$ in

Elastomer internal layer thickness

$h_{ri} := 0.5$ in

Required elastomer thickness **LRFD [14.7.6.1)**

$$\frac{h_{rcover}}{h_{ri}} \leq 0.7 \qquad \boxed{\frac{h_{rcover}}{h_{ri}} = 0.5}$$

$\boxed{\text{check} = \text{"< 0.7, OK"}}$

Determine the number of internal elastomer layers:

$$n := \frac{H_{rt} - 2 \cdot h_{rcover}}{h_{ri}}$$

Note: $\boxed{h_{rcover} = 0.25}$ in

$\boxed{n = 6.983}$ layers

Use: $\boxed{n = 7}$ layers

Total elastomer thickness:

$h_{rt} := 2 \cdot h_{rcover} + n \cdot h_{ri}$ $\boxed{h_{rt} = 4.0}$ in

Total height of reinforced elastomeric pad:

$H := h_{rt} + (n + 1) \cdot h_s$ $\boxed{H = 5.000}$ in

E27-1.5 Compressive Stress

$\sigma_{s\_all} \le 1.25$ and $\sigma_{s\_all} \le 1.25 \cdot G \cdot S$ **LRFD [14.7.6.3.2]**

$edge := 3$ in Transverse distance from the edge of the flange to edge of bearing

$W := 12 \cdot b_f - 2 \cdot edge$ Transverse dimension $\boxed{W = 24}$ in

$L \ge \dfrac{DL_{serv} + LL_{serv}}{W \cdot \sigma_{s\_all}}$ Since $\sigma_{s\_all} \le \dfrac{DL_{serv} + LL_{serv}}{L \cdot W}$

$\sigma_{s\_all} := 1.25$ ksi

$L := \dfrac{DL_{serv} + LL_{serv}}{W \cdot \sigma_{s\_all}}$ Longitudinal dimension $\boxed{L = 7.633}$ in

$increment := 5$ in <== Rounding increment

$\boxed{L = 10}$ in

(can be used to increase L dimension to satisfy subsequent stress checks, etc.)

Determine shape factor for internal layer **LRFD [Equation 14.7.5.1-1]**

$S_i := \dfrac{L \cdot W}{2 \cdot h_{ri} \cdot (L + W)}$ $\boxed{S_i = 7.059}$

$G := 0.1125$ ksi $0.1125\text{ksi} < G < 0.165\text{ksi}$

$\boxed{1.25 \cdot G \cdot S_i = 0.993}$ ksi (Verify that **LRFD** is satisfied for a full range of G values. The minimum G values is used here. See also E27-1.8)

$\sigma_s := \dfrac{DL_{serv} + LL_{serv}}{L \cdot W}$ $\boxed{\sigma_s = 0.954}$ ksi

$\boxed{\sigma_s = \text{"< 1.25GS, OK"}}$

E27-1.6 Stability

$H \le \dfrac{L}{3}$ and $H \le \dfrac{W}{3}$ **LRFD [14.7.6.3.6]**

$\boxed{H = 5.000}$ in

Bearing length check:

$L_{min} := 3 \cdot H$ $\boxed{L_{min} = 15}$ in

$\boxed{L = 10}$ in

Use the larger value: $\boxed{L = 15}$ in

Bearing width check:

$W_{min} := 3 \cdot H$ $\boxed{W_{min} = 15}$ in

$\boxed{W = 24}$ in

Use the larger value: $\boxed{W = 24}$ in

Revised shape factor and compressive stress for internal layer:

$\boxed{h_{ri} = 0.5}$ in

$\boxed{G = 0.1125}$ ksi

$S_i := \dfrac{L \cdot W}{2 \cdot h_{ri} \cdot (L + W)}$ $\boxed{S_i = 9.231}$

$\boxed{1.25 \cdot G \cdot S_i = 1.298}$ ksi

$\sigma_S := \dfrac{DL_{serv} + LL_{serv}}{L \cdot W}$ $\boxed{\sigma_S = 0.636}$ ksi

$\boxed{\sigma_S = \text{"< 1.25GSi, OK"}}$

Check **LRFD [C14.7.6.1]**: $S_i^2 / n < 20$ (for rectangular shape with $n \ge 3$)

$S_i^2 / n = (9.231)^2 / 8 = 10.7 < 20$ "OK"

Revised shape factor and compressive stress for the cover layer:

$\boxed{h_{rcover} = 0.25}$ in

$$S_{cover} := \frac{L \cdot W}{2 \cdot h_{rcover} \cdot (L + W)}$$

$\boxed{S_{cover} = 18.462}$ ksi

$\boxed{1.25 \cdot G \cdot S_{cover} = 2.596}$ ksi

$$\sigma_S := \frac{DL_{serv} + LL_{serv}}{L \cdot W}$$

$\boxed{\sigma_S = 0.636}$ ksi

$\boxed{\sigma_S = \text{"< 1.25GS, OK"}}$

E27-1.7 Compressive Deflection

LRFD [14.7.6.3.3, 14.7.5.3.6]

Average vertical compressive stress:

Average compressive stress due to total load

$\boxed{\sigma_S = 0.636}$ ksi

Average compressive stress due to live load

$$\sigma_L := \frac{LL_{serv}}{L \cdot W}$$

$\boxed{\sigma_L = 0.172}$ ksi

Average compressive stress due to dead load

$$\sigma_D := \frac{DL_{serv}}{L \cdot W}$$

$\boxed{\sigma_D = 0.464}$ ksi

Use **LRFD [Figure C14.7.6.3.3-1]** to estimate the compressive strain in the interior and cover layers. Average the values from the 50 Durometer and 60 Durometer curves to obtain values for 55 Durometer bearings.

LAYER	LOAD	S	STRESS (ksi)	50 DUROMETER STRAIN	60 DUROMETER STRAIN	AVERAGE STRAIN
INTERNAL	DEAD LOAD	9.231	0.464	2.3%	2.1%	2.2%
	TOTAL LOAD	9.231	0.636	3.1%	2.7%	2.9%
COVER	DEAD LOAD	18.462	0.464	1.8%	1.5%	1.7%
	TOTAL LOAD	18.462	0.636	2.2%	1.9%	2.1%

Initial compressive deflection of n-internal layers and 2 cover layers under total load:

$\boxed{\varepsilon_{int} = 0.029}$ Compressive strain in the interior layer

$\boxed{\varepsilon_{cover} = 0.021}$ Compressive strain in the cover layer

$\boxed{n = 7}$ layers

$\boxed{h_{ri} = 0.5}$ in

$\boxed{h_{rcover} = 0.25}$ in

$\delta := n \cdot h_{ri} \cdot \varepsilon_{int} + 2 \cdot h_{rcover} \cdot \varepsilon_{cover}$ Modification of **LRFD [Equation 14.7.5.3.6-1]**

$\boxed{\delta = 0.112}$ in

Initial compressive deflection under dead load:

$\boxed{\varepsilon_{intDL} = 0.022}$

$\boxed{\varepsilon_{coverDL} = 0.017}$

$\delta_{DL} := n \cdot h_{ri} \cdot \varepsilon_{intDL} + 2 \cdot h_{rcover} \cdot \varepsilon_{coverDL}$ $\boxed{\delta_{DL} = 0.086}$ in

Deflection due to creep:

$C_d := 0.30$ Average value between 50 and 60 Durometer
LRFD [Table 14.7.6.2-1]

$\delta_{CR} := C_d \cdot \delta_{DL}$ $\boxed{\delta_{CR} = 0.026}$ in

Compressive deflection due to live load:

$\delta_{LL} := \delta - \delta_{DL}$ $\boxed{\delta_{LL} = 0.027}$ in

Deflection due to creep and live load: **LRFD [C14.7.5.3.6]**

$\delta_{CRLL} := \delta_{CR} + \delta_{LL}$ $\boxed{\delta_{CRLL} = 0.052}$ in

$\boxed{\delta_{CRLL} = \text{"< 0.125 in., OK"}}$

Initial compressive deflection of a single internal layer:

$\varepsilon_{int} \cdot h_{ri} < 0.09 \cdot h_{ri}$ **LRFD [14.7.6.3.3]** $\boxed{\varepsilon_{int} = 0.029}$

$\boxed{\varepsilon_{int} = \text{"< 0.09, OK"}}$

E27-1.8 Anchorage

LRFD [14.8.3]

Shear force generated in the bearing due to temperature movement:

$H_u := G \cdot A \cdot \dfrac{\Delta_u}{h_{rt}}$ **LRFD [Equation 14.6.3.1-2]**

$G := 0.165$ conservative assumption, maximum value of G, ksi

Factored shear deformation of the elastomer

$\Delta_u := \gamma_{TU} \cdot \Delta_s$ $\boxed{\Delta_u = 1.996}$ in

Plan area of elastomeric element

$\boxed{L = 15}$ in $\boxed{W = 24}$ in

$A := L \cdot W$ $\boxed{A = 360}$ in^2

$H_u := G \cdot A \cdot \dfrac{\Delta_u}{h_{rt}}$ $\boxed{H_u = 29.638}$ kips

(This value of H_u can be used for substructure design)

Minimum vertical force due to permanent loads:

$\gamma_{DLserv} := 1.0$

$P_{sd} := \gamma_{DLserv} \cdot (DL_{serv} - DL_{ws})$ $\boxed{P_{sd} = 144}$ kips

$\sigma := \dfrac{P_{sd}}{A}$ $\boxed{\sigma = 0.400}$ ksi

$\boxed{\sigma = \text{"> 0.200 ksi, OK, anchorage is not required per WisDOT exception to AASHTO"}}$

E27-1.9 Reinforcement:

LRFD [14.7.6.3.7, 14.7.5.3.5]

Service limit state:

$h_{max} := h_{ri}$ $\boxed{h_{max} = 0.5}$ in

$\boxed{\sigma_s = 0.636}$ ksi

$\boxed{F_y = 36}$ ksi

$h_s \geq \dfrac{3 \cdot h_{max} \cdot \sigma_s}{F_y}$ **LRFD [Eq 14.7.5.3.5-1]** $\boxed{h_s = 0.125}$ in

$\boxed{\dfrac{3 \cdot h_{max} \cdot \sigma_s}{F_y} = 0.027}$ in

$\boxed{\text{check} = \text{"< hs,   OK"}}$

Fatigue limit state:

$h_s \geq \dfrac{2 \cdot h_{max} \cdot \sigma_L}{\Delta F_{TH}}$ **LRFD [Eq 14.7.5.3.5-2]** $\boxed{\sigma_L = 0.172}$ ksi

$\boxed{h_s = 0.125}$ in

$\Delta F_{TH} := 24.0$ ksi Constant amplitude fatigue threshold for Category A **LRFD [Table 6.6.1.2.5-3]**

$\boxed{\dfrac{2 \cdot h_{max} \cdot \sigma_L}{\Delta F_{TH}} = 0.007}$ in

$\boxed{\text{check} = \text{"< hs,   OK"}}$

E27-1.10 Rotation:

LRFD [14.7.6.3.5, C14.7.6.1]

Design for rotation in Method A is implicit in the geometric and stress limit requirements spelled out for this design method. Therefore no additional rotation calculations are required.

Check requirement for tapered plate: **LRFD [14.8.2]**

Find the angle between the alignment of the underside of the girder and a horizontal line. Consider the slope of the girder, camber of the girder, and rotation due to unfactored dead load deflection.

Inclination due to grade line:

$L_{span} := 150$ Span length, ft

@ pier:

$EL_{Pseat} := 856.63$ Beam seat elevation at the pier, in feet

$h_{Pbrg} := 0.5$ Bearing height at the pier, in

Bottom of girder elevation at the pier, in feet

$$EL_1 := EL_{Pseat} + \frac{h_{Pbrg}}{12} \qquad \boxed{EL_1 = 856.672}$$

@ abutment:

$EL_{Aseat} := 853.63$ Beam seat elevation at the abutment, in feet

$t_{plate} := 1.5$ Steel top plate thickness, in

$\boxed{H = 5}$ Total elastomeric bearing height, in

Total bearing height, at the abutment, in

$$h_{Abrg} := H + t_{plate} \qquad \boxed{h_{Abrg} = 6.5} \text{ in}$$

Bottom of girder elevation in feet

$$EL_2 := EL_{Aseat} + \frac{h_{Abrg}}{12} \qquad \boxed{EL_2 = 854.172}$$

Slope of girder

$$S_{GL} := \frac{\left|EL_1 - EL_2\right|}{L_{span}} \qquad \boxed{S_{GL} = 0.017}$$

Inclination due to grade line in radians

$$\theta_{GL} := \operatorname{atan}\left(S_{GL}\right) \qquad \boxed{\theta_{GL} = 0.017} \text{ radians}$$

Inclination due to residual camber:

$\Delta_{camber} := 3.83$ Maximum camber of girder, in

$\Delta_{DL} := 2.54$ Maximum dead load deflection, in

$\Delta_{LL} := 0.663$ Maximum live load deflection, in

Residual camber, in

$\Delta_{RC} := \Delta_{camber} - \Delta_{DL}$ $\boxed{\Delta_{RC} = 1.290}$ in

To determine the slope due to residual camber, use a straight line from C/L Bearing to the 1/10 point. Assume that camber at 1/10 point is 40% of maximum camber (at midspan).

$S_{RC} := \dfrac{0.4 \cdot \Delta_{RC}}{0.1 \cdot L_{span} \cdot 12}$ Slope due to residual camber $\boxed{S_{RC} = 0.003}$

Inclination due to residual camber in radians

$\theta_{RC} := \operatorname{atan}(S_{RC})$ $\boxed{\theta_{RC} = 0.003}$ radians

Total inclination due to grade line and residual camber in radians

$\theta_{SX} := \theta_{GL} + \theta_{RC}$ $\boxed{\theta_{SX} = 0.020}$ radians

$\boxed{\theta_{SX} = \text{"> 0.01 rad, NG, top plate must be tapered"}}$

(The plate should also be tapered if θ_{SX} x L_P >= 1/8")

Top plate dimensions:

$\boxed{t_{plate} = 1.5}$ Minimum thickness of top plate, in

$L_p := L + 2$ Length of top plate

$\boxed{L_p = 17}$ in

$\boxed{L_p \cdot \theta_{SX} = 0.332}$ in

Thickness of top plate on thicker edge

$t_{pmax} := t_{plate} + L_p \cdot \tan(\theta_{SX})$ $\boxed{t_{pmax} = 1.832}$ in

E27-1.11 Bearing summary:

Laminated Elastomeric Bearing Pad:

Length = 15 inches

Width = 24 inches

Steel reinforcing plates: 8 @ 1/8"

Internal elastomer layers: 7 @ 1/2"

Cover elastomer layers: 2 @ 1/4"

Total pad height: 5"

<u>Steel Top Plate (See standard detail):</u>

Length = 17 inches

Width = 30 inches

Thickness = 1 1/2" to 1 7/8"

Table of Contents

30.1 Crash Tested Bridge Railings

All bridge railings must have passed the crash tests as recommended in the NCHRP report 350 for Bridge Railings. In order to use railings other than Bridge Office Standard railing details, the railings must conform to crash tested rails which are available from the FHWA office. Any railings that are not crash tested must be reviewed by FHWA when they are used on bridge, culvert, retaining wall, etc.

Railings must meet the criteria for TL-3 or greater to be used on all roadways. Railings meeting TL-2 criteria may be used on roadways where the speed is 45 mph or less.

13. Chain Link Fence and Ornamental Protective Screening, as shown in the Standards, may be attached to the top of concrete parapets (or directly to the deck if on a sidewalk separated from the roadway by a crashworthy barrier). Ornamental Protective Screening should only be used when the design speed is 45 mph or less, or the screening is protected by a barrier between the roadway and sidewalk. Chain Link Fence can be used for any design speed.

See the *Facilities Development Manual 11-40-1* for additional railing application requirements. See 11-35-1, Table 1.1 for requirements on when barrier wall separation between roadway and sidewalk is required.

30.3 Design Details

1. 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 at its midspan and made continuous with a movable internal sleeve. On conventional structures where expansion joints are likely to occur at the abutments only, if tubular railing is employed, the posts may be placed at equal increments providing that no post is nearer than 2 feet from deflection joints in the parapet at the piers.

2. Epoxy coated bars are required for all concrete parapets, curbs, medians, and sidewalks.

3. Refer to Standard Detail, Vertical Face Parapet “A” for detailing concrete parapet or median deflection joints. These joints are used because of previous experience with transverse deck cracking beneath the parapet joints.

4. Horizontal cracking occurred 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 should not be allowed.

5. Detail erection joints at the one-sixth panel point for Type "F" railing. This location will insure primarily a shear transfer at the railing splices. For beam guard type railing, locate the expansion splice at a post or on either side of the expansion joint.

6. On skewed bridges where the length from the rail post to the first guard rail posts exceeds 3 feet, employ the following detail: Extend the railing to the back face of the abutment. Bolt a plate to the back of the rails right before the rail bend. In case of vehicle impact, this detail will cause the rails to act as a unit in preventing vehicle wheel snagging.

7. Note the AASHTO Specification for a maximum opening of 6 inches on lower rail elements.

8. Sidewalks - If there is a parapet between the roadway and a sidewalk and the roadway side of the parapet is more than 11’-0” from the exterior edge of deck, 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”. Access must be provided to the sidewalk for the “snooper truck” to inspect the underside of the bridge. The boom extension on most trucks does not exceed 11’-0” so provision must be made to get the truck closer to the edge.

9. 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.

30.4 Utilities

The maximum allowable conduits that can be placed in "__SS", "HF" or "LF" parapets are shown in the following sketches ("LF" only shown). Junction (Pull) boxes can only be used with 2 inch diameter conduit. The maximum length of 3 inch conduit is 190 feet, as no boxes are allowed.

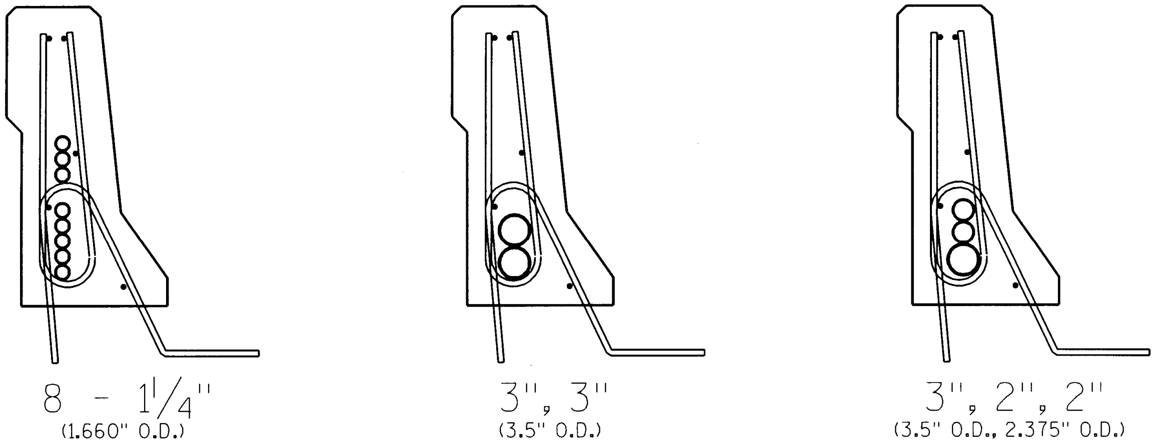

Figure 30.4-1
Maximum Allowable Conduits in "__SS", "HF" and "LF" Parapets

When light poles are mounted on top of parapets and the design speed exceeds 45 mph the light pole must be located behind the back edge of the parapet. The poles should also be placed over the piers unless there is an expansion joint. Place 4 feet away if this is the case.

FDM 9-25-5 addresses whether a bench mark disk should be set on a structure. Structures are not usually preferred due to possible elevation changes from various causes. WisDOT has discontinued the statewide practice of furnishing a disk and requiring it to be placed on a structure. WisDOT Region Offices may continue to provide a bench mark for the contract to be set. Consult the Region Office to determine if a bench mark should be included in the plan set.

30.5 Protective Screening

Protective screening is a special type fence constructed on the sides of an overpass to discourage and/or prevent people from dropping or throwing objects onto vehicles passing underneath the structure. Protective screening is generally chain link type fencing attached to steel posts mounted on top of a vertical parapet or on a sidewalk surface. The top of the protective screening may be curved inward toward the structure, if mounted on a parapet and on a sidewalk, to prevent objects from being thrown off the overpass structure. Aesthetics is enhanced by using a colored protective screening which can be coordinated with the color of the structure. See Chapter 30 and Chapter 37 standards for screening details.

Examples of situations that warrant consideration of protective screening are:

1. If there is a history of or instances of objects being dropped or thrown from an existing overpass.

2. For all new overpasses if there have been instances at other existing overpasses in the area.

3. On overpasses near a school, playground, residential area or any other location where the overpass may be used by children who are not accompanied by an adult.

In addition, all pedestrian overpasses should have protective screening on both sides.

When protective screening is warranted, the minimum design should require screening on the side of the structure with sidewalk. Designers can call for protective screening on sides without sidewalks if those sides are readily accessible to pedestrians.

Designers should insure that where protective screening is called for, it does not interfere with sight distances between the overpass and any ramps connecting it with the road below. This is especially important on cloverleaf and partial cloverleaf type interchanges.

Protective screening is not always warranted. An example of when it may not be warranted is on an overpass without sidewalks where pedestrians do not have safe or convenient access to either side because of high traffic volumes and/or the number of traffic lanes that must be crossed.

Occasionally access to light poles behind protective screening is required or the screening may need repair. To gain access, attach fence stretchers to the fencing and remove one vertical wire by threading or cutting. The vertical wire may be cut without using fence stretchers. To repair, attach fence stretchers and thread a vertical wire in place of the one removed by either reusing the one in place or using a new one.

Fence repair would follow this same process except the damaged fencing would be removed and replaced with new fencing.

See 30.3(8), for additional guidance with regards to “snooper” access and screening height.

Table of Contents

36.3.4.3.2 Depth of Fill less than 2.0 ft

Per **LRFD [5.14.5.3]**, for box culverts with less than 2.0 feet of fill follow **LRFD [5.8]** and **LRFD [5.13.3.6]**.

The shear resistance of the concrete, V_c, for slabs and walls of box culverts with less than 2.0 feet of fill, for one-way action per **LRFD [5.8.3.3]** shall be determined as:

$$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v \le 0.25 f'_c\, b_v d_v$$

With variables defined above in 36.3.4.3.1.

For box culverts where the top slab is an integral part of the wearing surface (depth of fill equal zero) the top slab shall be checked for two-way action, as discussed in 18.3.3.2.2.

36.3.5 Service Limit State

Service I Limit State shall be applied as restrictions on stress, deformation, and crack width under regular service conditions **LRFD [1.3.2.2]**.

36.3.5.1 Factored Resistance

The resistance factor, ϕ, for Service Limit State, is found in **LRFD [1.3.2.1]** and its value is 1.00.

36.3.5.2 Crack Control Criteria

Per **LRFD [12.11.3]**, the provisions of **LRFD [5.7.3.4]** shall apply to crack width control in box culverts. All reinforced concrete members are subject to cracking under any load condition, which produces tension in the gross section in excess of the cracking strength of the concrete. Provisions are provided for the distribution of tension reinforcement to control flexural cracking.

Crack control criteria does not use a factored resistance, but calculates a maximum spacing for flexure reinforcement based on service load stress in bars, concrete cover and exposure condition.

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, s, (in inches) of mild steel reinforcement in the layer closest to the tension face shall satisfy:

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \quad \text{(in.)}$$

in which:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

Where:

γ_e = Exposure factor
(1.0 for Class 1 exposure condition, 0.75 for Class 2 exposure condition, see **LRFD [5.7.3.4]** for guidance)

d_c = Thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.)

f_{ss} = Tensile stress in steel reinforcement at the service limit state (ksi)

h = Overall thickness or depth of the component (in.)

WisDOT Policy Item:

A class 1 exposure factor, γ_e = 1.0, shall be used for all cases for cast-in-place box culverts except for the top steel in the top slab of a box culvert with zero fill, where a class 2 exposure factor, γ_e = 0.75, shall be used.

36.3.6 Minimum Reinforcement Check

Per **LRFD [12.11.4.3]**, the area of reinforcement, A_s, in the box culvert cross-section should be checked for minimum reinforcement requirements per **LRFD [5.7.3.3.2]**.

The area 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) $1.33M_u$

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

Where:

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

f_r = $0.37\sqrt{f'_c}$ Modulus of rupture (ksi) **LRFD [5.4.2.6]**

I_g = Gross moment of inertia (in^4)

c = ½ *effective slab thickness (in.)

M_u = Total factored moment using Strength I Limit State (kip-in)

M_{cr} = Cracking strength moment (kip-in)

The factored resistance, M_r or moment capacity, shall be calculated as in 36.3.4.2 and shall satisfy:

$M_r \geq \min(M_{cr} \text{ or } 1.33\ M_u)$

36.3.7 Minimum Spacing of Reinforcement

Per **LRFD [5.10.3.1]**, the clear distance between parallel bars in a layer shall not be less than:

- 1.5 times the nominal diameter of the bars
- 1.5 times the maximum size of the course aggregate
- 1.5 inches

36.3.8 Maximum Spacing of Reinforcement

Per **LRFD [5.10.3.2]**, the spacing of reinforcement in walls and slabs shall not exceed:

- 1.5 times the thickness of the member (3.0 times for temperature and shrinkage)
- 18 inches

36.3.9 Edge Beams

Per **LRFD [12.11.2.1]**, for cast-in-place box culverts, and for precast box culverts with top slabs having span to thickness ratios (s/t) > 18 or segment lengths < 4.0 feet, edge beams shall be provided as specified in **LRFD [4.6.2.1.4]** as follows:

- At ends of culvert runs where wheel loads travel within 24.0 inches from the end of the culvert
- At expansion joints of cast-in-place culverts where wheel loads travel over or adjacent to the expansion joint

The edge beam provisions are only applicable for culverts with less than 2.0 ft of fill, **LRFD [C12.11.2.1]**.

36.4 Design Loads

36.4.1 Self Weight (DC)

Include the structure self weight based on a unit weight of concrete of 0.150 kcf. When there is no fill on the top slab of the culvert, the top slab thickness includes a ½" wearing surface. The weight of the wearing surface is included in the design, but its thickness is not included in the section properties of the top slab. When designing the bottom slab of a culvert do not forget that the weight of the concrete in the bottom slab acts in an opposite direction than the bottom soil pressure and thus reduces the design moments and shears. This load is designated as, DC, dead load of structural components and nonstructural attachments, for application of load factors and limit state combinations.

36.4.2 Future Wearing Surface (DW)

If the fill depth over the culvert is greater than zero, the weight of the future wearing surface shall be taken as zero. If there is no fill depth over the culvert, the weight of the future wearing surface shall be taken as 20 psf. This load is designated as, DW, dead load of wearing surfaces and utilities, for application of load factors and limit state combinations.

36.4.3 Vertical and Horizontal Earth Pressure (EH and EV)

WisDOT Policy Item:

Box Culverts are assumed to be rigid frames. Use Vertical Earth Pressure load factors for rigid frames, in accordance with **LRFD [Table 3.4.1-2]**.

Use Horizontal Earth Pressure load factors for active soil pressure, in accordance with **LRFD [Table 3.4.1-2]**. Using load factors for active soil pressure is a conservative assumption.

The weight of soil above the buried structure is taken as 0.120 kcf. A coefficient of lateral earth pressure of 0.5 is used for the lateral pressure from the soil. This coefficient of lateral earth pressure is based on an at-rest condition and an effective friction angle of 30º, **LRFD [3.11.5.2]**. The lateral earth pressure is calculated per **LRFD [3.11.5.1]**:

$$p = k_o \gamma_s z$$

Where:

p	=	Lateral earth pressure (ksf)
k_o	=	Coefficient of at-rest lateral earth pressure
γ_s	=	Unit weight of backfill (kcf)
z	=	Depth below the surface of earth (ft)

Table of Contents

E36-1 Twin Cell Box Culvert LRFD

This example shows design calculations for a twin cell box culvert. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Sixth Edition - 2012)***

E36-1.1 Design Criteria

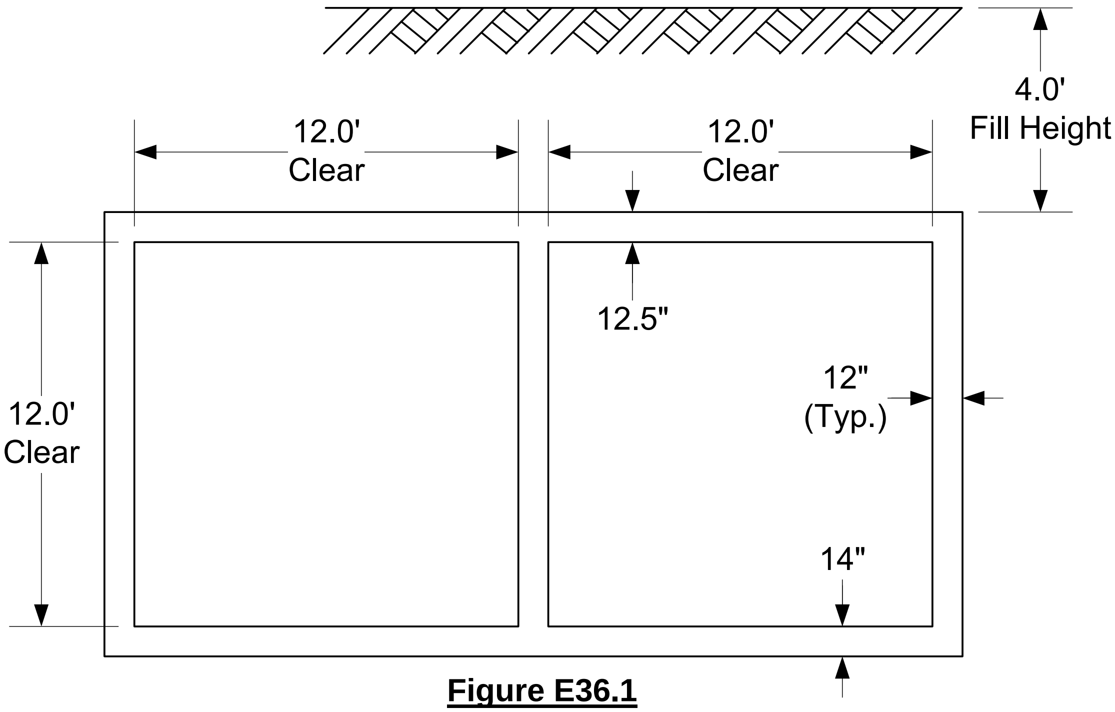

Figure E36.1
Box Culvert Dimensions

$NC = 2$	number of cells
$Ht = 12.0$	cell clear height, ft
$W_1 = 12.0$	cell 1 clear width, ft
$W_2 = 12.0$	cell 2 clear width, ft
$L = 134.0$	culvert length, ft
$t_{ts} = 12.5$	top slab thickness, in
$t_{bs} = 14.0$	bottom slab thickness, in
$t_{win} = 12.0$	interior wall thickness, in
$t_{wex} = 12.0$	exterior wall thickness, in

$H_{apron} := Ht + \frac{t_{ts}}{12}$ apron wall height above floor, ft

$H_{apron} = 13.04$

Calculate the nominal moment capacity of the rectangular section in accordance with **LRFD [5.7.3.2.3]**:

$a := \beta_1 \cdot c$ $\boxed{a = 0.83}$ in

$M_n := \left[A_s \cdot f_s \cdot \left(d_s - \frac{a}{2}\right) \frac{1}{12}\right]$ $\boxed{M_n = 23.0}$ kip-ft

For reinforced concrete cast-in-place box structures, $\phi_f = 0.90$ **LRFD [Table 12.5.5-1]**.

Therefore the usable capacity is:

$M_r := \phi_f \cdot M_n$ $\boxed{M_r = 20.7}$ kip-ft

The required capacity:

Corner Moment $\boxed{Mstr1_{CB} = 17.3}$ kip-ft

Check the section for minimum reinforcement in accordance with **LRFD [5.7.3.3.2]**:

$b = 12.0$ in width of the concrete design section, in

$h = 12.0$ in height of the concrete design section, in

$f_r := 0.37 \cdot \sqrt{f'_c}$ modulus of rupture, ksi **LRFD [5.4.2.6]** $f_r = 0.69$ ksi

$I_g := \frac{1}{12} \cdot b \cdot h^3$ gross moment of inertia, in^4 $I_g = 1728.00$ in^4

$\frac{h}{2} = 6.0$ distance from the neutral axis to the extreme element

$S_c := \frac{I_g}{\frac{h}{2}}$ section modulus, in^3 $S_c = 288.00$ in^3

The corresponding cracking moment is:

$M_{cr} = \gamma_3 (\gamma_1 \cdot f_r) S_c$ therefore, $M_{cr} = 1.1 (f_r) S_c$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$M_{cr} := 1.1 f_r \cdot S_c \cdot \frac{1}{12}$ $\boxed{M_{cr} = 18.3}$ kip-ft

$\boxed{1.33 \cdot Mstr1_{CB} = 23.1}$ kip-ft

Is $M_r = 20.7$ kip-ft greater than the lesser of M_{cr} and 1.33*M_{str}? $\boxed{check = \text{"OK"}}$

Per **LRFD [5.7.3.4]**, the spacing(s) of reinforcement in the layer closest to the tension face shall satisfy:

$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c \qquad \text{in which:} \qquad \beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$$

$\gamma_e := 1.0$ for Class 1 exposure condition

$h = 12.0$ height of the concrete design section, in

Calculate the ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face:

$$\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)} \qquad \boxed{\beta_s = 1.34}$$

Calculate the reinforcement ratio:

$$\rho := \frac{A_s}{b \cdot d_s} \qquad \boxed{\rho = 0.0043}$$

Calculate the modular ratio:

$$N := \frac{E_s}{E_c} \qquad \boxed{N = 8.06}$$

Calculate f_{ss}, the tensile stress in steel reinforcement at the Service I Limit State (ksi). The moment arm used in the equation below to calculate f_{ss} is: (j) (h-d_c)

$$k := \sqrt{(\rho \cdot N)^2 + (2 \cdot \rho \cdot N)} - \rho \cdot N \qquad \boxed{k = 0.2301}$$

$$j := 1 - \frac{k}{3} \qquad \boxed{j = 0.9233}$$

$Ms1_{CB} = 11.18$ service moment, kip-ft

$$f_{ss} := \frac{Ms1_{CB} \cdot 12}{A_s \cdot (j) \cdot (h - d_c)} \qquad \boxed{f_{ss} = 30.23} \text{ ksi}$$

Calculate the maximum spacing requirements per **LRFD [5.10.3.2]**:

$s_{max1} := \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$ $\qquad s_{max1} = 12.64$ in

$s_{max2} := \min(1.5 \cdot h, 18)$ $\qquad s_{max2} = 18.00$ in

$s_{max} := \min(s_{max1}, s_{max2})$ $\qquad \boxed{s_{max} = 12.64}$ in

Check that the provided spacing is less than the maximum allowable spacing

Is $spacing = 7.50$ in $\leq$ $s_{max} = 12.64$ in $\qquad \boxed{check = \text{"OK"}}$

Calculate the minimum spacing requirements per **LRFD [5.10.3.1]**. The clear distance between parallel bars in a layer shall not be less than:

$S_{min1} := 1.5 \cdot Bar_D(Bar_{No})$ $\qquad S_{min1} = 0.94$ in

$S_{min2} := 1.5 \cdot 1.5$ (maximum aggregate size = 1.5 inches) $\qquad S_{min2} = 2.25$ in

$S_{min3} := 1.5$ in

Is $spacing = 7.50$ in $\geq$ all minimum spacing requirements? $\qquad \boxed{check = \text{"OK"}}$

E36-1.8 Shrinkage and Temperature Reinforcement Check

Check shrinkage and temperature reinforcement criteria for the reinforcement selected in preceding sections.

The area of reinforcement (A_s) per foot, for shrinkage and temperature effects, on each face and in each direction shall satisfy: **LRFD [5.10.8]**

$$A_s \geq \frac{1.30 \cdot b \cdot (h)}{2 \cdot (b + h) \cdot f_y} \qquad \text{and} \qquad 0.11 \leq A_s \leq 0.60$$

Where:

A_s = area of reinforcement in each direction and each face $\left(\frac{in^2}{ft}\right)$

b = least width of component section (in.)

h = least thickness of component section (in.)

f_y = specified yield strength of reinforcing bars (ksi) $\leq$ 75 ksi

Check the minimum required temperature and shrinkage reinforcement, #4 bars at 15", in the thickest section. For the given cross section, the values for the corner bar design are:

$A_{s\_4\_at\_15} := \frac{Bar_A(4)}{1.25}$ $\qquad \boxed{A_{s\_4\_at\_15} = 0.16}$ $\frac{in^2}{ft}$

$b_{TS} := \max(t_{ts}, t_{bs}, t_{wex})$ $\boxed{b_{TS} = 14.0}$ in

$h_{TS} := 12(W_1 + W_2) + 2 \cdot t_{wex} + t_{win}$ $\boxed{h_{TS} = 324.0}$ in

$f_y = 60.00$ ksi

For each face, the required area of steel is:

$$A_{s\_TS} := \frac{1.30 \cdot (b_{TS}) \cdot h_{TS}}{2 \cdot (b_{TS} + h_{TS}) \cdot f_y} \qquad A_{s\_TS} = 0.15 \ \frac{in^2}{ft}$$

is $A_{s\_4\_at\_15} = 0.16$ in2 $\geq$ $A_{s\_TS} = 0.15$ in^2 ? $\boxed{check = \text{"OK"}}$

is $0.11 < A_{s\_4\_at\_15} < 0.60$? $\boxed{check = \text{"OK"}}$

Per **LRFD [5.10.8]**, the shrinkage and temperature reinforcement shall not be spaced farther apart than:

- 3.0 times the component thickness, or 18.0 in.
- 12.0 in for walls and footings greater than 18.0 in. thick
- 12.0 in for other components greater than 36.0 in. thick

$s_{max3} = 18.00$ in

Per **LRFD [5.10.3.2]**, the maximum center to center spacing of adjacent bars shall not exceed 1.5 times the thickness of the member or 18.0 in.

$s_{max4} = 18.00$ in

is the 15" spacing $\leq$ both maximum spacing requirements? $\boxed{check = \text{"OK"}}$

The results for the other bar locations are shown in the table below:

Results						
Location	ΦMn	$A_{S\ Req'd}$	$A_{S\ Actual}$	Bar Size	S_{max}	S_{actual}
Corner	20.7	0.48	0.50	5	12.6	7.5
Pos. Mom. Top Slab	21.8	0.49	0.50	5	13.0	7.5
Pos. Mom. Bot. Slab	28.9	0.54	0.57	5	18.0	6.5
Neg. Mom. Top Slab	23.3	0.50	0.53	5	12.1	7.0
Neg. Mom. Bot. Slab	28.4	0.54	0.62	5	13.4	6.0
Exterior Wall	16.9	0.37	0.40	4	18.0	6.0
Interior Wall	6.9	0.15	0.16	4	18.0	15.0

Is $M_{r2} = 10.6$ kip-ft greater than the lesser of M_{cr} and $1.33*M_{str}$? $\boxed{check = \text{"OK"}}$

$\boxed{M_{cr} = 18.3}$ kip-ft

$\boxed{1.33 \cdot Mstr1_{CBV2} = 10.5}$ kip-ft

Calculate f_{ss}, the tensile stress in steel reinforcement at the Service I Limit State (ksi).

$Ms1_{CBV2} = 3.43$ service moment at the second cutoff location, kip-ft

$$f_{ss2} := \frac{Ms1_{CBV2} \cdot 12}{A_s \cdot (j) \cdot (h - d_c)} \qquad \boxed{f_{ss} = 30.23} \text{ ksi}$$

Calculate the maximum spacing requirements per **LRFD [5.10.3.2]**:

$$s_{max2\_1} := \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss2}} - 2 \cdot d_c \qquad s_{max2\_1} = 51.69 \text{ in}$$

$$s_{max2\_2} := s_{max2} \qquad s_{max2\_2} = 18.00 \text{ in}$$

$$s_{max} := \min(s_{max2\_1}, s_{max2\_2}) \qquad \boxed{s_{max} = 18.00} \text{ in}$$

Check that the provided spacing (for half of the bars) is less than the maximum allowable spacing

$spacing2 := 2 \cdot spacing$ $spacing2 = 15.00$ in

Is $spacing2 = 15.00$ in $\leq$ $s_{max} = 18.00$ in $\boxed{check = \text{"OK"}}$

Extension Lengths:

The extension lengths for the corner bars are shown below. Calculations for other bars are similar.

Extension lengths for general reinforcement per **LRFD [5.11.1.2.1]**:

$$MaxDepth := \max\left(t_{ts} - cover, t_{wex} - cover, t_{bs} - cover_{bot}\right) \qquad MaxDepth = 11.00 \quad in$$

Effective member depth $\dfrac{MaxDepth - \frac{1}{2} Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.89$ ft

15 x bar diameter $\dfrac{15 \cdot Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.78$ ft

1/20 times clear span $\dfrac{\max\left(W_1, W_2\right)}{20} = 0.60$ ft

The maximum of the values listed above:

$$ExtendLength\_gen_{CB} = 0.89 \quad ft$$

Extension lengths for negative moment reinforcement per **LRFD [5.11.1.2.3]**:

Effective member depth $\dfrac{MaxDepth - \frac{1}{2} Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.89$ ft

12 x bar diameter $\dfrac{12 \cdot Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.63$ ft

0.0625 times clear span $0.0625 \max\left(W_1, W_2\right) = 0.75$ ft

The maximum of the values listed above:

$$ExtendLength\_neg_{CB} = 0.89 \quad ft$$

The development length:

$$DevLength_{CB} = 1.00 \quad ft$$

Distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement:

$$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2} \qquad \boxed{d_s = 9.69} \text{ in}$$

The effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; **LRFD [5.8.2.9]**

$$d_{v\_i} = d_s - \frac{a}{2}$$

from earlier calculations:

$$\boxed{\beta_1 = 0.85}$$

$$\boxed{f_s = 60} \text{ ksi}$$

$$\boxed{A_{s\_XW} = 0.40} \text{ in}^2$$

The distance between the neutral axis and the compression face:

$$c := \frac{A_{s\_XW} \cdot f_s}{0.85 \cdot f'_c \cdot \beta_1 \cdot b_v} \qquad \boxed{c = 0.79} \text{ in}$$

$$a := \beta_1 \cdot c \qquad \boxed{a = 0.67} \text{ in}$$

The effective shear depth:

$$d_{v\_i} := \left(d_s - \frac{a}{2}\right) \qquad \boxed{d_{v\_i} = 9.35}$$

d_v need not be taken to be less than the greater of 0.9 d_s or 0.72h (in.)

$$d_v := \max\left(d_{v\_i}, \max\left(0.9d_s, 0.72t_{wex}\right)\right) \qquad 0.9 \cdot d_s = 8.72$$

$$d_v = 9.35 \text{ in} \qquad 0.72 \cdot t_{wex} = 8.64$$

For reinforced concrete cast-in-place box structures, $\phi_v = 0.85$, **LRFD [Table 12.5.5-1]**.

Therefore the usable capacity is:

$$V_{r1w} := \phi_v \cdot \left(0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v\right) \qquad \boxed{V_{r1w} = 11} \text{ kips}$$

$$\text{but} \le \quad V_{r2w} := \phi_v \cdot \left(0.25 \cdot f'_c \cdot b_v \cdot d_v\right) \qquad \boxed{V_{r2w} = 83} \text{ kips}$$

$$V_{rw} := \min\left(V_{r1w}, V_{r2w}\right) \qquad \boxed{V_{rw} = 11} \text{ kips}$$

$$V_u := Vstr1_{XW} \qquad \boxed{V_u = 8.7} \text{ kips}$$

Check that the provided shear capacity is adequate:

Is $V_u = 8.7\ \text{kip} \le V_{rw} = 11.3\ \text{kip}$? $\boxed{\text{check} = \text{"OK"}}$

Table of Contents

37.1 Structure Selection

Most pedestrian bridges are located in urban areas and carry pedestrian and/or bicycle traffic over divided highways, expressways and freeway systems. The structure type selected is made on the basis of aesthetics and economic considerations. A wide variety of structure types are available and each type is defined by the superstructure used. Some of the more common types are as follows:

- Concrete Slab
- Prestressed Concrete Girder
- Steel Girder
- Prefabricated Truss

Several pedestrian bridges are a combination of two structure types such as a concrete slab approach span and steel girder center spans. One of the more unique pedestrian structures in Wisconsin is a cable stayed bridge. This structure was built in 1970 over USH 41 in Menomonee Falls. It is the first known cable stayed bridge constructed in the United States. Generally, pedestrian bridges provide the designer the opportunity to employ long spans and medium depth sections to achieve a graceful structure.

Pedestrian boardwalks will not be considered “bridges” when their clear spans are less than or equal to 20 feet, and their height above ground and/or water is less than 10 feet. Boardwalks falling under these constraints will not be required to follow the design requirements in the WisDOT Bridge Manual, but will need to follow the standards established in the *Wisconsin Bicycle Facility Design Handbook.*

37.2 Specifications and Standards

The designer shall refer to the following related specifications:

- "*AASHTO LRFD Bridge Design Specifications*"
- *"AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges",* hereafter referred to as the *"Pedestrian Bridge Guide"*
- See *Standardized Special Provision (STSP)* titled "*Prefabricated Steel Truss Pedestrian Bridge LRFD*" for the requirements for this bridge type

For additional design information, refer to the appropriate Wisconsin Bridge Manual chapters relative to the structure type selected.

The pedestrian live load (PL) shall be as follows: (from "*Pedestrian Bridge Guide")*

- 90 psf **[Article 3.1]**
- Dynamic load allowance is not applied to pedestrian live loads **[Article 3.1]**

The vehicle live load shall be applied as follows: (from *"Pedestrian Bridge Guide")*

- Design for an occasional <u>single</u> maintenance vehicle live load (LL) **[Article 3.2]**

Clear Bridge Width (w)	Maintenance Vehicle
7 ft $\leq$ w $\leq$ 10 ft	H5 Truck (10,000 lbs)
w > 10 ft	H10 Truck (20,000 lbs)

- Clear bridge widths of less than 7 feet need not be designed for maintenance vehicles. **[Article 3.2]**
- The maintenance vehicle live load shall not be placed in combination with the pedestrian live load. **[Article 3.2]**
- Dynamic load allowance is not applied to the maintenance vehicle. **[Article 3.2]**
- Strength I Limit State shall be used for the maintenance vehicle loading. **[Article 3.2, 3.7]**

On Federal Aid Structures FHWA requests a limiting gradient of 8.33 percent (1:12) on ramps for pedestrian facilities to accommodate the physically handicapped and elderly as recommended by the "*American Standard Specifications for Making Buildings and Other Facilities Accessible to, and Usable by, the Physically Handicapped*". This is slightly flatter than the gradient guidelines set by AASHTO which states gradients on ramps should not be more than 15 percent and preferably not steeper than 10 percent.

The minimum inside clear width of a pedestrian bridge on a pedestrian accessible route is 8 feet. (*AASHTO Guide for the Planning, Design, and Operation of Pedestrian Facilities, 2004).*

The width required is based on the type, volume, and direction of pedestrian and/or bicycle traffic.

The vertical clearance on the pedestrian bridge shall be a minimum of 10 feet for bicyclists' comfort and to allow access for maintenance and emergency vehicles. The Wisconsin Department of Natural Resources recommends a vertical clearance on the bridge of at least 12 feet to accommodate maintenance and snow grooming equipment on state trails. Before beginning the design of the structure, the Department of Natural Resources and the Bureau of Structures should be contacted for the vertical clearance requirements for all vehicles that require access to the bridge.

In addition, ramps should have rest areas or landings 5 feet to 6 feet in length which are level and safe. Rest area landings are mandatory when the ramp gradient exceeds 5 percent. Recommendations are that landings be spaced at 60 foot maximum intervals, as well as wherever a ramp turns. This value is based on a maximum gradient of 8.33 percent on pedestrian ramps, and placing a landing at every 5 feet change in vertical elevation. Also, ramps are required to have handrails on both sides. See Standard Details for handrail location and details.

Minimum vertical clearance for a pedestrian overpass can be found in the *Facilities Development Manual (FDM)* Procedure 11-35-1, Attachment 8 and 9. Horizontal clearance is provided in accordance with the requirement found in *(FDM)* Procedure 11-35-1, Attachment 5 and 6.

Live load deflection limits shall be in accordance with the provisions of **LRFD [2.5.2.6.2]** for the appropriate structure type.

37.3 Protective Screening

Protective Screening is recommended on all pedestrian overpasses due to the increased number of incidents where objects were dropped or thrown onto vehicles traveling below. Several types of screening material are available such as aluminum, fiberglass and plastic sheeting, and chain link type fencing. A study of the various types of protective screening available indicates that chain link fencing is the most economical and practical for pedestrian overpasses. For recommended applications refer to the Standard Details.

The top of the protective screening may be enclosed (not required) with a circular section in order to prevent objects from being thrown over the sides and to discourage people from climbing on (over) the top. The opening at the bottom is held at a 1 inch clearance to prevent objects from being pushed under the fence.

The core wire of the fence fabric shall be a minimum of 9 gauge (0.148 inch) thickness, galvanized and woven in a 2 inch mesh. A 1 inch mesh may be used in highly vulnerable areas. A vinyl coating may also be used for aesthetic purposes. Add a special provision to the contract if these additional features are used. Special provisions for common items are available as STSP's or on the Wisconsin Bridge Manual website.

Region project staff should be consulted with regards to fencing preferences.

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Table of Contents

spalling 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 pedestrian fencing where required.

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 depth from top of rail to top of footing is 6'-6" (unless bedrock is present).

- Drainage Ditches - Show ditches and direction of flow.

- Utilities - Show all utilities that are near structure footings and proposed relocation is required.

Table of Contents

40.5 Deck Overlays

If the bridge is a candidate for replacement or a new deck, serviceability may be extended 3 to 7 years by patching and/or overlaying the deck with only a 1 1/2" minimum thickness asphaltic mat on lightly traveled roadways. Experience indicates the asphalt tends to slow down the rate of deterioration while providing a smooth riding surface. However, these decks must be watched closely for shear or punching shear failures as the deck surface problems are concealed.

For applications where the deck is structurally sound and service life is to be extended there are other methods to use. A polymer modified asphaltic overlay may be used to increase deck service life by approximately 15 years. If the concrete deck remains structurally sound, it may be practical to remove the existing overlay and place a new overlay before replacing the deck.

A 1 1/2" concrete overlay is expected to extend the service life of a bridge deck for 15 to 20 years. On delaminated but structurally sound decks a concrete overlay is often the only alternative to deck replacement. Prior to placing the concrete overlay, a minimum of 1" of existing deck surface should be removed. On all bridges low slump Grade E concrete is the specified standard with close inspection of concrete consolidation and curing. If the concrete deck remains structurally sound; it may be practical to remove the existing overlay and place a second deck overlay before replacing the entire deck. After the concrete overlay is placed, it is very important to seal all the deck cracks. Experience shows that salt water passes thru these cracks and causes deterioration of the underlying deck.

On deck overlays preparation of the deck is an important issue after removal of the top surface. Check the latest Special Provisions and/or specifications for the method of payment for Deck Preparation where there are asphalt patches or unsound concrete.

Micro-silica concretes have been effectively used as an alternate type of concrete overlay. It provides excellent resistance to chloride penetration due to its low permeability. Micro-silica modified concrete overlays appear very promising; however, they are still under experimental evaluation. Latex overlays when used in Wisconsin have higher costs without noticeable improved performance.

Ready mixed Grade E concrete with superplasticizer and fiber mesh have been tried and do not perform any better than site mixed concrete produced in a truck mounted mobile mixer.

Bridges with Inventory Ratings less than HS10 with an overlay shall not be considered for concrete overlays, unless approved by Structures Design. Bridges reconstructed with overlays shall have their new inventory and operating ratings shown on the bridge rehabilitation plans. Verify the desired transverse cross slope with the Regions as they may want to use current standards.

40.5.1 Guidelines for Bridge Deck Overlays

As a structure ages, rehabilitation is a necessary part of insuring a level of acceptable serviceability. Overlays can be used to extend the service lives of bridge decks that have surface deficiencies. Guidelines for determining if an overlay should be used are:

- The structure is capable of carrying the overlay deadload;
- 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 structure life.

40.5.2 Deck Overlay Methods

An AC Overlay or Polymer Modified Asphaltic Overlay should not be considered on a bridge deck which has a longitudinal grade in excess of four percent or an extensive amount of stopping and starting traffic. All full depth repairs shall be made with PC concrete.

Guidelines for determining the type of deck overlay method to achieve the desired extended service life are:

AC Overlay (ACO): 5 years average life expectancy

- The minimum asphaltic overlay thickness is 1 1/2”.
- The grade change due to overlay thickness can be accommodated at minimal cost.
- Deck or bridge replacement is programmed within 7 years.
- Raising of floor drains or joints is not required.
- Spalls can be patched with AC or PC concrete with minimal surface preparation.

Polymer Modified Asphaltic Overlay: 15 to 20 years life expectancy

- This product may be used as an experimental alternate to LSCO given below. CAUTION – Core tests have shown the permeability of this product is dependent on the aggregate. Limestone should not be used.

Polymer Overlay: 10 to 15 years life expectancy

- A 3/8-inch thick, two layer system comprised of a two-component polymer in conjunction with natural or synthetic aggregates. Use 5 psf for dead load, DW.
- Works well to seal decks and/or provide traction.

The minimum required concrete age is 28 days prior to application, although a longer period of time would allow more initial concrete cracking to occur which the resin would then be able to seal.

AC Overlay with a Waterproofing Membrane (ACOWM): (Currently not used)

Low Slump Concrete Overlay *(LSCO): 15 to 20 years life expectancy

- Minimum thickness is 1 1/2" PC concrete overlay.
- Joints and floor drains will be modified to accommodate the overlay.
- Deck deficiencies will be corrected with PC concrete.
- The prepared deck surfaces will be scarified or shot blasted.
- There is no structural concern for excessive leaching at working cracks.
- Combined distress area is less than 25%.
- May require crack sealing the following year and periodically thereafter.

* Note: Or another PC concrete product as approved by Structures Development and coordinated with the Region.

40.5.3 Maintenance Notes

- All concrete overlays crack immediately. If the cracks in the deck are not sealed periodically, the rate of deterioration can increase rapidly.
- AC overlays with a waterproofing membrane can also be used on new decks or older decks that are in good condition as preventive maintenance.

40.5.4 Special Considerations

On continuous concrete slab bridges with extensive spalling in the negative moment area, not more than 1/3 of the top bar steel should be exposed if the bar ends are not anchored. This is to maintain the continuity of the continuous spans and should be stated on the final structure plans.

If more than 1/3 of the steel is exposed, either the centers of adjacent spans must be shored or only longitudinally overlay 1/3 of the bridge at a time.

40.5.5 Railings and Parapets

The top of the overlay should not go above the 3-inch vertical portion of a concrete parapet. Additionally, overlays increase vehicle lean over sloped face parapets resulting in vehicles on bridges with higher ADT and/or speed having an increased likelihood of impact with lights/obstructions on top of, or behind, the parapet.

Sub-standard railings and parapets should be improved. An example of such a sub-standard barrier would be a curb with a railing or parapet on top. Contact the Bureau of Structures Development section to discuss solutions.

40.6 Deck Replacements

Depending on the structure age or site conditions, the condition of original deck or deck overlay, a complete deck replacement may be the most cost effective and extend the life of the bridge by 40 years or more. Epoxy coated rebars are required on bridge deck replacements under the same criteria as for new bridges. The new deck and parapet or railing shall be designed per the most recent edition of the WisDOT Bridge Manual, including continuity bars and overhang steel. Refer to the criteria in 40.3 of this chapter for additional 3R project considerations. The top flange of steel girders should be painted.

The following condition or rating criteria are the minimum requirements on STN bridges eligible for deck replacements:

Item	Existing Condition	Condition after Construction
Deck Condition	≤ 4	≥ 8
Inventory Rating	---	≥ HS15*
Superstructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Substructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Horizontal and Vertical Alignment Condition	> 3	---
Shoulder Width	6 ft	6 ft

Table 40.6-1
Condition Requirements for Deck Replacements

*Rating evaluation is based on the criteria found in Chapter 45 – Bridge Rating. An exception to requiring a minimum Inventory Rating of HS15 is made for continuous steel girder bridges, with the requirement for such bridges being a minimum Inventory Rating of HS10. For all steel girder bridges, assessment of fatigue issues as well as paint condition (and lead paint concerns) should be included in the decision as to whether a deck replacement or a superstructure/bridge replacement is the best option.

For any bridge not meeting the conditions for deck replacement, a superstructure, or likely a complete bridge replacement is recommended. For all Interstate Highway deck replacements, the bridges are to have a minimum Inventory Rating of HS20 after the deck is replaced.

WisDOT policy item:

Please contact the Bureau of Structures Development Section if a deck replacement for an Interstate Highway bridge would result in an Inventory Rating greater than HS18, but less than HS20.

45" Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	102	112
6'-6"	100	110
7'-0"	98	108
7'-6"	96	102
8'-0"	94	100
8'-6"	88	98
9'-0"	88	96
9'-6"	84	90
10'-0"	84	88
10'-6"	82	86
11'-0"	78	85
11'-6"	76	84
12'-0"	70	80

54" Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	130	138
6'-6"	128	134
7'-0"	124	132
7'-6"	122	130
8'-0"	120	128
8'-6"	116	124
9'-0"	112	122
9'-6"	110	118
10'-0"	108	116
10'-6"	106	112
11'-0"	102	110
11'-6"	100	108
12'-0"	98	104

70" Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	150*	160*
6'-6"	146*	156*
7'-0"	144*	152*
7'-6"	140*	150*
8'-0"	138*	146*
8'-6"	134*	142*
9'-0"	132*	140*
9'-6"	128*	136
10'-0"	126*	134
10'-6"	122	132
11'-0"	118	128
11'-6"	116	126
12'-0"	114	122

Table 40.7-1
Maximum Span Length vs. Girder Spacing

* For lateral stability during lifting these girder lengths will require pick up point locations greater than distance d (girder depth) from the ends of the girder. The designer shall assume that the pick up points will be at the 1/10 points from the end of the girder and provide extra non-prestressed steel in the top flange if required.

40.8 Widenings

Deck widenings, except on the Interstate, are attached to the existing decks if they are structurally sound and the remaining width is more than 50 percent of the total new width. Also, reference is made to the criteria in 40.3 of this Chapter for additional 3R project considerations. If the existing deck is over 20 percent surface delaminated or spalled, the existing deck shall be replaced. For all deck widenings on Interstate Highway bridges, the total deck should be replaced in order that total deck life is equal and costs are likely to be less when considering future traffic control. Evaluate the cost of traffic control for deck widenings on other highway bridges. The total deck should be replaced in these cases where the life-cycle cost difference is minimal if future maintenance costs are substantially reduced.

The design details must provide a means of moment and shear transfer through the joint between the new and existing portions of the deck. Lapped reinforcing bars shall have adequate development length and are preferable to doweled bars. The reinforcing laps must be securely tied or the bars joined by mechanical methods. When practical, detail lapped rebar splices. Mechanical splice couplers or threaded rebar couples are expensive and should only be detailed when required. Generally, shear transfer is more than sufficient without a keyway. Bridge Maintenance Engineers have observed that if the existing bar steel is uncoated, newly lapped coated bars will accelerate the uncoated bar steel deterioration rate.

When widening a deck on a prestressed girder bridge, if practical, use the latest standard shape (e.g. 54W" rather than 54"). Spacing the new girder(s) to maintain comparable bridge stiffness is desirable. *The girders used for widenings may be the latest Chapter 19 sections designed to LRFD or the sections from Chapter 40 designed LFD, or LRFD with non-prestressed reinforcement as detailed in the Standard Details.*

For multi-columned piers, consider the cap connection to the existing cap as pinned and design the new portion to the latest LRFD criteria. The new column(s) are not required to meet AASHTO 3.6.5 (600 kip loading) as a widening is considered rehabilitation. *It is intended to provide standard details in the Bridge Manual for a crash barrier that could, at the option of the Region, be used to strengthen and provide motorists protection for existing piers, including widenings.*

Abutments shall be widened to current LRFD criteria as well as the Standard for Abutment Widening.

For foundation support, use the most current method available.

All elements of the widening shall be designed to current LRFD criteria. Railings and parapets placed on the new widened section shall be up to current design and safety standards. For the non-widened side, substandard railings and parapets should be improved. Contact the Bureau of Structures Development section to discuss solutions.

For prestressed girder widenings, only use intermediate steel diaphragms in-line with existing intermediate diaphragms (i.e. don't add intermediate lines of diaphragms).

40.9 Superstructure Replacements/Moved Girders (with Widening)

When steel girder bridges have girder spacings of 3' or less and require expensive redecking or deck widening; consider replacing the superstructure with a concrete slab. Approval is required from BOS for all Superstructure replacement projects. Also, additional costs will occur if bearings, floor drains, expansion joints, railings, or other structural elements require replacement.

When replacing a deck on a steel girder bridge, the cost of painting and structural rehab shall be considered to determine whether a deck or superstructure replacement is more cost effective (provided the substructure is sufficient to support the loading).

Evaluate the existing piers using current LRFD criteria. If an existing multi-columned pier has 3 or more columns, the 600 kip vehicular impact loading need not be considered if the pier is adjacent to a roadway with a design speed ≤ 40 mph. If the design speed is 45 mph or 50 mph, the 600 kip vehicular impact loading need not be considered if a minimum of “vehicle protection” is provided as per FDM 11-35-1. For design speeds > 50 mph, all criteria as per 13.4.10 must be met.

For abutments, evaluate the piles, or bearing capacity of the ground if on spread footings, utilizing Service I loading. The abutment body should be evaluated using Strength I loading.

The superstructure shall be design to current LRFD criteria.

40.10 Replacement of Impacted Girders

When designing a replacement project for girders that have been damaged by vehicular impact, replace the girder in-kind using the latest details. Either HS loading or HL-93 loading may be used if originally designed with non-LRFD methods. Consider using the latest deck details, especially with regard to overhang bar steel.

For the parapet or railing, the designer should match the existing.

40.11 New Bridge Adjacent to Existing Bridge

For a new bridge being built adjacent to an existing structure, the design of the new structure shall be to current LRFD criteria for the superstructure and abutment.

The pier design shall be to current LRFD criteria, including the 600 kip impact load for the new bridge. It is not required to strengthen or protect the existing adjacent pier for the 600 kip impact load. However, it would be prudent to discuss with the Region the best course of action. If the Region wants to provide crash protection, it may be desirable to provide TL-5 barrier/crash wall protection for both structures, thus eliminating the need to design the new pier for the 600 kip impact load. The Region may also opt to provide typical barrier protection (< TL-5) to both sets of piers, in which case the design engineer would still be required to design the new pier for the 600 kip impact load. This last option is less expensive than providing TL-5 barrier to both structures. Aesthetics are also a consideration in the above choices.

40.12 Timber Abutments

The use of timber abutments shall be limited to rehabilitation or widening of off-system structures.

Timber abutments consist of a single row of piling capped with timber or concrete, and backed with timber to retain the approach fill. The superstructure types are generally concrete slab or timber. Timber-backed abutments currently exist on Town Roads and County Highways where the abutment height does not preclude the use of timber backing.

Piles in bents are designed for combined axial load and bending moments. For analysis, the assumption is made that the piles are supported at their tops and are fixed 6 feet below the stream bed or original ground line. For cast-in-place concrete piling, the concrete core is designed to resist the axial load. The bending stress is resisted by the steel shell section. Due to the possibility of shell corrosion, steel reinforcement is placed in the concrete core equivalent to a 1/16-inch steel shell perimeter section loss. The reinforcement design is based on equal section moduli for the two conditions. Reinforcement details and bearing capacities are given on the Standard Detail for Pile Details. Pile spacing is generally limited to the practical span lengths for timber backing planks.

The requirements for tie rods and deadmen is a function of the abutment height. Tie rods with deadmen on body piling are used when the height of "freestanding" piles is greater than 12 feet for timber piling and greater than 15 feet for cast-in-place concrete and steel "HP" piling. The "freestanding" length of a pile is measured from the stream bed or berm to grade. If possible, all deadmen should be placed against undisturbed soil.

Commercial grade lumber as specified in AASHTO having a minimum flexural resistance of 1.2 ksi is utilized for the timber backing planks. The minimum recommended nominal thickness and width of timber backing planks are 3 and 10 inches, respectively. If nominal sizes are specified on the plans, analysis computations must be based on the dressed or finished sizes of the timber. Design computations can be used on the full nominal sizes if so stated on the bridge plans. For abutments constructed with cast-in-place concrete or steel "HP" piles, the timber planking is attached with 60d common nails to timber nailing strips which are bolted to the piling.

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

All steel bridge bearings should be replaced as shown in Bridge Manual Chapter 27. Compressive load and adhesion tests will be waived for steel laminated elastomeric bearings where these bearings are detailed to meet height requirements.

In general replace lubricated bronze bearings with Teflon bearings. If only outside bearings are replaced, the difference in friction factors can be ignored. Where lubricated bronze bearings might be used, following is the design criteria.

For the expansion bearings, two additional plates are employed over fixed bearings, a top plate and a lubricated bronze plate. Current experience indicates that a stainless steel top plate reduces corrosion activity and is the recommended alternate to steel. The top plate is set on top of a lubricated bronze plate allowing expansion and contraction to occur. Laboratory testing of lubricated bronze plates indicated a maximum coefficient of friction varying from 8 to 14 percent for a loading of 200 kips. Current Office practice for steel girder Type “A” and prestressed girder expansion bearings of employing a 10 percent maximum and a 6 percent minimum friction value for design is in accordance to laboratory test results. For Type “A” bearing details refer to Standard Details.

40.16 Concrete Masonry Anchors for Rehabilitation

"Type S" and "Type L" concrete masonry anchors are used mostly for bridge rehabilitation projects and anchoring pedestrian rail posts. One of the main differences between the two types of anchors is the duration of loading. It may be helpful to think of the "S" as Short-term loading and the "L" as Long-term loading. "Type L" anchors have greater embedment lengths in tension to account for longer duration loading.

For both types of anchors the minimum pullout capacity (Nominal Tensile Resistance) specified on the plan is determined according to 40.16.3. If additional capacity is required, a more refined analysis per ACI 318-11 Appendix D is allowable. References in this section, **ACI [D._]** refer to articles in this document. (*AASHTO* currently does not have guidance for anchors.) *It should be noted that WisDOT is currently evaluating adhesive anchored parapet replacements, which would be crash tested and consideration of the resistance factor would not be required*.

40.16.1 Adhesive Anchor Requirements

For all non-mechanical anchors, a two-part adhesive is either mixed and poured into a drilled hole or pumped into the hole by a dispenser which combines the two components at the nozzle just prior to entering the hole or within the hole. The hole must be properly cleaned and a sufficient amount of adhesive used so that the hole is completely filled with adhesive when the rebar or bolt is inserted. The adhesive bond stress is determined by the 5 percent fractile of results of tests performed and evaluated according to ACI 355.4. The bond stress must be at least equal to:

$$\tau = \frac{500}{d_a} + 1100 \text{ (psi)}$$

Where:

d_a = Anchor diameter (in)

40.16.2 Masonry Anchor Types and Usage

"Type S" anchors are either mechanical wedge or adhesive anchors for installing studs, rebar, or bolts of a designated size. They are primarily used for anchoring bolts for attaching rail posts or other bolted objects and mostly smaller size rebars. "Type S" mechanical wedge anchors are seldom used for bridge rehabilitation. Because of creep, shrinkage and deterioration under load and freeze-thaw cycles, "Type S" adhesive anchors should not be used in situations where the rebar experiences a constant tension stress. When "Type S" anchors are used to anchor rebars, the rebar is listed in the "Bill of Bars" and paid for under the bid item "Bar Steel Reinforcement HS (Coated) Bridges". Unless shown as an option on a Standard, "Type S" anchors may be used only with the approval of the Bureau of Structures and design calculations must be submitted.

"Type L" anchors are adhesive anchors used to anchor rebars when the rebar is subject to continuous loading. "Type L" anchors are typically used for abutment and pier widenings, but may be used in other applications. For “Type L” anchors the rebar is listed in the “Bill of Bars” and paid for under the bid item “Bar Steel Reinforcement HS (Coated) Bridges”.

The anchor steel is considered ductile if the tensile test elongation is at least 14 percent and reduction in area is at least 30 percent. Steel meeting the requirements of ASTM A307 are also considered ductile. Rebar used as anchor steel is considered ductile. Steel that does not meet these requirements is considered brittle.

Usage Restrictions: 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. **Contact the Bureau of Structures if considering any extension of a hammerhead pier (without additional vertical support from an added column) or for vertical overhead installations.**

The manufacturer and product name of the “Type L” anchors and “Type S” adhesive anchors used by the contractor must be on the Department’s approved product list for “Concrete Masonry Anchors, Type L”.

The required minimum anchor spacing is 6 times the diameter of the anchor. The minimum edge distance is 6 times the diameter of the anchor for adhesive anchors, and 10 times the diameter for mechanical anchors. The minimum member thickness is the greater of the embedment depth plus 4 in. and 3/2 of the embedment depth. The maximum embedment depth for adhesive anchors is 20 times the diameter.

40.16.3 Masonry Anchor Reinforcement

Reinforcement used to transfer the full design load from the anchors into the structural member is anchor reinforcement. **ACI [D.5.2.9]** and **ACI [D.6.2.9]** give guidance for designing anchor reinforcement. Reinforcement that acts to restrain the potential concrete breakout but is not designed to transfer the full design load is supplementary reinforcement.

40.16.4 Masonry Anchor Tensile Capacity

Masonry anchors in tension fail in either steel tensile rupture, concrete breakout or adhesive bond. Figure 40.16-1 shows the concrete breakout failure for anchors in tension. As long as the minimum bond stress and the embedment depths defined in Table 40.16-1 are met, the adhesive anchor bond strength will not control. For bond stresses lower than the minimum or embedment depths other than those given in Table 40.16-1 for adhesive anchors, the adhesive bond strength must be checked per **ACI [D.5.5]**.

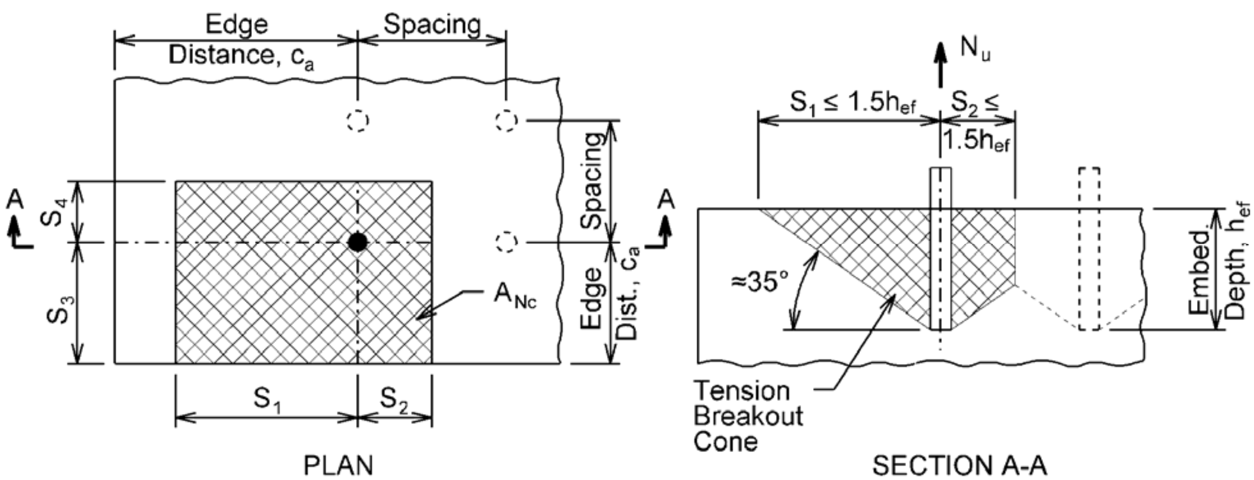


Figure 40.16-1
Concrete Breakout of Masonry Anchors in Tension

The projected concrete breakout area, A_{Nc}, shown in Figure 40.16-1 is limited in each direction by S_i:

S_i = Minimum of 1.5 times the embedment depth, half of the spacing to the next anchor, or the edge distance (in)

Anchor Size, d_a	Type S Anchors			Type L Anchors
	Mechanical	Adhesive		
	Min h_{ef} in	Min h_{ef} in	Max h_{ef} in	Min h_{ef} in
#4 or 1/2"	2.5	4.5	5.0	7.5
#5 or 5/8"	2.5	5.0	6.0	10.5
#6 or 3/4"	3.0	6.0	7.0	13.0
#7 or 7/8"	3.5	6.5	8.5	16.0
#8 or 1"	4.0	7.0	10.0	19.0
#9 or 1-1/8"	4.5	7.5	11.5	22.0

Table 40.16-1
Tension Embedment Depths for Concrete Masonry Anchors, Type S and Type L

The factored tension force on each anchor, N_u, must be less than or equal to the factored tensile resistance, N_r:

$$N_r = \phi_{tc} N_n \leq \phi_{ts} A_s f_u$$

$$N_n = \frac{A_{Nc}}{9(h_{ef})^2} \Psi_{et} \Psi_{ct} N_b \text{ (Nominal Tensile Resistance)}$$

In which:

$$N_b = 0.752\sqrt{f'_c}\left(h_{ef}\right)^{1.5} \text{ (kips)}$$

$$\Psi_{et} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} \leq 1.00$$

Ψ_{ct} = $\dfrac{c_{a,min}}{c_{ac}} \geq \Psi_{c,min} \leq 1.0$ for anchors without supplementary reinforcement per 40.16.3 located in a region of a concrete member where analysis indicates no cracking at service load levels. If no supplementary reinforcement exists for a Type L anchor, bond strength must be checked per **ACI [D.5.5]**.

= 0.71 if cracking exists at service load levels

= 1.0 if supplementary reinforcement exists to control splitting per 40.16.3

Where:

ϕ_{tc} = 0.65 for anchors without supplemental reinforcement

= 0.75 for anchors with supplemental reinforcement to control splitting per 40.16.3

ϕ_{ts} = 0.65 for brittle steel, and 0.75 for ductile steel as defined in 40.16.2

A_s = Effective anchor cross sectional area (in^2)

f_u = Steel tensile strength, not greater than the smaller of $1.9f_y$ and 125 ksi

f'_c = Concrete compressive strength, not greater than 3.5 ksi for adhesive anchors (ksi)

A_{Nc} = $(S_1+S_2)\cdot(S_3+S_4)$ Projected concrete breakout area, see Figure 40.16-1

$c_{a,min}$ = Minimum edge distance, see Figure 40.16-1 (in)

h_{ef} = Embedment depth per Table 40.16-1, maximum of $20d_a$ for adhesive anchors (in)

c_{ac} = Critical edge distance, $4.0h_{ef}$ for mechanical anchors and $2.0h_{ef}$ for adhesive anchors (in)

$\Psi_{c,min}$ = 0.375 for mechanical anchors, 0.75 for adhesive anchors

For Type L anchors, in addition to the check above, the factored sustained tension force, $N_{u,s}$, must be less than or equal to the factored sustained tensile resistance, N_{rs}:

$$N_{rs} = \phi_{tc} 0.55 N_{ba}$$

$$N_{ba} = \tau \pi d_a h_{ef}$$

Where:

τ = Adhesive bond stress (ksi)

When anchor reinforcement is provided per 40.16.3, the factored tensile resistance may be increased per **ACI [D.5.2.9]**. The factored tensile resistance N_r shall not exceed $\phi_{ts} A_s f_u$. For adhesive anchors, the factored tensile resistance shall also not exceed the bond strength of the anchor per **ACI [D.5.5]**.

40.16.5 Masonry Anchor Shear Capacity

Masonry anchors in shear fail in steel shear rupture, concrete breakout or concrete pryout. Figure 40.16-2 shows the concrete breakout failure in shear. The concrete pryout is equal to 2 times the factored anchor tensile resistance, N_r.

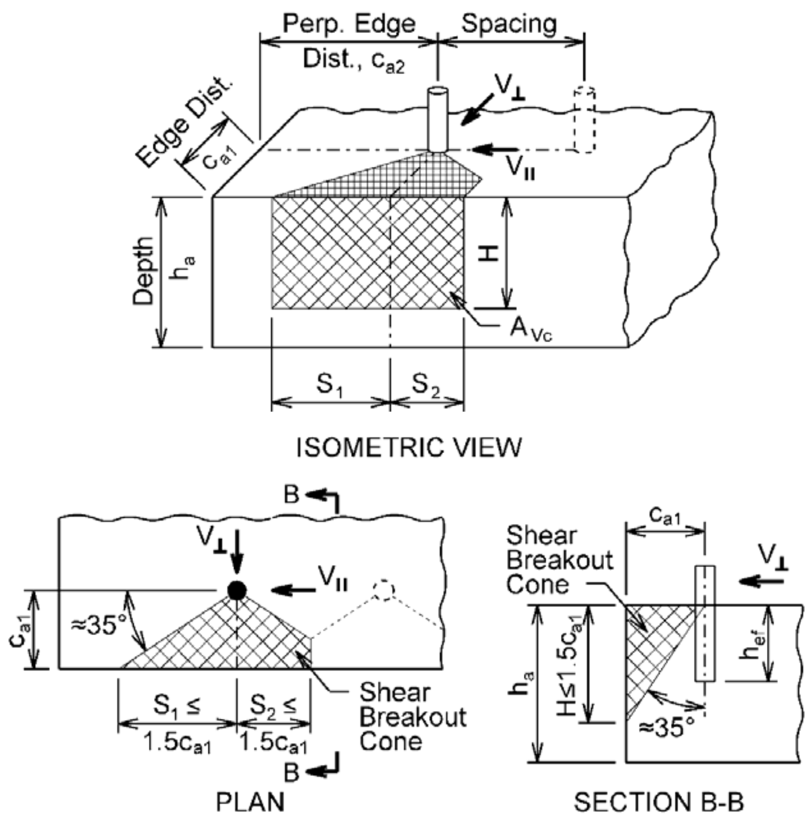
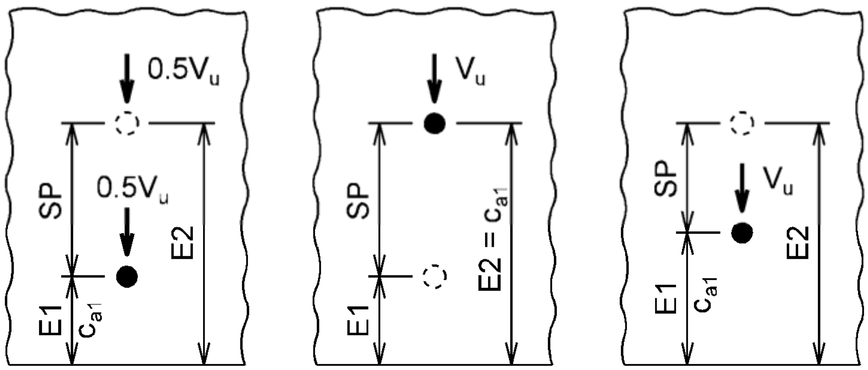


Figure 40.16-2
Concrete Breakout of Masonry Anchors in Shear

The projected concrete breakout area, A_{Vc}, shown in Figure 40.16-2 is limited vertically by H, and in both horizontal directions by S_i:

H = Minimum of the member depth h_a or 1.5 times the edge distance c_{a1} (in)

S_i = Minimum of half the anchor spacing S, the perpendicular edge distance c_{a2}, or 1.5 times the edge distance c_{a1} (in)



Case 1; SP≥E1
c_{a1} = E1

Case 2; SP≥E1
c_{a1} = E2

Case 3; SP<E1
c_{a1} = E1

Figure 40.16-3
Masonry Anchors Shear Force Cases

If the shear is applied to more than one row of anchors as shown in Figure 40.16-3, the shear capacity must be checked for the worst of three cases. If the row spacing SP is at least equal to the distance from the edge to the front anchor E1, check both Case 1 and Case 2. In Case 1, the front anchor is checked with the shear load evenly distributed between the rows of anchors. In Case 2, the far anchor is checked with the full shear load. If the row spacing SP is less than the edge distance E1, then check Case 3, applying the full shear load to the front anchor. If the anchors are welded to an attachment to evenly distribute the force to all anchors, only Case 2 needs to be checked.

Anchor Size, d_a	Minimum Embedment Depth, h_{ef}		Concrete Breakout Strength Factor, v	
	Mechanical in	Adhesive in	Mechanical, v_m	Adhesive, v_a
#4 or 1/2”	2.5	4.0	5.57	4.22
#5 or 5/8”	2.5	5.0	4.98	3.78
#6 or 3/4”	3.0	6.0	4.55	3.52
#7 or 7/8”	3.5	7.0	4.21	3.52
#8 or 1”	4.0	8.0	3.94	3.52
#9 or 1-1/8”	4.5	9.0	3.71	3.52

Table 40.16-2
Shear Design Table for Concrete Masonry Anchors, Type S and Type L

The factored shear force on each anchor, V_u, must be less than or equal to the factored shear resistance, V_r:

$$V_r = \phi_{vc} V_n \frac{A_{Vc}}{4.5(c_{a1})^2} \Psi_{hv} \Psi_{ed} \Psi_{cv} \Psi_{pv} \le \phi_{vs} 0.6 A_s f_u \le \phi_{vp} 2.0 N_n$$

In which:

$$V_n = \frac{\sqrt{f'_c}}{v} c_{a1}^{1.5} \text{ (kips)}$$

$$\Psi_{hv} = \sqrt{\frac{1.5c_{a1}}{h_a}} \ge 1.00$$

Ψ_{ed} = $0.70 + 0.30\dfrac{c_{a2}}{1.5c_{a1}} \le 1.00$ for perpendicular shear, see Figure 40.16-2

= 1.0 for parallel shear

Where:

ϕ_{vc} = 0.70 if no supplementary reinforcement exists to control splitting, and 0.75 if supplementary reinforcement exists per 40.16.3

A_{Vc} = $H \cdot (S_1+S_2)$ Projected concrete breakout area, see Figure 40.16-2 (in^2)

c_{a1} = Edge distance in the direction of the perpendicular shear force, see Figure 40.16-2 and Figure 40.16-3 (in)

Ψ_{cv} = 1.4 for anchors located in a region of a concrete member where analysis indicates no cracking at service loads

= 1.0 when cracking exists at service loads with no supplementary reinforcement per 40.16.3 or with edge reinforcement smaller than a No. 4 bar

= 1.2 when cracking exists at service loads with reinforcement of a No. 4 bar or greater between the anchor and the edge

= 1.4 when cracking exists at service loads with reinforcement of a No. 4 bar or greater between the anchor and the edge, and with stirrups around the edge reinforcement spaced at not more than 4 in.

Ψ_{pv} = 1.0 for shear perpendicular to the edge, and 2.0 for shear parallel to the edge, see Figure 40.16-2

ϕ_{vs} = 0.65 for ductile steel, and 0.60 for brittle steel as defined in 40.16.2

A_s = Effective anchor cross sectional area (in^2)

f_u = Steel tensile strength, not greater than the smaller of $1.9f_y$ and 125 ksi

ϕ_{vp} = 0.65

N_n = Nominal tensile resistance, per 40.16.3 (kips)

f'_c = Concrete strength (ksi)

v = v_m for mechanical anchors, v_a for adhesive anchors per Table 40.16-2

c_{a2} = Perpendicular edge distance, see Figure 40.16-2 (in)

h_a = Depth of the concrete member, see Figure 40.16-2 (in)

For shear in two directions, check both the parallel and the perpendicular shear capacity. For shear on an anchor near a corner, check the shear capacity for both edges and use the minimum.

When anchor reinforcement is provided per 40.16.3, the factored shear resistance may be increased per **ACI [D.6.2.9]**. The factored shear resistance V_r shall not exceed $\phi_{vs}0.6A_sf_u$ or $\phi_{vp}2.0N_n$.

40.16.6 Interaction of Tension and Shear

If $V_u/V_r \leq 0.2$, then the full strength in tension is permitted: $N_u \leq N_r$. If $N_u/N_r \leq 0.2$ then the full strength in shear is permitted: $V_u \leq V_r$. If $N_u/N_r > 0.2$ and $V_u/V_r > 0.2$, then:

$$\frac{N_u}{N_r} + \frac{V_u}{V_r} \leq 1.2$$

40.16.7 Plan Preparation

The required minimum pullout capacity (as stated on the plans) is equal to the Nominal Tensile Resistance, N_n.

<u>Typical notes for bridge plans (shown in all capital letters):</u>

MASONRY ANCHORS TYPE S 5/8-INCH. MIN. PULLOUT CAPACITY OF 17 KIPS. EMBED 6" IN CONCRETE. *(Illustrative only, values must be calculated for the specific situation).*

When using "Type S" anchors to anchor bolts or studs, the bolt, nut, and washer or the stud as detailed on the plans is included in the bid item.

For “Type S” anchors using rebar, the rebar is listed in the “Bill of Bars” and paid for under the bid item “Bar Steel Reinforcement HS (Coated) Bridges”.

MASONRY ANCHORS TYPE L NO. 5 BARS. MIN. PULLOUT CAPACITY OF 17 KIPS. EMBED 1’-0” IN CONCRETE. *(Illustrative only, values must be calculated for the specific situation).*

For “Type L” anchors, the rebar is listed in the “Bill of Bars” and paid for under the bid item “Bar Steel Reinforcement HS (Coated) Bridges”.

It should be noted that AASHTO is considering adding specifications pertaining to concrete masonry anchors. This chapter will be updated once that information is available.

40.17 Plan Details

1. Excavation for Structures on Overlays

   There is considerable confusion on when to use or not use the bid item "Excavation for Structures" on overlay projects. In order to remove the confusion the following note is to be added to all overlay projects that only involve removal of the paving block or less.

   Any excavation required to complete the overlay or paving block at the abutments is to be considered incidental to the bid item "Concrete Masonry Overlay Decks".

   If the overlay project has any excavation other than the need to replace a paving block or prepare the end of the deck for the overlay the "Excavation" bid item should be used and the above note left off the plan.

2. For steel girder bridge deck replacements; show the existing exterior girder size including the lower flange size on the plans to assist the contractor in ordering falsework brackets.

3. On structure deck overlay projects:

   Verify desired transverse or roadway cross slopes with Regions considering the changes in bridge rating and approach pavement grade. Although relatively flat by today's standard of an 0.02'/' cross slope; a cross slope of 0.01'/'/0.015'/' may be the most desirable.

   The designer should evaluate 3 types of repairs. (Preparation Decks Type 1) is concrete removal to the top of the bar steel. (Preparation Decks Type 2) is concrete removal below the bar steel. (Full Depth Deck Repair) is full depth concrete removal and repair. The designer should compute the quantity and cost for each type and a total cost. Compare the total cost to the estimated cost of a deck replacement and proceed accordingly. Show the location of (Full Depth Deck Repair) on the plan sheet.

   Contractors have gotten projects where full depth concrete removal was excessive and no locations designated. It would have been a better decision to replace the deck.

4. When detailing two stage concrete deck construction; consider providing pier cap construction joints to coincide with the longitudinal deck construction joint. Also, transverse deck bar steel lap splicing is preferred over the use of bar couplers. This is applicable where there is extra bridge width and the falsework can be left for both deck pours.

5. Total Estimated Quantities

   For all Bituminous Material Overlays:

Surface Preparation for Sheet Membrane Waterproofing	Area of Deck
Sheet Membrane Waterproofing	Area of Deck
HMA Pavement Type E-xx	Check (E-xx) with Region
Asphaltic Material PGxx-xx	Check (PGx-xx) with Region
If Asked for in Structure Survey Report	
Preparation Decks Type 1 & 2	If Blank, call Region
Concrete Masonry, Deck Patching	Use 1/2 Slab Thickness
Sawing Pavement, Deck Preparation, Curb or Joint Repair	If asked for by Region

Table 40.17-1
Quantities for Asphaltic Overlays

40.18 Retrofit of Steel Bridges

Out of plane bending stresses may occur in steel bridges that introduce early fatigue cracks or worse yet brittle fractures. Most, if not all, problems are caused at connections that are either too flexible or too rigid. It is important to recognize the correct condition as retrofitting for the wrong condition will probably make it worse.

40.18.1 Flexible Connections

A connection is too flexible when minor movement can occur in a connection that is designed to be rigid. Examples are transverse or bearing stiffeners fitted to a tension flange, floorbeams attached to the web only and not both flanges.

The solution for stiffeners and transverse connection plates is to shop weld the stiffener to the web and flanges. This becomes a Category C weld but is no different than the terminated web to stiffener weld. If the condition exists in the field, welding is not recommended. Retrofit is to attach a T-section with four bolts to the stiffener and four bolts to the flange. Similar details should be used in attaching floorbeams to the girder.

40.18.2 Rigid Connections

A connection is too rigid when it is fitted into place and allowed to move but the movement can only occur in a refined area which introduces high stresses in the affected area. Examples are welded gusset connection plates for lower lateral bracing that are fitted around transverse or bearing stiffeners.

Other partial constraint details are:

1. Intersecting welds
2. Gap size-allowing local yielding
3. Weld size
4. Partial penetration welds versus fillet welds
5. Touching and intersecting welds

The solution is to create spaces large enough (approximately ¼” or more) for more material to flex thus reducing the concentration of stress. For gusset connection plates provide a larger gap than ¼” and no intersecting welds. For existing conditions it may be necessary to drill holes at high stress concentrations. For new conditions it would be better to design a rigid connection and attach to the flange rather than the web. For certain situations a fillet weld should be used over a partial penetration weld to allow slight movement.

40.19 Reinforcing Steel for Deck Slabs on Girders for Deck Replacements

Effective Span Ft-In	T=Slab Thickness Inches	Transverse Bars & Spacing	Longitudinal Bars & Spacing	Longitudinal* Continuity Bars & Spacing
4-0 4-3	6.5 6.5	#5 @ 8" #5 @ 7.5"	#4 @ 8.5" #4 @ 7.5"	#5 @ 7.5" #5 @ 7.5"
4-6 4-9 5-0	6.5 6.5 6.5	#5 @ 7.5" #5 @ 7" #5 @ 6.5"	#4 @ 7.5" #4 @ 7.5" #4 @ 7"	#5 @ 7.5" #5 @ 7.5" #5 @ 7"
5-3 5-6 5-9	6.5 6.5 6.5	#5 @ 6.5" #5 @ 6" #5 @ 6"	#4 @ 7" #4 @ 6.5" #4 @ 6.5"	#5 @ 7" #5 @ 6.5" #5 @ 6.5"
2-3 2-6 2-9	7 7 7	#4 @ 9" #4 @ 8.5" #4 @ 8"	#4 @ 11" #4 @ 11" #4 @ 11"	#5 @ 6.5" #5 @ 6.5" #5 @ 6.5"
3-0 3-3 3-6	7 7 7	#4 @ 7.5" #4 @ 7" #4 @ 6.5"	#4 @ 11" #4 @ 11" #4 @ 11"	#5 @ 6.5" #5 @ 6.5" #5 @ 6.5"
3-9 4-0 4-3	7 7 7	#4 @ 6.5" #4 @ 6" #5 @ 9"	#4 @ 10.5" #4 @ 10" #4 @ 9.5"	#5 @ 6.5" #5 @ 6.5" #5 @ 7"
4-6 4-9 5-0	7 7 7	#5 @ 8.5" #5 @ 8" #5 @ 8"	#4 @ 9" #4 @ 8.5" #4 @ 8.5"	#5 @ 7" #5 @ 7" #5 @ 7"
4-3 5-6 5-9	7 7 7	#5 @ 7.5" #5 @ 7' #5 @ 7"	#4 @ 8" #4 @ 7" #4 @ 7"	#5 @ 7" #5 @ 7" #5 @ 7"
6-0 6-3 6-6	7 7 7	#5 @ 6.5" #5 @ 6.5" #5 @ 6"	#4 @ 7" #4 @ 7" #4 @ 6.5"	#5 @ 7" #5 @ 7" #5 @ 6.5"
6-9 7-0	7 7	#5 @ 6" #5 @ 6"	#4 @ 6.5" #4 @ 6"	#5 @ 6.5" #5 @ 6"
4-0 4-3	7.5 7.5	#4 @ 7" #4 @ 6.5"	#4 @ 10.5" #4 @ 10.5"	#5 @ 6" #5 @ 6"
4-6 4-9 5-0	7.5 7.5 7.5	#4 @ 6.5" #4 @ 6" #5 @ 9"	#4 @ 10" #4 @ 10" #4 @ 9.5"	#5 @ 6" #5 @ 6" #5 @ 6"

5-3	7.5	#5 @ 8.5"	#4 @ 9"	#5 @ 6.5"
5-6	7.5	#5 @ 8"	#4 @ 8.5"	#5 @ 6.5"
5-9	7.5	#5 @ 8"	#4 @ 8.5"	#5 @ 6.5"
6-0	7.5	#5 @ 7.5"	#4 @ 8"	#5 @ 6.5"
6-3	7.5	#5 @ 7.5"	#4 @ 7.5"	#5 @ 6.5"
6-6	7.5	#5 @ 7"	#4 @ 7.5"	#5 @ 6.5"
6-9	7.5	#5 @ 7"	#4 @ 6.5"	#5 @ 6.5"
7-0	7.5	#5 @ 7"	#4 @ 6.5"	#5 @ 6.5"
7-3	7.5	#5 @ 6.5"	#4 @ 6.5"	#5 @ 6.5"
7-6	7.5	#5 @ 6.5"	#5 @ 10"	#5 @ 6.5"
7-9	7.5	#5 @ 6"	#5 @ 10"	#5 @ 6.5"
8-0	7.5	#5 @ 6"	#5 @ 10"	#5 @ 6.5"
8-3	7.5	#5 @ 6"	#5 @ 9.5"	#5 @ 6.5"

Table 40.19-1

Reinforcing Steel for Deck Slabs on Girders for Deck Replacements-HS20 Loading

Max. Allowable Design Stresses: f_c' = 4000 psi, f_y = 60 ksi, Top Steel 2 1/2" Clear, Bottom Steel 1-1/2" Clear, 20 lbs/ft. Future Wearing Surface. Transverse and longitudinal bars shown in table are for one layer only. Place identical steel in both top and bottom layer, except in negative moment region. "Use in top layer for slab on steel girders in negative moment region when not designed for negative moment composite action.

40.20 References

1. *A Study of Policies for the Protection, Repair, Rehabilitation, and Replacement of Concrete Bridge Decks* by P.D. Cady, Penn. DOT.

2. *Concrete Sealers for Protection of Bridge Structures, NCHRP Report 244*, December, 1981.

3. *Durable Bridge Decks* by D. G. Manning and J. Ryell, Ontario Ministry Transportation and Communications, April, 1976.

4. *Durability of Concrete Bridge Decks, NCHRP Report 57*, May, 1979.

5. *Rehabilitation and Replacement of Bridges on Secondary Highways and Local Roads, NCHRP Report 243*, December, 1981.

6. *Strength of Concrete Bridge Decks* by D. B. Beal, Research Report 89 NY DOT, July, 1981.

7. *Latex Modified Concrete Bridge Deck Overlays - Field Performance Analysis* by A. G. Bisharu, Report No. FHWA/OH/79/004, October, 1979.

8. *Standard Practice for Concrete Highway Bridge Deck Construction by ACI Committee 345, Concrete International*, September, 1981.

9. *The Effect of Moving Traffic on Fresh Concrete During Bridge Deck Widening* by H. L. Furr and F. H. Fouad, Paper Presented 61 Annual TRB Meeting, January, 1982.

10. *Control of Cracking in Concrete Structures by ACI Committee 224*, Concrete International, October, 1980.

11. *Discussion of Control of Cracking in Concrete Structures* by D. G. Manning, Concrete International, May, 1981.

12. *Effects of Traffic-Induced Vibrations on Bridge Deck Repairs, NCHRP Report 76*, December, 1981.

13. *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary,* 2011.

$$c := \frac{A_{ps} \cdot f_{pu} - 0.85 \cdot f'_{cd} \cdot (b - b_{tf}) \cdot h_f}{0.85 \cdot f'_{cd} \cdot \beta_1 \cdot b_{tf} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}} \qquad \boxed{c = 10.937} \text{ in}$$

$$a := \beta_1 \cdot c \qquad \boxed{a = 9.30} \text{ in}$$

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 [5.7.3.2]**:

$$M_n := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + 0.85 \cdot f'_{cd} \cdot (b - b_{tf}) \cdot h_f \cdot \left(\frac{a}{2} - \frac{h_f}{2}\right)\right] \cdot \frac{1}{12}$$

$$\boxed{M_n = 15717} \text{ kip-ft}$$

For prestressed concrete, $\phi_f := 1.00$, **LRFD [5.5.4.2.1]**. 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 [5.7.3.3.2]**

$$\gamma_{LL} := 1.75 \qquad \gamma_{DC} = 1.250 \qquad \eta := 1.0$$

$$Mu := \eta \cdot \left[\gamma_{DC} \cdot (M_{DC1} + M_{DC2}) + \gamma_{LL} \cdot M_{LLIM}\right] \qquad \boxed{Mu = 11832} \text{ kip-ft}$$

$$\boxed{1.33 \cdot Mu = 15737} \text{ kip-ft}$$

Calculate M_{cr} next and compare its value with 1.33 Mu

M_{cr} is calculated as follows:

$f_r := 0.37 \cdot \sqrt{f'_c}$ **LRFD [5.4.2.6]** $\boxed{f_r = 1.047}$ ksi

$$f_{cpe} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_b}$$ $\boxed{f_{cpe} = 4.414}$ 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

$$M_{cr} := \gamma_3 \cdot \left[S_c \cdot \left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) \cdot \frac{1}{12} - M_{dnc} \cdot \left(\frac{S_c}{S_{nc}} - 1 \right) \right]$$ $\boxed{M_{cr} = 11921}$ kip-ft

$M_{cr} = 11921$ 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 [5.9.5.2]**

$T_{oi} := ns \cdot f_{tr} \cdot A_s$ $\boxed{= 46 \cdot 202.5 \cdot 0.217 = 2021}$ 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