Table of Contents

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

Hydraulic Data

100 YEAR FREQUENCY

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

2 YEAR FREQUENCY

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

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

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

(See Chapter 8 – Hydraulics for additional information)

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

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

6.2.2.4 Utilities

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

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

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

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

6.2.3 Distribution of Exhibits

6.2.3.1 Federal Highway Administration (FHWA).

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

1. Projects on the Interstate System
2. Projects on the National Highway System (NHS) but not on the Interstate System, unless it is determined by FHWA and WisDOT that the responsibilities can be assumed by WisDOT
3. Projects on non-NHS Federal-aid highways, and eligible projects on public roads which are not Federal-aid highways if WisDOT determines that it is not appropriate for WisDOT to assume the responsibilities

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

Unusual bridges have the following characteristics:

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

Examples of unusual bridges:

- Cable-stayed
- Suspension
- Arch
- Segmental concrete
- Movable

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

Examples of unusual structures:

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

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

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

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

12. Information on substructure and foundation types

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

6.2.3.2 Other Agencies

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

- Department of Natural Resources

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

- Railroad (FDM Chapter 17)

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

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

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

- Coast Guard (FDM)

- Regions

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

- Native American Tribal Governments

- Corps of Engineers

- Other governing municipalities

- State Historic Preservation Office

- Environmental Protection Agency

- Other DOTs

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. Wings should be numbered starting from the lower left corner and increasing in a clockwise order.

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

2. Elevation

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

    b. Give vertical dimension of wing.

3. Wing Elevation

4. Body Section

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

5. Wing Sections

6. Bar Steel Listing and Detail

Use the following views where necessary:

7. Pile Plan & Splice Detail

8. View Showing Limits of Excavation and Backfill

9. Special Details for Utilities

10. Drainage Details

6.3.2.4 Piers

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

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

1. Plan View

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

2. Elevation

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

3. Footing Plan

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

4. Bar Steel Listing and Details

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

6. Cross Section thru Column and Pier Cap

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

6.3.2.5 Superstructure

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

6.3.2.5.1 All Structures

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

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

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

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

Indicate girder numbers about the centerline of bearing in each span. Girders should be numbered in increasing order from left to right in the cross-section view. For rehabilitation projects, indicate the existing girder numbers and assign new girder numbers in increasing order from left to right.

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

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

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

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

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

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

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

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

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

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

6.3.2.5.2 Steel Structures

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

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

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

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

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

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

6.3.2.5.3 Railing and Parapet Details

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

6.3.3 Miscellaneous Information

6.3.3.1 Bill of Bars

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

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

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

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

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

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

6.3.3.2 Box Culverts

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

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

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

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

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

6.3.3.3 Miscellaneous Structures

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

6.3.3.4 Standard Drawings

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

6.3.3.5 Insert Sheets

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

6.3.3.6 Change Orders and Maintenance Work

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

6.3.3.7 Name Plate and Bench Marks

WisDOT has discontinued the statewide practice of furnishing bench mark disks and requiring them to be placed on structures. However, WisDOT Region Offices may continue to provide bench mark disks for the contract to be set. When requested, bench mark disks shall be shown on bridge and larger culvert plans. Locate the bench mark disks on a horizontal surface flush with the concrete. Bench marks to be located on top of the parapet on the bridge deck, above the first right corner of the abutment traveling in the highway cardinal directions of North or

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

6.3.4 Checking Plans

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

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

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

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

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

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

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

1. QC/QA sign-off sheet

2. Design computations and computer runs

3. Quantity computations

4. Bridge Special Provisions and STSP's (only those STSP's requiring specific blanks to be filled in or contain project specific information)

5. Final Structure Survey Report form (not including photos, cross-sections, project location maps, etc.)

6. Final Geotechnical Report

7. Final Hydrology and Hydraulic computations and structure sizing report

8. Contour map

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

1. Miscellaneous correspondence and transmittal letters

2. Preliminary drawings and computations

3. Prints of soil borings and plan profile sheets

4. Shop steel quantity computations*

5. Design checker computations

6. Layout sheets

7. Elevation runs and bridge geometrics

8. Falsework plans*

9. Miscellaneous Test Report

10. Photographs of bridge rehabs

* These items are added to the packet during construction.

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

1. All "void" material

2. All copies except one of preliminary drawings

3. Extra copies of plan and profile sheets

4. Preliminary computer design runs

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

6.4 Computation of Quantities

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

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

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

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

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

6.4.1 Excavation for Structures Bridges (Structure)

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

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

6.4.2 Granular Materials

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

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

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

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

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

Refer to 9.10 for additional information about granular materials.

6.4.3 Concrete Masonry Bridges

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

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

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

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

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

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

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

6.4.6 Bar Steel Reinforcement HS Stainless Bridges

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

6.4.7 Structural Steel Carbon or Structural Steel HS

See 24.2.4.

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

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

6.4.9 Piling Test Treated Timber (Structure)

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

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

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

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

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

6.4.11 Preboring CIP Concrete Piling or Steel Piling

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

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

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

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

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

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

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

Record this quantity to the nearest 5 cubic yards.

6.4.15 Pile Points

When recommended in soils report. Bid as each.

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

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

6.4.17 Cofferdams (Structure)

Lump Sum

6.4.18 Rubberized Membrane Waterproofing

Record the quantity to the nearest square yard.

6.4.19 Expansion Devices

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

6.4.20 Electrical Work

Refer to Standard Construction Specifications for bid items.

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

Record this quantity in feet.

6.4.22 Preparation Decks Type 1 or Preparation Decks Type 2

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

6.4.23 Cleaning Decks

Record this quantity to the nearest square yard.

6.4.24 Joint Repair

Record this quantity to the nearest square yard.

6.4.25 Concrete Surface Repair

Record this quantity to the nearest square foot.

6.4.26 Full-Depth Deck Repair

Record this quantity to the nearest square yard.

6.4.27 Concrete Masonry Overlay Decks

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

6.4.28 Removing Old Structure STA. XX + XX.XX

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

6.4.29 Anchor Assemblies for Steel Plate Beam Guard

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

6.4.30 Steel Diaphragms (Structure)

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

6.4.31 Welded Stud Shear Connectors X -Inch

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

6.4.32 Concrete Masonry Seal

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

6.4.33 Geotextile Fabric Type

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

6.4.34 Concrete Adhesive Anchors

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

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

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

6.4.36 Piling Steel Sheet Temporary

This quantity is used when the designer determines that retention of earth is necessary during excavation and soil forces require the design of steel sheet piling. This item is seldom used now that railroad excavations have a unique SPV.

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

6.4.37 Temporary Shoring

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

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

6.4.38 Concrete Masonry Deck Repair

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

6.4.39 Sawing Pavement Deck Preparation Areas

Use 10 lineal feet per SY of Preparation Decks Type 1.

6.4.40 Removing Bearings

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

6.4.41 Ice Hot Weather Concreting

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

6.4.42 Asphaltic Overlays

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

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

Coordinate asphaltic quantity assumptions with the Region and roadway designers.

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

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

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

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

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

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

6.5.1 Approvals, Distribution, and Work Flow

Consultant	Meet with Regional Office and/or local units of government to determine need.
	Prepare Structure Survey Report including recommendation of structure type.
Geotechnical Consultant	Make site investigation and prepare Subsurface Investigation Report.

Consultant	Submit hydrology report via Esubmit or as an email attachment to the supervisor of the Consultant Review and Hydraulics Unit. Submit 60 days prior to preliminary plan submittal.
	Prepare preliminary plans according to 6.2.
	Coordinate with Region and other agencies per 6.2.3.
	Submit preliminary plans, SSR and supporting documents via e-submit for review and approval of type, size and location.
Structures Design Section	Record project information in HSIS.
	Review hydraulics for Stream Crossings.
	Review Preliminary Plan. A minimum of 30 days to review preliminary plans should be expected.
	Coordinate with other agencies per 6.2.3.
	Return preliminary plans and comments from Structures Design Section and other appropriate agencies to Consultant with a copy to the Regional Office.
	Forward Preliminary Plan and Hydraulic Data to DNR.
Consultant	Modify preliminary plan as required, and provide explanation for preliminary comments not incorporated in final plan.
	Prepare and complete final design and plans for the specified structure.
	Write special provisions.
	At least **two months** in advance of the PS&E date, submit the required final design documents via e-submit per 6.5.3.
Structures Design Section	Determine which final plans will be reviewed and perform quality assurance review as applicable.
	For final plans that are reviewed, return comments to Consultant and send copy to Regional Office, including FHWA as appropriate.
Consultant	Modify final plans and specifications as required.
	Submit modified final plans via e-submit as required.
Structures Design Section	Review modified final plans as applicable.
	Sign final plans and send performance evaluation form to Region and Consultant.
Bureau of Project Development	Prepare final accepted structure plans for pre-development contract administration.

Consultant	If a plan change is needed after being advertised but before being let, an addendum is required per FDM 19-22-1 and 19.1 Attachment 1.2.
Structures Design Section	Review structure addendum as applicable.
	Sign structure addendum.
Bureau of Project Development	Distributes structure addendum to bidders.
Consultant	If a plan change is required after being let, a post-let revision is required per 6.5.5.
Structures Design Section	Review post-let revision as applicable.
	Stamp post-let revision plan as accepted.
	Delivers revised plan to DOT construction team for distribution.

Table 6.5-1
Approvals, Distribution and Work Flow

6.5.2 Preliminary Plan Requirements

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

1. Hydrology Report
2. Structure Survey Report
3. Preliminary plan, including log borings shown on the subsurface exploration sheet
4. Evaluation of subsurface investigation report
5. Contour map
6. Plan and profile, and typical section for roadway approaches
7. Hydraulic/Sizing Report (see Chapter 8 - Hydraulics) and hydraulic files are required for stream crossing structures
8. County map showing location of new and/or existing structures and FEMA map
9. Any other information or drawings which may influence location, layout or design of structure, including DNR initial review letter and photographs

6.5.3 Final Plan Requirements

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

1. Final Drawings

2. Design and Quantity Computations

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

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

4. QA/QC Verification Sheet

5. Inventory Data Sheet

6. Bridge Load Rating Summary Form

7. LRFD Input File (Excel ratings spreadsheet)

8. On-Time Improvement Form

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

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

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

6.5.4 Addenda

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

6.5.5 Post-Let Revisions

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

6.5.6 Local-Let Projects

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

6.6 Structures Data Management and Resources

6.6.1 Structures Data Management

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Table 6.6-1
Various Inspection Reports

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

6.6.2 Resources

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

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

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

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

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

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

http://bridges.transportation.org

https://www.arema.org/

Table of Contents

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.

7.1.4.1.1 Precast Piers

Precast concrete piers are optimally used when constructed adjacent to traffic. This application can be best visualized for a two span bridge with a pier located between median barriers. The use of precast piers minimizes traffic disruptions and construction work near traffic.

7.1.4.1.2 Application

Precast concrete piers have successfully been used on past projects. However, these projects did not allow the use of cast-in-place concrete piers which is currently not practical for most projects. An approach that allows for either cast-in-place or precast construction (or a combination thereof) after the contract has been awarded provides contractors greater flexibility to meet schedule demands, provides a safer work environment, and has the potential to reduce costs.

Optional precast concrete pier elements are currently being used on the I-39/90 Project. To aid in the continued development of precast piers, several bridges on the I-39/90 Project required the use of precast pier elements. These mandatory locations will follow the optional precast pier requirements, but prohibit cast-in-place construction. The remaining I-39/90 Project bridges, unless provided an exception, are being delivered as traditional cast-in-place piers with a noted allowance for the contractor to select a precast option. The precast option provides the Project Team and contractors with more flexibility while requiring minimal coordination with designers and the Bureau of Structures.

WisDOT policy item:

Pier configurations shall be determined by providing the most efficient cast-in-place concrete pier design, unless approved otherwise. When the cast-in-place design can accommodate a precast option, include a noted allowance. See Standards for Precast Pier (Optional) Cap and Columns. Contact the Bureau of Structures Development Section for further guidance.

In some cases, optional precast piers may not be suitable for a particular bridge location and there are specific limitations that can cause concern. The designer shall investigate the potential viability of precast pier elements for any proposed bridge. The designer should be aware of the common criteria for use and the limitations of the pier system. Some specific limitations for the optional precast pier element usage are the following:

- Piers shall be designed to allow either cast-in-place or precast concrete construction, but with only cast-in-place detailed on the plans. Differences between construction methods shall be limited to pier column connections, beam seats details, and diaphragm details. If the pier configuration is not able to reasonably accommodate interchangeability between the two construction types, optional piers may be exempt from the precast option.
- Multi-column piers (3x4 ft rectangular) grade separations over roadways only.
- Fixed piers supporting prestressed concrete girders only.
- Precast elements shall be limited to 90 kips.
- Deep foundations are recommended when multiple pier caps are used. Shallow foundations may be considered if differential settlement is not expected.
- Integral barriers or crashwalls are currently excluded from the precast option.
- Applications where the top of the footing may become submerged are prohibited.

An exception to the precast pier option may be given by the Bureau of Structures.

7.1.4.1.3 Design Considerations

Precast concrete piers shall be designed in conformance with the current *AASHTO LRFD*, in accordance with the WisDOT Bridge Manual, and as given in the Special Provisions.

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:

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

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

9.2 Concrete

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

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

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

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

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

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

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

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

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

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

9.3 Reinforcement Bars

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

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

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

At all abutments, use epoxy coated bars in the parapets and in the wing walls. For A3 abutments, use epoxy coated bars in the paving block and in the abutment backwall. For A1(fixed) abutments, use epoxy coated dowel bars.

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

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

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

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

When determining the anchorage requirements for bars, consider the bar size, the development length for straight bars and the development length for standard hooks. Note in Table 9.9-1 and Table 9.9-2 that smaller bars require considerably less development length than larger bars and the development length is also less if the bar spacing is 6 inches or more. By detailing smaller bars to get the required area and providing a spacing of 6 inches or more, less steel is used. Bar hooks can reduce the required bar development lengths, however the hooks may cost more to fabricate. In cases such as footings for columns or retaining walls, hooks may be the only practical solution because of the concrete depth available for developing the capacity of the bars.

Fabricators stock all bar sizes in 60 foot lengths. For ease of handling, the detailed length for #3 and #4 bars is limited to 45 feet. Longer bars may be used for these bar sizes at the discretion of the designer, when larger quantities are required for the structure. All other bar sizes are detailed to a length not to exceed 60 feet, except for vertical bars. Bars placed in a vertical position are detailed to match optional construction joint spacing plus lap. The location of optional horizontal construction joints in pier shafts or columns will generally determine the length of vertical bars in piers. All bars are detailed to the nearest inch.

The number of bars in a bundle shall not exceed four, except in flexural members the bars larger than #11 shall not exceed two in any one bundle. Individual bars in a bundle, cut off within the span of a member, shall be terminated at different points with at least a 40-bar diameter stagger. Where spacing limitations are based on bar size, bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area **LRFD [5.10.3.1.5]**.

Stainless steel deformed reinforcement meeting the requirements of ASTM A955 has been used on a limited basis with the approval of the Bureau of Structures. It has been used in bridge decks, parapets and in the structural approach slabs at the ends of the bridge. Fabricators typically stock #6 bars and smaller in 60 foot lengths and #7 bars and larger in 40 foot lengths. Follow the guidance above for selecting bar lengths based on ease of handling.

9.3.1 Development Length and Lap Splices for Deformed Bars

Table 9.9-1 and Table 9.9-2 provide the development length, ℓ_d , for straight bars and the required lap length of spliced tension bars according to **LRFD [5.11.2.1, 5.11.5.3]**. The basic development length, ℓ_{db} , is a function of bar area, A_b , bar diameter, d_b , concrete strength, f'_c and yield strength of reinforcement, f_y. The basic development length is multiplied by applicable modification factors to produce the required development length, ℓ_d. The lap lengths for spliced tension bars are equal to a factor multiplied times the development length, ℓ_d . The factor applied depends on the classification of the splice; Class A, B or C. The class selected is a function of the numbers of bars spliced at a given location and the ratio of the area of reinforcement provided to the area required. The values for development length (required embedment) are equal to Class "A" splice lengths shown in these tables. Table 9.9-1 gives the development lengths and required lap lengths for a concrete compressive strength of f'_c = 3.5 ksi and a reinforcement yield strength of f_y = 60 ksi. Table 9.9-2 gives these same lengths for a concrete compressive strength of f'_c = 4 ksi and a reinforcement yield strength of f_y = 60 ksi. In tensile stress zones the maximum allowable change in bar size at a lap is three bar sizes. The spacing of lap splice reinforcement is provided in **LRFD [5.10.3.1.4]**, but values on Standards should be used where provided.

The development length of individual bars within a bundle, shall be that for the individual bar, increased by 20% for a three-bar bundle and by 33% for a four-bar bundle **LRFD [5.11.2.3]**. For determining the modification factors specified in **LRFD [5.11.2.1.2, 5.11.2.1.3]**, a unit of bundled bars shall be treated as a single bar of a diameter determined from the equivalent total area.

Lap splices within bundles shall be as specified in **LRFD [5.11.2.3]**. Individual bar splices within a bundle shall not overlap. Entire bundles shall not be lap spliced **LRFD [5.11.5.2.1]**.

Hook and embedment requirements for transverse (shear) reinforcement are stated in **LRFD [5.11.2.6.2]**. The lap length for pairs of U-stirrups or ties that are placed to form a closed unit shall be considered properly anchored and spliced where lengths of laps are not less than 1.7 ℓ_d **LRFD [5.11.2.6.4]**. In members not less than 18 inches deep, the length of the stirrup leg for anchoring closed stirrup splices is described in **LRFD [5.11.2.6.4]**.

The Bureau of Structures interprets the lap length to be used for temperature and distribution reinforcement to be a Class "A" splice (using "top" or "others", as appropriate). See Table 9.9-1 and Table 9.9-2 for definition of "top" bars.

The required development length, ℓ_{dh}, for bars in tension terminating in a standard hook is detailed in **LRFD [5.11.2.4]**. This length increases with the bar size. The basic development length, ℓ_{hb}, for a hooked bar is a function of bar diameter, d_b, and concrete strength, f'_c. The basic development length is multiplied by applicable modification factors to produce the required development length, ℓ_{dh}. Figure 9.9-2 shows typical development lengths for standard hooks in tension.

Embedment depth is increased for dowel bars (with hooked ends) that run from column or retaining wall into the footing, if the hook does not rest on top of the bar steel mat in the bottom of the footing. This is a construction detail which is the preferred method for anchoring these bars before the concrete is placed.

Dowel bars are used as tensile reinforcement to tie columns or retaining walls to their footings. The amount of bar steel can be reduced by varying the dowel bar lengths projecting above the footing, so that only half the bars are spliced in the same plane. This is a consideration for long retaining walls and for some columns. This allows a Class "B" splice to be used, as opposed to a Class "C" splice where equal length dowel bars are used and all bars are spliced in the same plane.

The length of lap, ℓ_c , for splices in compression is provided in **LRFD [5.11.5.5.1]**.

9.3.2 Bends and Hooks for Deformed Bars

Figure 9.9-1 shows standard hook and bend details for development of longitudinal tension reinforcement. Figure 9.9-3 shows standard hook and bend details for transverse reinforcement (open stirrups and ties). Figure 9.9-4 shows details for transverse reinforcement (closed stirrups). Dimensions for the bending details are shown as out to out of bar, as stated in the *Standard Specifications* Section 505.3.2. The diameter of a bend, measured on the inside of the bar for a standard bend is specified in **LRFD [5.10.2.3]**. Where a larger bend radius is required (non-standard bend) show the inside bend radius on the bar detail. When computing total bar lengths account for the accumulation in length in the bends. Use the figures mentioned above to account for accumulation in length for standard hooks and bends. One leg of bent bars is not dimensioned so that the tolerance for an error in the length due to bending is placed there. Fabrication tolerances for bent bars are specified in the *Concrete Reinforcing Steel Institute (CRSI) Manual of Standard Practices* or the *American Concrete Institute (ACI) Detailing Manual* as stated in Section 505.2.1 of the *Standard Specifications*.

Figure 9.3-1 shows typical detailing procedures for bars with bends.

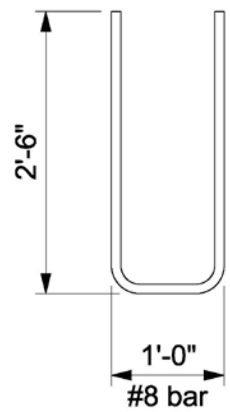


Figure 9.3-1
Bar Bend Detail (#8 bar)

Bar length = 1.0 ft + (2)(2.5 ft) – (2)(0.21 ft) = 5.58 ft or 5'-7" (to the nearest inch)

Where (0.21 ft) is (2.5"/12) and is the standard bar bend deduction found in Figure 9.9-1 for a #8 bar bent 90°.

9.3.3 Bill of Bars

Figure 9.9-5 shows a sample Bill of Bars table for a concrete slab. Different bar letter designations are used for abutments, slabs, and culverts, etc. If bundled bars are used, place a symbol adjacent to the bar mark of the bundled bars and a note below the Bill of Bars table stating the symbol represents bars to be bundled. A column for Bar Series is included if bars are cut.

9.3.4 Bar Series

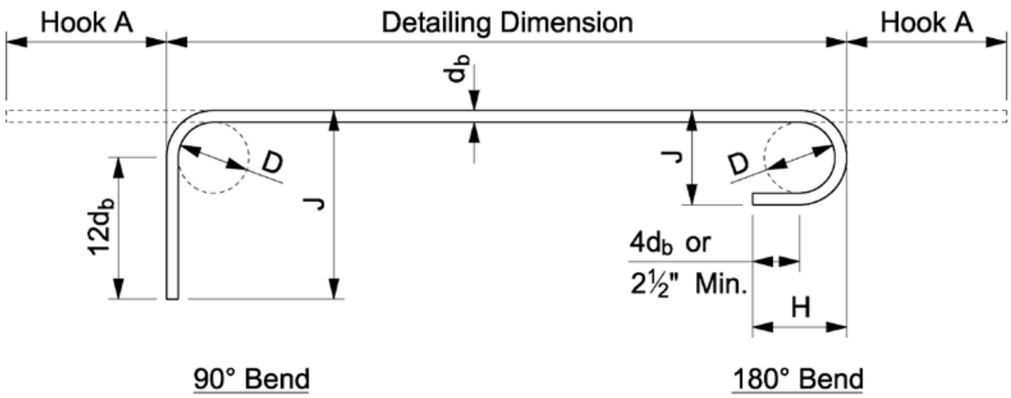
A Bar Series table enables the detailer to detail bar steel in the simplest manner if it is used properly. Also, it helps the fabricator to prepare the Bill of Bars table.

The following general rules apply to the Bar Series table:

- Equal spacing of bars is required.
- There may be more than 1 Series with same number of bars.
- The total length of a bar is 60 feet (maximum).
- The minimum number of bars per Series is 4.

- Bent bars are bent after cutting.
- Set numbers are assigned to each Series used.

Figure 9.9-6Figure 9.9-6 provides a sample layout for a Bar Series table. The Bill of Bars table will show the number of bars and the average bar length in the Series.



d_b = nominal diameter of reinforcing bar (in)

Definition of standard hooks **LRFD [5.10.2.1, C5.11.2.4.1]**

MINIMUM BEND DIAMETER (D) – **LRFD [5.10.2.3]**

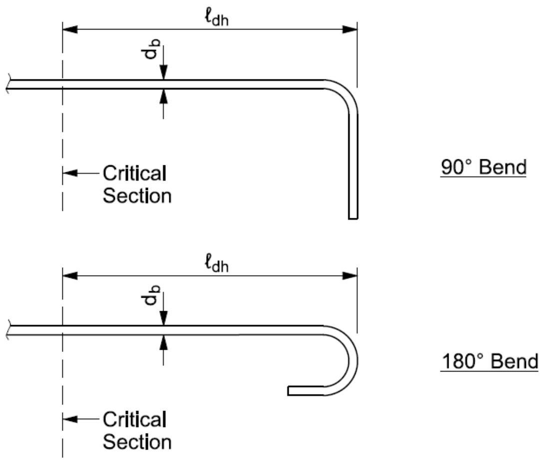
D = $6d_b$ FOR #3 THRU #8

D = $8d_b$ FOR #9, #10, and #11

BAR SIZE	MINIMUM HOOK, ALL GRADES					
	90° HOOKS			180° HOOKS		
	HOOK A	J	J MINUS HOOK A [1]	HOOK A	J	APPROX. H
4	7	8	1	6	4	4 ½
5	8 ½	10	1 ½	7	5	5
6	10	1-0	2	8	6	6
7	1-0	1-2	2	10	7	7
8	1-1 ½	1-4	2 ½	11	8	8
9	1-4	1-7	3	1-3	11 ¼	10 ¼
10	1-6	1-9 ½	3 ½	1-5	1-0 ¾	11 ½
11	1-8	2-0	4	1-7	1-2 ¼	1-0 ¾

Figure 9.9-1
Standard Hooks and Bends for Deformed Longitudinal Reinforcement

[1] "J" MINUS "HOOK A" = DEDUCTION FOR ONE BEND



f'c=3.5 ksi; fy=60 ksi		
Bar Size	ℓ_{dh}	
	Uncoated $\ell_{hb}(0.7)$	Epoxy $\ell_{hb}(0.7)(1.2)$
3	0' - 6"	0' - 7"
4	0' - 8"	0' - 9"
5	0' - 9"	0' - 11"
6	0' - 11"	1' - 1"
7	1' - 1"	1' - 3"
8	1' - 3"	1' - 6"
9	1' - 5"	1' - 8"
10	1' - 7"	1' - 10"
11	1' - 9"	2' - 1"

Figure 9.9-2
Development Length for Standard Hooks in Tension
(See Figure 9.9-1 for bend details)

The development length for standard hooks in tension, ℓ_{dh}, shall not be less than the product of the basic tension development length, ℓ_{hb}, and the appropriate modification factor(s), λ_i, $8d_b$, or 6-inches. The following equation is for the required development length for standard hooks in tension (in):

$\ell_{dh} = \max(\ell_{hb}\,\lambda_i,\ 8d_b,\ 6.0)$ **LRFD [5.11.2.4.1]**

Where:

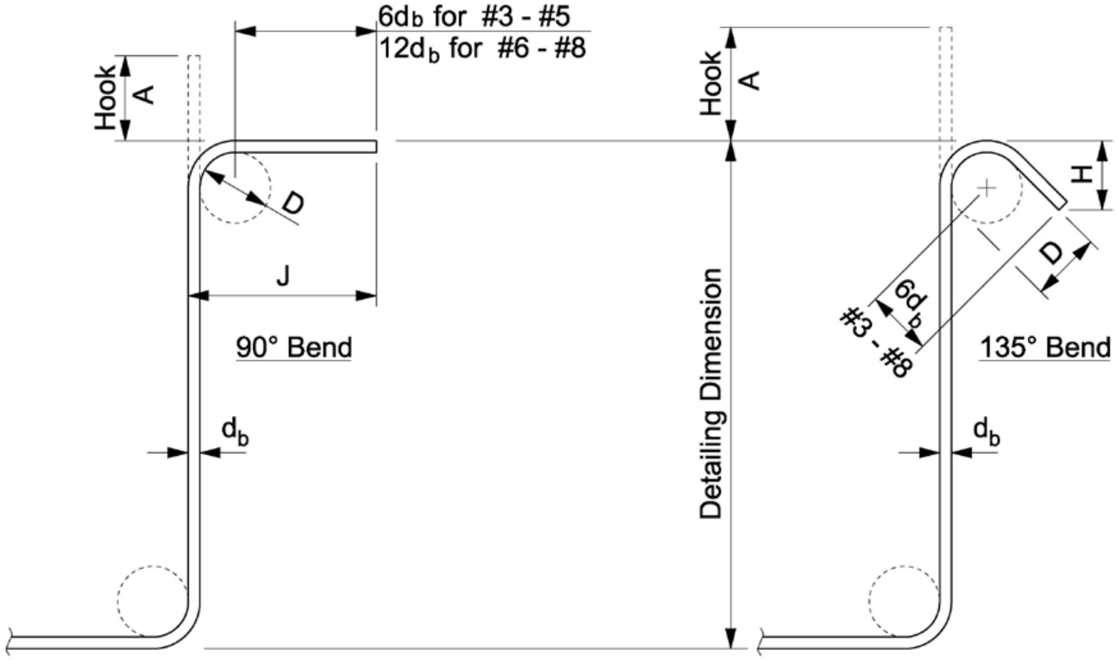
$\ell_{hb} = 38d_b / (f'_c)^{1/2}$ = basic hook development length (in.) **LRFD [5.11.2.4.1-1]**

λ_i = modification factor(s) **LRFD [5.11.2.4.2]**

- (0.70) Side cover for #11 bar and smaller, normal to plane of hook, is not less than 2.5 inches, and 90 hook, cover on bar extension beyond hook not less than 2.0 inches
- (0.80) Hooks for #11 bar and smaller enclosed vertically or horizontally within ties or stirrups ties which are placed along the full development length, ℓ_{dh}, at a spacing not exceeding $3d_b$
- (1.20) Epoxy coated reinforcement

d_b = diameter of bar (in.)

f'_c = specified compressive strength of concrete (ksi)



d_b = nominal diameter of reinforcing bar (in)

Definition of Standard Hooks **LRFD [5.10.2.1]**

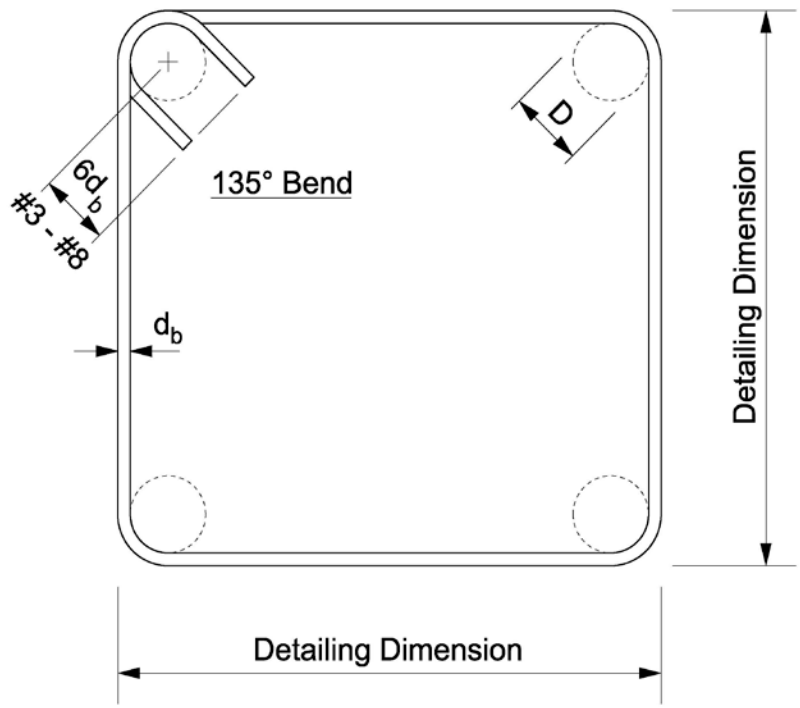
MINIMUM BEND DIAMETER (D) – **LRFD [5.10.2.3]**

D = 4d_b FOR #3 THRU #5

D = 6d_b FOR #6 THRU #8

MINIMUM HOOK, ALL GRADES					
		90° HOOKS		135° HOOKS	
BAR SIZE	D	HOOK A	APPROX J	HOOK A	H
3	1 ½	3	4	4	2 ½
4	2	3 ½	4 ½	4 ½	3
5	2 ½	4 ½	6	5 ½	3 ¾
6	4 ½	10	1-0	8	4 ½

Figure 9.9-3
Standard Hooks and Bends for Deformed Transverse Reinforcement
(Stirrups and Ties)



Stirrup Bar Length equals sum of all Detailing Dimensions plus "Stirrup Add-On" from table

d_b = nominal diameter of reinforcing bar (in)

Definition of Standard Hooks **LRFD [5.10.2.1]**

MINIMUM BEND DIAMETER (D) – **LRFD [5.10.2.3]**

D = 4d_b FOR #3 THRU #5

D = 6d_b FOR #6 THRU #8

BAR SIZE	D	STIRRUP ADD-ON
3	1 ½	5
4	2	6
5	2 ½	8
6	4 ½	10
7	5 ¼	12
8	6	13

Figure 9.9-4
Standard Details and Bends for Deformed Transverse Reinforcement
(Closed Stirrups)

BILL OF BARS

NOTE: THE FIRST OR FIRST TWO DIGITS OF THE BAR MARK SIGNIFIES THE BAR SIZE.

BAR MARK	COAT	NO. REQ'D	LENGTH	BENT	BAR SERIES	LOCATION
S501		10	4-2		∆	SLAB - TRANS.
S502		20	6-3		∆	SLAB - TRANS.
S503	X	19	42-8			SLAB - LONG.

∆ LENGTH SHOWN FOR BAR IS AN AVERAGE LENGTH AND SHOULD ONLY BE USED FOR BAR WEIGHT CALCULATIONS. SEE BAR SERIES TABLE FOR ACTUAL LENGTHS.

Figure 9.9-5
Bill of Bars

BAR SERIES TABLE

MARK	NO. REQ'D.	LENGTH
S501	1 SERIES OF 10	2-1 TO 6-3
S502	2 SERIES OF 10	3-2 TO 9-5

BUNDLE AND TAG EACH SERIES SEPARATELY

Figure 9.9-6
Bar Series Table

BAR SIZE	BAR WEIGHT (lbs/ft)	NOM. DIA (in)	NOM. AREA (in^2)	NUMBER OF BARS								
				2	3	4	5	6	7	8	9	10
4	0.668	0.500	0.20	0.40	0.60	0.80	1.00	1.20	1.40	1.60	1.80	2.00
5	1.043	0.625	0.31	0.62	0.93	1.24	1.55	1.86	2.17	2.48	2.79	3.10
6	1.502	0.750	0.44	0.88	1.32	1.76	2.20	2.64	3.08	3.52	3.96	4.40
7	2.044	0.875	0.60	1.20	1.80	2.40	3.00	3.60	4.20	4.80	5.40	6.00
8	2.670	1.000	0.79	1.58	2.37	3.16	3.95	4.74	5.53	6.32	7.11	7.90
9	3.400	1.128	1.00	2.00	3.00	4.00	5.00	6.00	7.00	8.00	9.00	10.00
10	4.303	1.270	1.27	2.54	3.81	5.08	6.35	7.62	8.89	10.16	11.43	12.70
11	5.313	1.410	1.56	3.12	4.68	6.24	7.80	9.36	10.92	12.48	14.04	15.60

Table 9.9-3
Bar Areas Per Number of Bars (in^2)

BAR SIZE	4 ½“	5”	5 ½“	6”	6 ½“	7”	7 ½”	8”	8 ½”	9”	10”	11”	12”
4	0.52	0.47	0.43	0.39	0.36	0.34	0.31	0.29	0.28	0.26	0.24	0.21	0.20
5	0.82	0.74	0.67	0.61	0.57	0.53	0.49	0.46	0.43	0.41	0.37	0.33	0.31
6	1.18	1.06	0.96	0.88	0.82	0.76	0.71	0.66	0.62	0.59	0.53	0.48	0.44
7	1.60	1.44	1.31	1.20	1.11	1.03	0.96	0.90	0.85	0.80	0.72	0.66	0.60
8	2.09	1.88	1.71	1.57	1.45	1.35	1.26	1.18	1.11	1.05	0.94	0.86	0.79
9	---	2.40	2.18	2.00	1.85	1.71	1.60	1.50	1.41	1.33	1.20	1.09	1.00
10	---	3.04	2.76	2.53	2.34	2.17	2.02	1.90	1.79	1.69	1.52	1.38	1.27
11	---	3.75	3.41	3.12	2.88	2.68	2.50	2.34	2.21	2.08	1.87	1.70	1.56

Table 9.9-4
Area of Bar Reinf. (in^2 / ft) vs. Spacing of Bars (in)

9.10 Granular Materials

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

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

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

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

Table 9.10-5
Recommendations for Granular Material Usage

9.11 References

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

9.12 Appendix - Draft Bar Tables

The following Draft Bar Tables are provided for information only. We expect the tables to be moved into the main text of Chapter 9 in January of 2019, and at that time to begin their use. We are delaying their use to allow time for modification of details and software that are affected.

The 2015 Interim Revisions to the AASHTO LRFD Bridge Design Specifications (7th Edition), modified the tension development lengths and tension lap lengths for straight deformed bars as follows:

The tension development length, ℓ_d ,shall not be less than the product of the basic tension development length, ℓ_{db} , and the appropriate modification factors, λ_i . **LRFD [5.11.2.1.1]**

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

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

where:

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

λ_{rl} = reinforcement location factor

λ_{cf} = coating factor

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

λ_{rc} = reinforcement confinement factor

λ_{er} = excess reinforcement factor

f_y = specified yield strength of reinforcing bars (ksi)

d_b = diameter of bar (in.)

f'_c = specified compressive strength of concrete (ksi)

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

Per **LRFD [5.11.5.3.1]**, there are two lap splice classes, Class A and Class B.

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

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

Draft Table

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

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

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

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

Draft Table

Uncoated (f'$_c$ = 4,000 psi; f$_y$ = 60,000 psi)

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

Uncoated (f'$_c$ = 4,000 psi; f$_y$ = 60,000 psi)

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

Draft Table

Epoxy Coated (f'_c = 3,500 psi; f_y = 60,000 psi)

Basic Lap - Others					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	1'-7"	1'-7"	2'-0"	2'-0"
	2.0"	1'-7"	1'-7"	2'-0"	2'-0"
	≥ 2.5"	1'-7"	1'-7"	2'-0"	2'-0"
5	1.5"	2'-5"	2'-5"	3'-2"	3'-2"
	2.0"	2'-5"	1'-11"	3'-2"	2'-6"
	≥ 2.5"	2'-5"	1'-11"	3'-2"	2'-6"
6	1.5"	3'-2"	2'-11"	4'-1"	3'-9"
	2.0"	3'-2"	2'-11"	4'-1"	3'-9"
	≥ 2.5"	3'-2"	2'-4"	4'-1"	3'-0"
7	1.5"	3'-10"	3'-10"	5'-0"	5'-0"
	2.0"	3'-9"	3'-5"	4'-10"	4'-5"
	≥ 2.5"	3'-9"	3'-5"	4'-10"	4'-5"
8	1.5"	4'-10"	4'-10"	6'-3"	6'-3"
	2.0"	4'-10"	3'-11"	6'-3"	5'-0"
	≥ 2.5"	4'-10"	3'-11"	6'-3"	5'-0"
9	1.5"	6'-1"	6'-0"	7'-11"	7'-10"
	2.0"	6'-1"	4'-10"	7'-11"	6'-3"
	≥ 2.5"	6'-1"	4'-4"	7'-11"	5'-8"
10	1.5"	7'-10"	7'-3"	10'-2"	9'-5"
	2.0"	7'-10"	5'-11"	10'-2"	7'-8"
	≥ 2.5"	7'-10"	5'-2"	10'-2"	6'-8"
11	1.5"	9'-8"	8'-9"	12'-7"	11'-4"
	2.0"	9'-8"	7'-1"	12'-7"	9'-2"
	≥ 2.5"	9'-8"	6'-5"	12'-7"	8'-4"

Epoxy Coated (f'_c = 3,500 psi; f_y = 60,000 psi)

Horizontal Lap w/ >12" Concrete Cast Below - Top					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	2'-0"	2'-0"	2'-8"	2'-8"
	2.0"	2'-0"	2'-0"	2'-8"	2'-8"
	≥ 2.5"	2'-0"	2'-0"	2'-8"	2'-8"
5	1.5"	2'-9"	2'-9"	3'-7"	3'-7"
	2.0"	2'-9"	2'-6"	3'-7"	3'-3"
	≥ 2.5"	2'-9"	2'-6"	3'-7"	3'-3"
6	1.5"	3'-7"	3'-4"	4'-7"	4'-3"
	2.0"	3'-7"	3'-4"	4'-7"	4'-3"
	≥ 2.5"	3'-7"	3'-0"	4'-7"	3'-11"
7	1.5"	4'-4"	4'-4"	5'-7"	5'-7"
	2.0"	4'-3"	3'-10"	5'-6"	5'-0"
	≥ 2.5"	4'-3"	3'-10"	5'-6"	5'-0"
8	1.5"	5'-6"	5'-6"	7'-1"	7'-1"
	2.0"	5'-6"	4'-5"	7'-1"	5'-8"
	≥ 2.5"	5'-6"	4'-5"	7'-1"	5'-8"
9	1.5"	6'-11"	6'-10"	9'-0"	8'-10"
	2.0"	6'-11"	5'-5"	9'-0"	7'-1"
	≥ 2.5"	6'-11"	4'-11"	9'-0"	6'-5"
10	1.5"	8'-11"	8'-2"	11'-7"	10'-8"
	2.0"	8'-11"	6'-8"	11'-7"	8'-8"
	≥ 2.5"	8'-11"	5'-10"	11'-7"	7'-7"
11	1.5"	10'-11"	9'-10"	14'-3"	12'-10"
	2.0"	10'-11"	8'-0"	14'-3"	10'-5"
	≥ 2.5"	10'-11"	7'-3"	14'-3"	9'-5"

Draft Table

Uncoated (f'_c = 3,500 psi; f_y = 60,000 psi)

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

Uncoated (f'_c = 3,500 psi; f_y = 60,000 psi)

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

This page intentionally left blank.

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, geotechnical and design. For a typical bridge design, the following four steps are required (see 6.2):

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

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

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

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

11.1.2 Foundation Type Selection

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

- Magnitude and direction of loads.
- Depth to suitable bearing material.
- Potential for liquefaction, undermining or scour.
- Frost potential.

For cohesive soils, point resistance can be calculated using the following equation:

$$R_p = 9 S_u A_p$$

Where:

R_p = Point resistance capacity (tons)

S_u = Undrained shear strength of the cohesive soil near the pile base (tsf)

A_p = Pile end area (feet^2)

This equation represents the maximum value of point resistance for cohesive soil. This value is often assumed to be zero because substantial movement of the pile tip (1/10 of the pile diameter) is needed to mobilize point resistance capacity. This amount of tip movement seldom occurs after installation.

A point resistance (or end bearing) pile surrounded by soil is not a structural member like a column. Both experience and theory demonstrate that there is no danger of a point resistance pile buckling due to inadequate lateral support if it is surrounded by even the very softest soil. Therefore, pile stresses can exceed column stresses. Although, exposed pile bent piles may act as structural columns.

11.3.1.15.3 Group Capacity

The nominal resistance capacity of pile groups may be less than the sum of the individual nominal resistances of each pile in the group for friction piles founded in cohesive soil. For pile groups founded in cohesive soil, the pile group must be analyzed as an equivalent pier for block failure in accordance with **LRFD [10.7.3.9]**. WisDOT no longer accepts the Converse-Labarre method of analysis to account for group action. If the pile group is tipped in a firm stratum overlying a weak layer, the weak layer should be checked for possible punching failure in accordance with **LRFD [10.6.3.1.2a]**. Experience in Wisconsin indicates that in most thixotropic clays where piles are driven to a hammer bearing as determined by dynamic formulas, pile group action is not the controlling factor to determine pile resistance capacity. For pile groups in sand, the sum of the nominal resistance of the individual piles always controls the group resistance.

11.3.1.16 Lateral Load Resistance

Structures supported by single piles or pile groups are frequently subjected to lateral forces from lateral earth pressure, live load forces, wave action, ice loads and wind forces. Piles subjected to lateral forces must be designed to meet combined stress and deflection criteria to prevent impairment or premature failure of the foundation or superstructure. To solve the soil-structure interaction problems, the designer must consider the following:

- Pile group configuration.

- Pile stiffness.
- Degree of fixity at the pile connection with the pile footing.
- Maximum bending moment induced on the pile from the superstructure load and moment distribution along the pile length.
- Probable points of fixity near the pile tip.
- Soil response (P-y method) for both the strength and service limit states.
- Pile deflection permitted by the superstructure at the service limit state.

If a more detailed lateral load investigation is desired, a P-y analysis is typically performed using commercially available software such as COM624P, FB Multi-Pier or L-Pile. A resistance factor of 1.0 is applied to the soil response when performing a P-y analysis using factored loads since the soil response represents a nominal (ultimate) condition. For a more detailed analysis of lateral loads and displacements, refer to the listed FHWA design references at the end of this chapter or a geotechnical engineering book.

WisDOT policy item:

A detailed analysis is required for the lateral resistance of piles used in A3 abutments.

11.3.1.17 Other Design Considerations

Several other topics should be considered during design, as presented below.

11.3.1.17.1 Downdrag Load

Negative shaft resistance (downdrag) results in the soil adhesion forces pulling down the pile instead of the soil adhesion forces resisting the applied load. This can occur when settlement of the soil through which the piling is driven takes place. It has been found that only a small amount of settlement is necessary to mobilize these additional pile (drag) loads. This settlement occurs due to consolidation of softer soil strata caused by such items as increased embankment loads (due to earth fill) or a lowering of the existing ground water elevation. The nominal pile resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer acting to produce negative skin resistance. When this condition is present, the designer may provide time to allow consolidation to occur before driving piling, or **LRFD [10.7.3.8.6]** may be used to estimate the available pile resistance to withstand the downdrag plus structure loads. Other alternatives are to pre-auger the piling, drive the pile to bearing within a permanent pipe sleeve that is placed from the base of the substructure unit to the bottom of the soft soil layer(s), coat the pile with bitumen above the compressible soil strata or use proprietary materials to encase the piles (within fill constructed after the piling is installed). The Department has experienced problems with bitumen coatings.

Table of Contents

12.1 General

Abutments are used at the ends of bridges to retain the embankment and to carry the vertical and horizontal loads from the superstructure to the foundation, as illustrated in Figure 12.1-1. The design requirements for abutments are similar to those for retaining walls and for piers; each must be stable against overturning and sliding. Abutment foundations must also be designed to prevent differential settlement and excessive lateral movements.

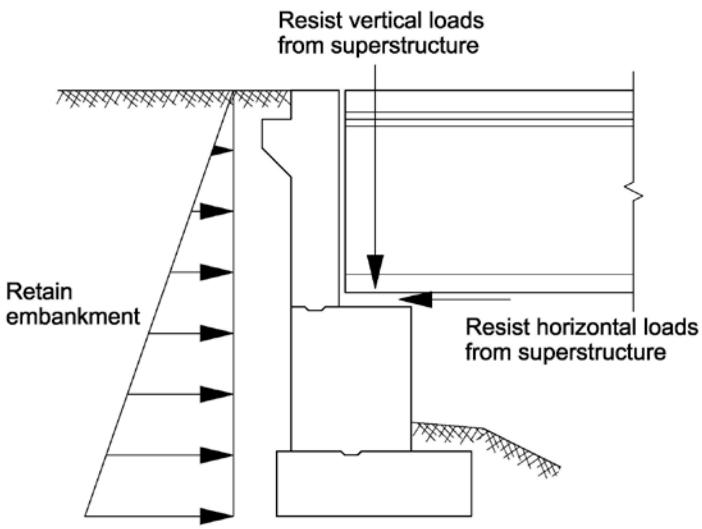


Figure 12.1-1
Primary Functions of an Abutment

The components of a typical abutment are illustrated in Figure 12.1-2.

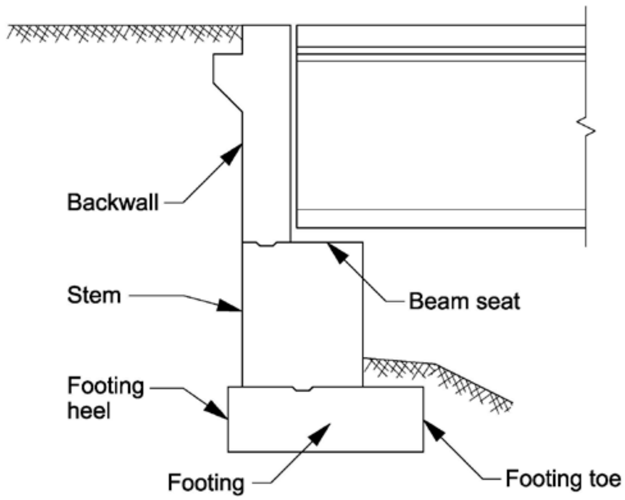


Figure 12.1-2
Components of an Abutment

Many types of abutments can be satisfactorily utilized for a particular bridge site. Economics is usually the primary factor in selecting the type of abutment to be used. For river or stream crossings, the minimum required channel area and section are considered. For highway overpasses, minimum horizontal clearances and sight-distances must be maintained.

An abutment built on a slope or on top of a slope is less likely to become a collision obstacle than one on the bottom of the slope and is more desirable from a safety standpoint. Aesthetics is also a factor when selecting the most suitable abutment type.

12.2 Abutment Types

Several different abutment types can be used, including full-retaining, semi-retaining, sill, spill-through or open, pile-encased and special designs. Each of these abutment types is described in the following sections.

12.2.1 Full-Retaining

A full-retaining abutment is built at the bottom of the embankment and must retain the entire roadway embankment, as shown in Figure 12.2-1. This abutment type is generally the most costly. However, by reducing the span length and superstructure cost, the total structure cost may be reduced in some cases. Full-retaining abutments may be desirable where right of way is critical.

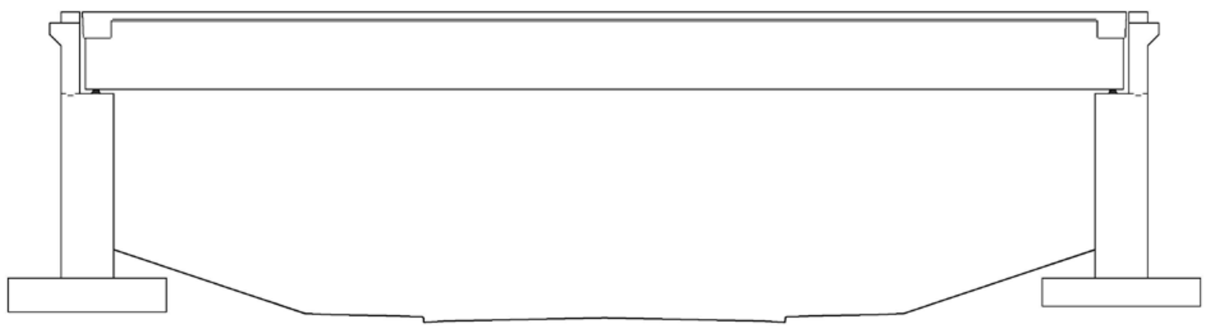


Figure 12.2-1
Full-Retaining Abutment

Rigid-frame structures use a full-retaining abutment poured monolithically with the superstructure. If both abutments are connected by fixed bearings to the superstructure (as in rigid frames), the abutment wings are joined to the body by a mortised expansion joint. For a non-skewed abutment, this enables the body to rotate about its base and allows for superstructure contraction and expansion due to temperature and shrinkage, assuming that rotation is possible.

An objectionable feature of full-retaining abutments is the difficulty associated with placing and compacting material against the body and between the wing walls. It is possible that full-retaining abutments may be pushed out of vertical alignment if heavy equipment is permitted to work near the walls, and this temporary condition is not accounted for in a temporary load combination. The placement of the embankment after abutment construction may cause foundation settlement. For these reasons, as much of the roadway embankment as practical should be in place before starting abutment construction. Backfilling above the beam seat is prohibited until the superstructure is in place.

Other disadvantages of full-retaining abutments are:

- Minimum horizontal clearance

- Minimum sight distance when roadway underneath is on a curved alignment
- Collision hazard when abutment front face is not protected
- Settlement

12.2.2 Semi-Retaining

The semi-retaining abutment (Type A3) is built somewhere between the bottom and top of the roadway embankment, as illustrated in Figure 12.2-2. It provides more horizontal clearance and sight distance than a full-retaining abutment. Located on the embankment slope, it becomes less of a collision hazard for a vehicle that is out of control.

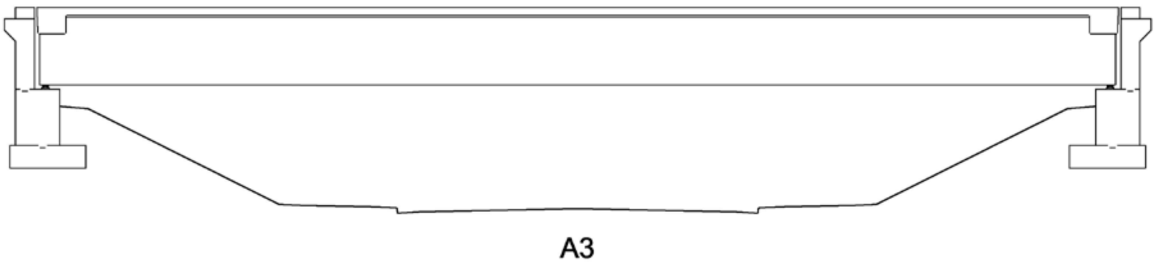


Figure 12.2-2
Semi-Retaining Abutment

The description of full-retaining abutments in 12.2.1 generally applies to semi-retaining abutments as well. They are used primarily in highway-highway crossings as a substitute for a shoulder pier and sill abutment. Semi-retaining abutments generally are designed with a fixed base, allowing wing walls to be rigidly attached to the abutment body. The wings and the body of the abutment are usually poured monolithically.

12.2.3 Sill

The sill abutment (Type A1) is constructed at the top of the slope after the roadway embankment is close to final grade, as shown in Figure 12.2-3. The sill abutment helps avoid many of the problems that cause rough approach pavements. It eliminates the difficulties of obtaining adequate compaction adjacent to the relatively high walls of closed abutments. Since the approach embankment may settle by forcing up or bulging up the slope in front of the abutment body, a berm is often constructed at the front of the body. The weight of the berm helps prevent such bulging.

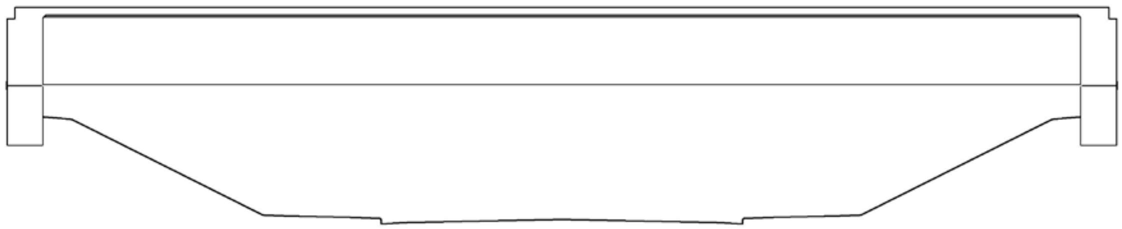


Figure 12.2-3
Sill Abutment

Sill abutments are the least expensive abutment type and are usually the easiest to construct. However, this abutment type results in a higher superstructure cost, so the overall cost of the structure should be evaluated with other alternatives.

For shallow superstructures where wing piles are not required, the Type A1 abutment is used with a fixed seat. This minimizes cracking between the body wall and wings. However, for shallow superstructures where wing piles are required, the Type A1 abutment is used with a semi-expansion seat. This allows superstructure movement, and it reduces potential cracking between the wings and body.

The parallel-to-abutment-centerline wings or elephant-ear wings, as shown on the Standard Details for Wings Parallel to A1 Abutment Centerline, should be used for grade separations when possible. This wing type is preferred because it increases flexibility in the abutment, it simplifies compaction of fill, and it improves stability. However, parallel-to-abutment-centerline wings should not be used for stream crossings when the high water elevation is above the bottom of the abutment. This wing configuration may not adequately protect bridge approaches and abutment backfill from the adjacent waterway.

12.2.4 Spill-Through or Open

A spill-through or open abutment is mostly used where an additional span may be added to the bridge in the future. It may also be used to satisfy unique construction problems. This abutment type is situated on columns or stems that extend upward from the natural ground. It is essentially a pier being used as an abutment.

It is very difficult to properly compact the embankment materials that must be placed around the columns and under the abutment cap. Early settlement and erosion are problems frequently encountered with spill-through or open abutments.

If the abutment is to be used as a future pier, it is important that the wings and backwall be designed and detailed for easy removal. Construction joints should be separated by felt or other acceptable material. Reinforcing steel should not extend through the joints. Bolts with threaded inserts should be used to carry tension stresses across joints.

12.2.5 Pile-Encased

Pile-encased abutments (Type A5) should only be used where documented cost data shows them to be more economical than sill abutments due to site conditions. For local roads right-of-way acquisition can be difficult, making the A5 a good option. Requiring crane access from only one side of a stream may be another reason to use a single span bridge with A5 abutments, as would savings in railing costs. Steeper topography may make A5 abutments a more reasonable choice than sill abutments. In general, however, using sill abutments with longer bridges under most conditions has cost advantages over using the Type A5 abutments. Type A5 abutments may require additional erosion control measures that increase construction cost.

The wall height of pile-encased abutments is limited to a maximum of 10 feet since increased wall height will increase soil pressure, resulting in uneconomical pile design due to size or spacing requirements. Reinforcement in the abutment body is designed based on live load surcharge and soil pressure on the back wall.

Pile-encased abutments with fixed seats are limited to a maximum skew of 15 degrees for girder structures and 30 degrees for slab structures in order to limit damage due to thermal expansion and contraction of the superstructure. Pile-encased abutments with a semi-expansion seat are limited to a maximum skew of 30 degrees. Wing skew angles are at 45 degrees relative to the body to prevent cracking between the abutment body and wings. These wings may be used for stream crossings when the high water elevation is above the bottom of the abutment. Parallel-to-roadway wings may be considered for extreme hydraulic conditions, however this will require a special design.

12.2.6 Special Designs

In addition to the standard abutment types described in the previous sections, many different styles and variations of those abutment types can also be designed. Such special abutment designs may be required due to special aesthetic requirements, unique soil conditions or unique structural reasons. Special designs of abutments require prior approval by the Bureau of Structures Development Chief.

12.3 Types of Abutment Support

Piles, drilled shafts and spread footings are the general types of abutment support used. This section provides a brief description of each type of abutment support.

WisDOT policy item:

Geotechnical design of abutment supports shall be in accordance with the 4th Edition of the AASHTO LRFD Bridge Design Specifications for Highway Bridges. No additional guidance is available at this time.

Structural design of abutment supports shall be in accordance with LRFD, as specified in the 4th Edition of AASHTO LRFD Bridge Design Specifications

12.3.1 Piles or Drilled Shafts

Most abutments are supported on piles to prevent abutment settlement. Bridge approach embankments are usually constructed of fill material that can experience settlement over several years. This settlement may be the result of the type of embankment material or the original foundation material under the embankment. By driving piles through the embankment and into the original ground, abutments usually do not settle with the embankment. A settling embankment may be resisted by the abutment piles through friction between the piles and fill material. The added load to friction piles and the need for preboring should be considered.

It is generally not necessary to prebore non-displacement piles for any fill depths, and it is not necessary to prebore displacement piles for fill depths less than 15 feet below the bottom of footing. However, for some problem soils this may not apply. See the soils report to determine if preboring is required. If required, the Special Provisions must be written with preboring guidelines.

Battered piles may cause more of a problem than vertical piles and are given special consideration. When driving battered piles, reduced hammer efficiency may be experienced, and battered piles should not be considered when negative skin friction loads are anticipated.

Fill embankments frequently shift laterally, as well as vertically. A complete foundation site investigation and information on fill material is a prerequisite for successful pile design.

Piles placed in prebored holes cored into rock do not require driving. The full design loading can be used if the hole is of adequate size to prevent pile hangups and to allow filling with concrete.

Piles in abutments are subject to lateral loads. The lateral resistance on a pile is usually determined from an acceptable level of lateral displacement and not the ultimate load that causes a stress failure in the pile. The lateral resistance on a pile may be more dependent on the material into which the pile is driven than on the pile type. See Chapter 11 – Foundation Support for a more thorough discussion of piles and allowable pile loads.

12.3.2 Spread Footings

Abutments on spread footings are generally used only in cut sections where the original soil can sustain reasonable pressures without excessive settlement. The bearing resistance is determined by the Geotechnical Section or the geotechnical consultant.

With improved procedures and better control of embankment construction, spread footings can be used successfully on fill material. It is important that construction be timed to permit the foundation material to consolidate before the spread footings are constructed. An advantage of spread footings is that the differential settlement between approach fills and abutments is minimized.

The use of spread footings is given greater consideration for simple-span bridges than for continuous-span bridges. However, under special conditions, continuous-span bridges can be designed for small amounts of settlement. Drainage for abutments on spread footings can be very critical. For these reasons, pile footings are usually preferred.

Lateral forces on abutments are resisted by passive earth pressure and friction between the soil and concrete. A shear key provides additional area on which passive earth pressure can act. A berm in front of the abutment may be necessary to prevent a shear failure in the soil along the slope.

12.4 Abutment Wing Walls

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

12.4.1 Wing Wall Length

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

12.4.1.1 Wings Parallel to Roadway

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

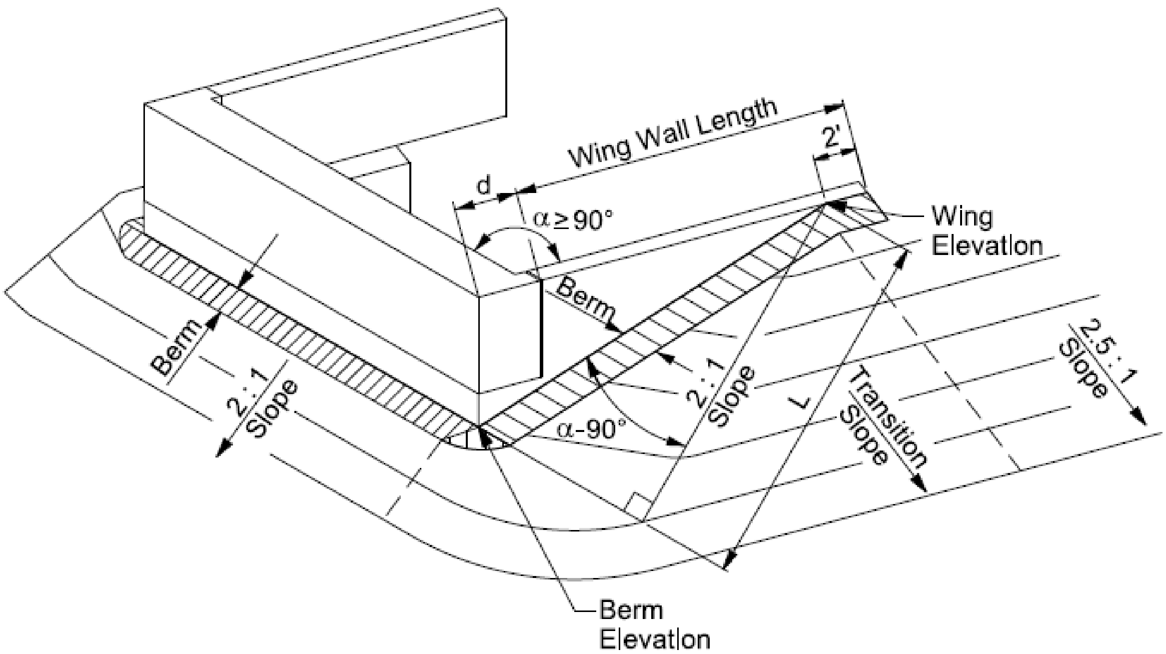


Figure 12.4-1
Wings Parallel to Roadway and Wing Wall Angle ≥ 90°

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

L = (Wing Elevation – Berm Elevation) (2)

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

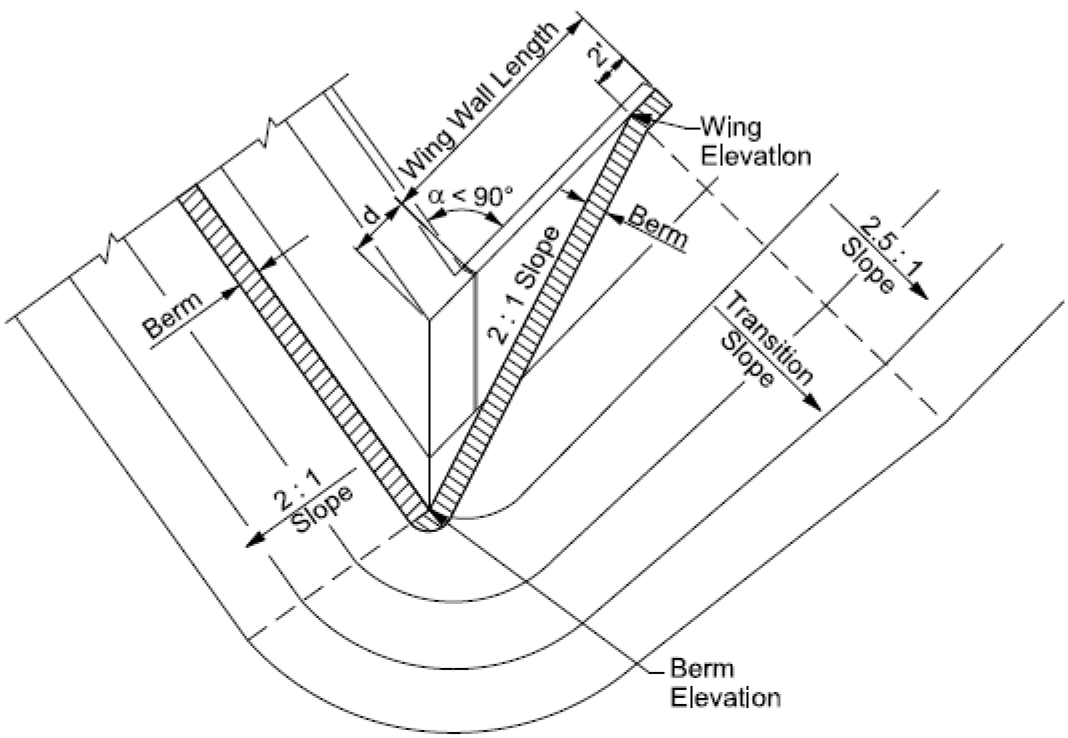


Figure 12.4-2
Wings Parallel to Roadway and Wing Wall Angle < 90°

For wing wall angle, $\alpha < 90°$:

Wing Wall Length = (Wing Elevation – Berm Elevation) (2) – d – 2.0 feet

Note: The above calculations provide the minimum required wing wall length and should be rounded accordingly.

12.4.1.2 Wings Not Parallel to Roadway and Equal Slopes

The calculation of wing wall lengths for wings that are not parallel to the roadway and that have equal slopes is illustrated in Figure 12.4-3.

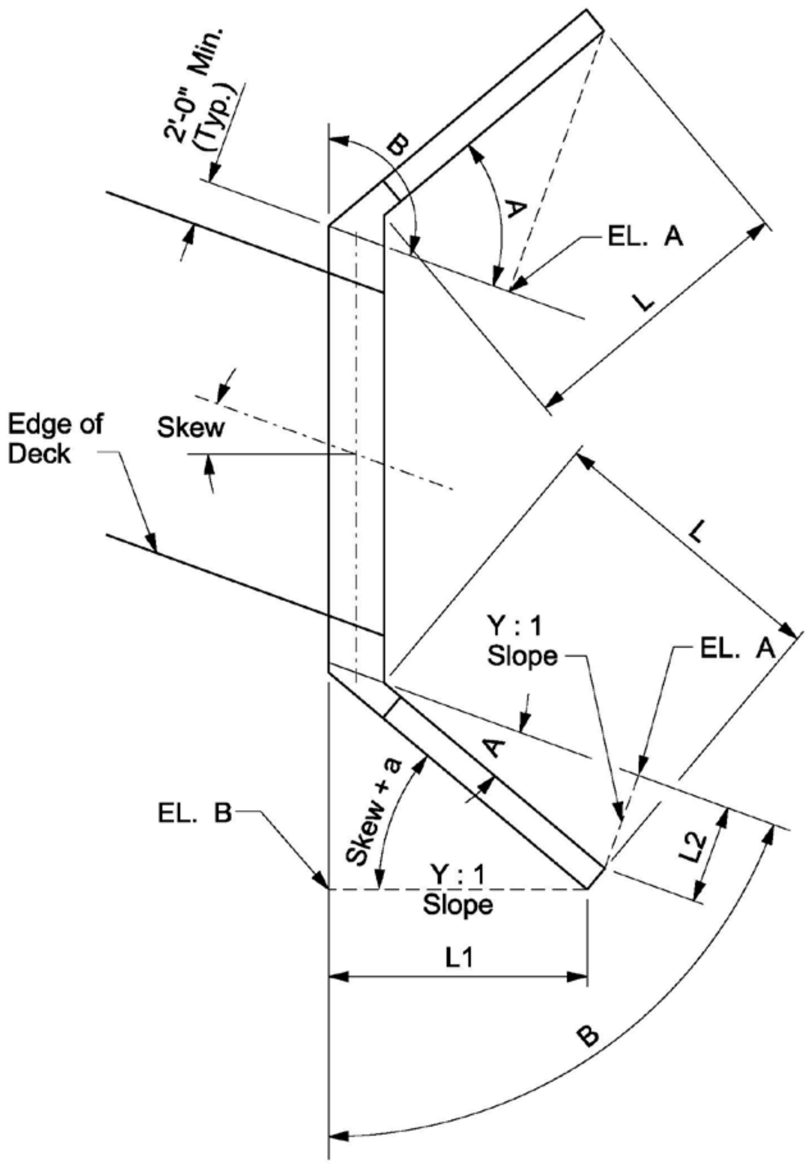


Figure 12.4-3
Wings Not Parallel to Roadway and Equal Slopes

For angle B ≥ 90°:

$$L1 + L2 = (EL.\ A - EL.\ B)(Y)$$

$$\cos(a - Skew) = \frac{L1}{L}$$

$$\sin(a) = \frac{L2}{L}$$

$$L = \frac{Y\,(EL.\,A - EL.\,B)}{\cos(a - Skew) + \sin(a)}$$

For angle B < 90°:

$$L1 + L2 = (EL.\,A - EL.\,B)(Y)$$

$$\cos(Skew + a) = \frac{L1}{L}$$

$$\sin(a) = \frac{L2}{L}$$

$$L = \frac{Y\,(EL.\,A - EL.\,B)}{\cos(Skew + a) + \sin(a)}$$

12.4.2 Wing Wall Loads

Wing walls are designed as retaining walls. Earth loads and surcharge loads are applied to wing walls similar to how they are applied to the stem of a retaining wall. Wing walls are analyzed as cantilevers extending from the abutment body.

The parapet on top of the wing is designed to resist railing loading, but it is not necessary that the railing loads be applied to the wing walls. Railing loads are dynamic or impact loads and are absorbed by the mass of the wing wall and if necessary by passive earth pressure.

The forces produced by the active earth pressure are resisted by the wing piles and the abutment body. Passive earth pressure resistance generally is not utilized, because there is a possibility that the approach fill slopes may slide away from the wings. This may seem like a conservative assumption, but it is justified due to the highly unpredictable forces experienced by a wing wall.

Wing walls without special footings that are poured monolithically with the abutment body are subjected to a bending moment, shear force and torsion. The primary force is the bending moment. Torsion is usually neglected.

The bending moment induced in the cantilevered wing wall by active earth pressure is reduced by the expected lateral resistance of the wing pile group times the distance to the section being investigated. This lateral pile resistance is increased by using battered piles. Individual piles offer little lateral resistance because of small wing deflections. See Chapter 11 – Foundation Support for lateral pile resistance.

12.4.3 Wing Wall Parapets

Steel plate beam guard is used at bridge approaches and is attached to the wing wall parapets. This helps to prevent vehicles from colliding directly into the end of the parapet.

A vehicle striking a guard rail may produce a high-tension force in the guard rail. It is important that sufficient longitudinal parapet steel be provided to resist this force. If the concrete in the parapet is demolished, the longitudinal steel continues to act as a cable guard rail if it remains attached to the steel plate beam guard.

12.5 Abutment Depths, Excavation and Construction

This section describes some additional design considerations for abutments, including depth, excavation and construction.

Abutment construction must satisfy the requirements for construction joints and beam seats presented in 12.9.1 and 12.9.2, respectively.

The abutment body is generally located above the normal water. Refer to the *Standard Specifications* or Special Provisions if part of the abutment body is below normal water.

12.5.1 Abutment Depths

The required depth of the abutment footing to prevent frost damage depends on the amount of water in the foundation material. Frost damage works in two directions. First, ice lenses form in the soil, heaving it upward. These lenses grow by absorbing additional water from below the frost line. Silts are susceptible to heaves, but well-drained sands and dense clays generally do not heave. Second, the direction of frost action is downward. The ice lenses thaw from the top down, causing a layer of water to be trapped near the surface. This water emulsifies the soil, permitting it to flow out from under the footing.

Sill and semi-retaining abutments are constructed on slopes which remain relatively moisture free. Sill abutments have been constructed in all parts of Wisconsin with footings only 2.5 feet below ground and have experienced no frost heave problems.

Full-retaining abutments are constructed at the bottom of embankment slopes, and their footings are more likely to be within a soil of high moisture content. Therefore, footings for full-retaining abutments must be located below the level of maximum frost penetration. Maximum frost penetration varies from 4 feet in the southeastern part of Wisconsin to 6 feet in the northwestern corner.

12.5.2 Abutment Excavation

Abutment excavation is referred to as "Excavation for Structures Bridges." It is measured as a unit for each specific bridge and is paid for at the contract lump sum price.

When a new bridge is constructed, a new roadway approaching the bridge is generally also constructed. Since the roadway contractor and bridge contractor are not necessarily the same, the limits of excavation to be performed by each must be specified. The roadway contractor cuts or fills earth to the upper limits of structural excavation as specified on the bridge plans or in the *Standard Specifications for Highway and Structure Construction*. If the bridge contractor does his work before the roadway contractor or if there is no roadway contract, the upper limit of structural excavation is the existing ground line. For sill abutments, the upper limit is specified in the *Standard Specifications* and need not be shown on the abutment plans.

For semi-retaining and full-retaining abutments, the upper limits are shown on the abutment plans. If a cut condition exists, the upper limit is usually the subgrade elevation and the top surface of the embankment slope (bottom of slope protection). Earth above these limits is removed by the roadway contractor. A semi-retaining or full-retaining abutment placed on fill is considered a unique problem by the design engineer, and limits of excavation must be set accordingly. Construction sequence and type of fill material are considered when setting excavation limits. Slopes greater than 1.5 horizontal to 1 vertical are difficult to construct and generally are not specified. It is sometimes advantageous to have the roadway contractor place extra fill that later must be excavated by the bridge contractor, because the overburden aids in compaction and reduces subsequent settlement.

Lateral limits of excavation are not defined in the *Standard Specifications*. The contractor must excavate whatever is necessary within the right-of-way for the placement of the forms.

12.6 Abutment Drainage and Backfill

This section describes abutment design considerations related to drainage and backfill. The abutment drainage and backfill must be designed and detailed properly to prevent undesirable loads from being applied to the abutment.

12.6.1 Abutment Drainage

Abutment drainage is necessary to prevent hydrostatic pressure and frost pressure. Hydrostatic pressure, including both soil and water, can amount to an equivalent fluid unit weight of soil of 85 pcf. Frost action, which can occur in silty backfill, may result in extremely high pressures. On high abutments, these pressures will produce a very large force which could result in structural damage or abutment movement if not accounted for in the design.

To prevent these additional pressures on abutments, it is necessary to drain away whatever water accumulates behind the body and wings. This is accomplished using a pervious granular fill on the inside face of the abutment. Pipe underdrain must be provided to drain the fill located behind the abutment body and wings. For rehabilitation of structures, provide plan details to replace inadequate underdrain systems.

Past experience indicates that sill abutments are not capable of withstanding hydrostatic pressure on their full height without leaking.

Semi-retaining and full-retaining abutments generally will be overstressed or may slide if subject to large hydrostatic or frost pressures unless accounted for in the design. Therefore, “Pipe Underdrain Wrapped 6-inch” is required behind all abutments. This pipe underdrain is used behind the abutment and outside the abutment to drain the water away. It is best to place the pipe underdrain at the bottom of footing elevation as per standards. However, if it is not possible to discharge the water to a lower elevation, the pipe underdrain should be placed higher.

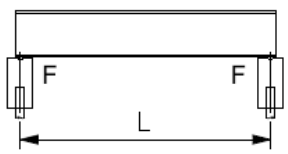
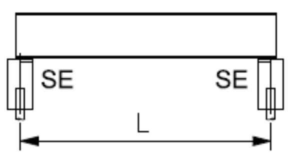
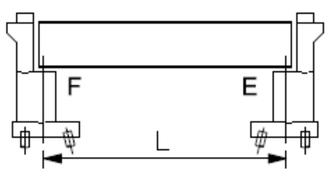
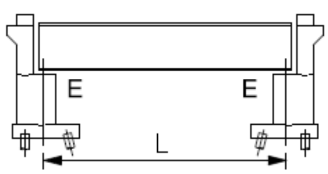
Pipe underdrains and weepholes may discharge water during freezing temperatures. In urban areas, this may create a problem due to the accumulation of flow and ice on sidewalks.

12.6.2 Abutment Backfill Material

All abutments and wings shall utilize “Backfill Structure” to facilitate drainage. See Standard Detail 9.01 – Structure Backfill Limits and Notes – for typical pay limits and plan notes.

12.7 Selection of Standard Abutment Types

From past experience and investigations, the abutment types presented in Figure 12.7-1 are generally most suitable and economical for the given conditions. Although piles are shown for each abutment type, drilled shafts or spread footings may also be utilized depending on the material conditions at the bridge site. The chart in Figure 12.7-1 provides a recommended guide for abutment type selection.

Abutment Arrangements	Superstructures		
	Concrete Slab Spans	Prestressed Girders	Steel Girders
Type A1 (F-F) 	a. L ≤ 150' S ≤ 30° AL ≤ 50'	a. L ≤ 150' S ≤ 15° AL ≤ 50'	a. L ≤ 150' S ≤ 15° AL ≤ 50'
Type A1 (SE-SE) 	a. L ≤ 300' S ≤ 30° AL > 50'	a. L ≤ 300' S ≤ 40°	a. L ≤ 150' S ≤ 40°
Type A3 (F-E) 	Not used	Single span and (S > 40°)	Single span and (L > 150' or S > 40°)
Type A3 (E-E) 	b. L >300' and S ≤ 30° with rigid piers	L>300' or (S > 40° and multi-span)	Multi-span and (L>150' or S > 40°) with rigid piers

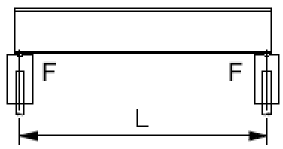
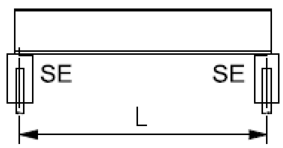
Abutment Arrangements	Superstructures		
	Concrete Slab Spans	Prestressed Girders	Steel Girders
Type A5 (F-F)  	L ≤ 150' S ≤ 30° AL ≤ 50'	L ≤ 150' S ≤ 15° AL ≤ 50'	L ≤ 150' S ≤ 15° AL ≤ 50'
Type A5 (SE-SE)  	L ≤ 200' S ≤ 30° AL > 50'	L ≤ 200' S ≤ 30°	L ≤ 150' S ≤ 30°
ABUTMENT TYPES			

Figure 12.7-1
Recommended Guide for Abutment Type Selection

Where:

S = Skew

AL = Abutment Length

F = Fixed seat

SE = Semi-Expansion seat

E = Expansion seat

L = Length of continuous superstructure between abutments

a.) Type A1 fixed abutments are not used when wing piles are required. The semi-expansion seat is used to accommodate superstructure movements and to minimize cracking between the wings and body wall. See Standards for Abutment Type A1 (Integral Abutment) and Abutment Type A1 for additional guidance.

b.) Consider the flexibility of the piers when choosing this abutment type. Only one expansion bearing is needed if the structure is capable of expanding easily in one direction. With rigid piers, symmetry is important in order to experience equal expansion movements and to minimize the forces on the substructure units.

c.) For two-span prestressed girder bridges, the sill abutment is more economical than a semi-retaining abutment if the maximum girder length is not exceeded. It also is usually more economical if the next girder size is required.

d.) For two-span steel structures with long spans, the semi-retaining abutments may be more economical than sill abutments due to the shorter bridge lengths if a deeper girder is required.

12.8 Abutment Design Loads and Other Parameters

This section provides a brief description of the application of abutment design loads, a summary of load modifiers, load factors and other design parameters used for abutment and wing wall design, and a summary of WisDOT abutment design policy items.

12.8.1 Application of Abutment Design Loads

An abutment is subjected to both horizontal and vertical loads from the superstructure. The number and spacing of the superstructure girders determine the number and location of the concentrated reactions that are resisted by the abutment. The abutment also resists loads from the backfill material and any water that may be present.

Although the vertical and horizontal reactions from the superstructure represent concentrated loads, they are commonly assumed to be distributed over the entire length of the abutment wall or stem that support the reactions. That is, the sum of the reactions, either horizontal or vertical, is divided by the length of the wall to obtain a load per unit length to be used in both the stability analysis and the structural design. This procedure is sufficient for most design purposes.

Approach loads are not considered in the example below. However, designers shall include vertical reactions from reinforced concrete approaches as they directly transmit load from the approaches to the abutment. Reinforced concrete approaches include the concrete approach slab system (refer to FDM 14-10-15) and the structural approach slab system (as described in this chapter).

The first step in computing abutment design loads is to compute the dead load reactions for each girder or beam. To illustrate this, consider a 60-foot simple span structure with a roadway width of 44 feet, consisting of steel beams spaced at 9 feet and carrying an HL-93 live loading.

The dead load forces, DC and DW, acting on the abutments shall include reactions from the superstructure. DC dead loads include structural components and nonstructural attachments, and DW dead loads include wearing surfaces and utilities. If the total DC dead load is 1.10 kips per foot of girder and the total DW dead load is 0.18 kips per foot of girder, then the dead load reaction per girder is computed as follows:

$$R_{DC} = (1.10\ K/ft)\left(\frac{60\ Feet}{2}\right) = 33.0\ kips$$

$$R_{DW} = (0.18\ K/ft)\left(\frac{60\ Feet}{2}\right) = 5.4\ kips$$

These dead loads are illustrated in Figure 12.8-1. The dead loads are equally distributed over the full length of the abutment.

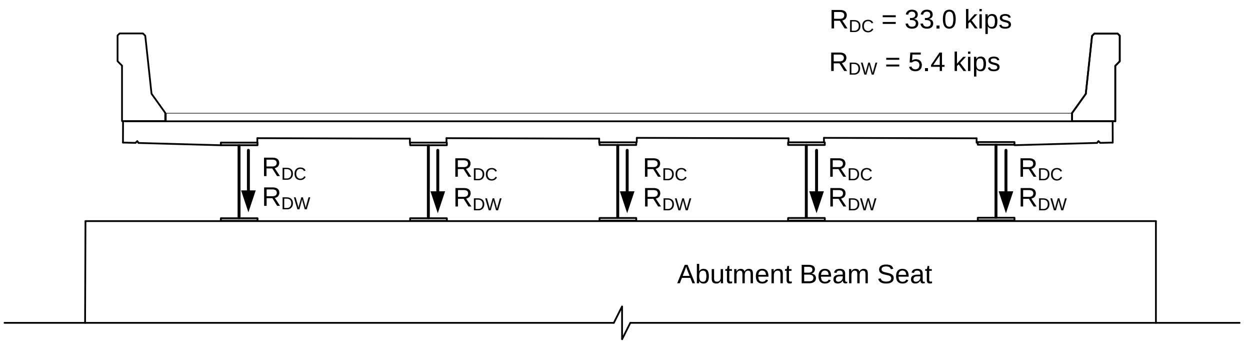


Figure 12.8-1
Dead Load on Abutment Beam Seat

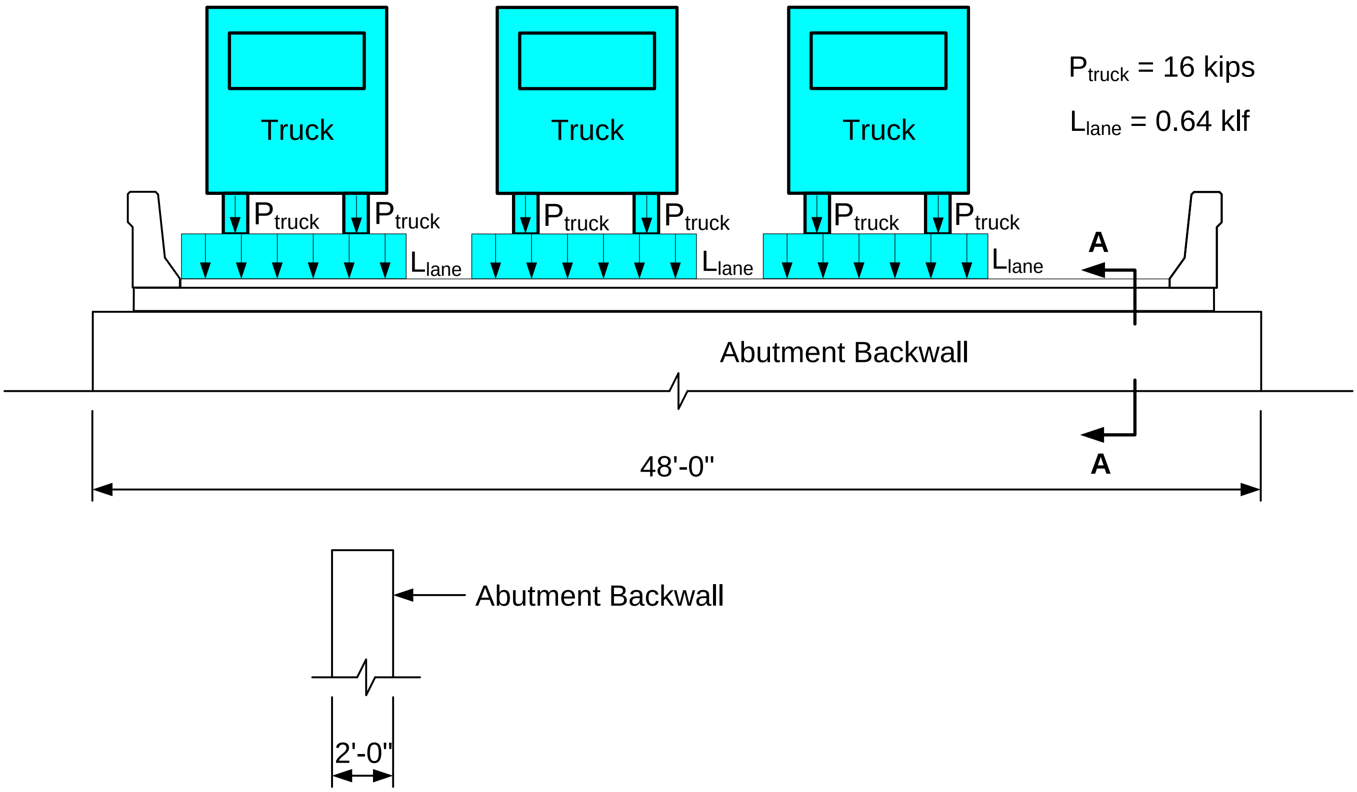
The next step is to compute the live load applied to the abutment. To compute live load reactions to bearings, live load distribution factors must be used to compute the maximum live load reaction experienced by each individual girder. However, to compute live loading on abutments, the maximum number of design lanes are applied to the abutment to obtain the live load per foot of length along the abutment. Live load distribution factors are not used for abutment design, because it is too conservative to apply the maximum live load reaction for each individual girder; each individual girder will generally not experience its maximum live load reaction simultaneously because each one is based on a different configuration of design lane locations.

To illustrate the computation of live loads for abutment design, consider the same 60-foot simple span bridge described previously. Since the roadway width is 44 feet, the maximum number of design lanes is three (44 / 12 = 3.67 ≈ 3 lanes). The backwall live load is computed by placing the three design truck axles along the abutment and calculating the load on a per foot basis. The dynamic load allowance and multiple presence factor shall be included. The load is applied to the entire length of the abutment backwall and is assumed to act at the front top corner (bridge side) of the backwall. This load is not applied, however, when designing the abutment wall (stem) or footing. Assuming an abutment length of 48 feet and a backwall width of 2.0 feet, the backwall live load is computed as follows:

$$R_{\text{LL backwall}} = \frac{(0.85)\left[(3\text{ lanes})\left(\frac{2\text{ wheels}}{\text{lane}}\right)\left(\frac{16\text{ kips}}{\text{wheel}}\right)(1.33) + (3\text{ lanes})(0.64\text{ klf})(2.0\text{ feet})\right]}{48\text{ feet}}$$

$$= 2.33\,\frac{\text{K}}{\text{ft}}$$

It should be noted that dynamic load allowance is applied to the truck live load only and not to the lane live load. This live load configuration on the abutment backwall is illustrated in Figure 12.8-2.



Section A-A

Figure 12.8-2
Live Load on Abutment Backwall

To compute the live loads applied to the abutment beam seat, the live load reactions should be obtained for one lane loaded using girder design software. For this example, for one design lane, the maximum truck live load reaction is 60.8 kips and the maximum lane live load reaction is 19.2 kips. In addition, assume that the abutment is relatively high; the load can therefore be distributed equally over the full length of the abutment. For wall (stem) design, the controlling maximum live loads applied at the beam seat are computed as follows, using three design lanes and using both dynamic load allowance and the multiple presence factor:

$$R_{LL\ stem} = \frac{(3\ \text{lanes})(0.85)\left[(60.8\ \text{kips})(1.33)+(19.2\ \text{kips})\right]}{48\ \text{feet}} = 5.32\ \frac{\text{K}}{\text{ft}}$$

This live load configuration for an abutment beam seat is illustrated in Figure 12.8-3.

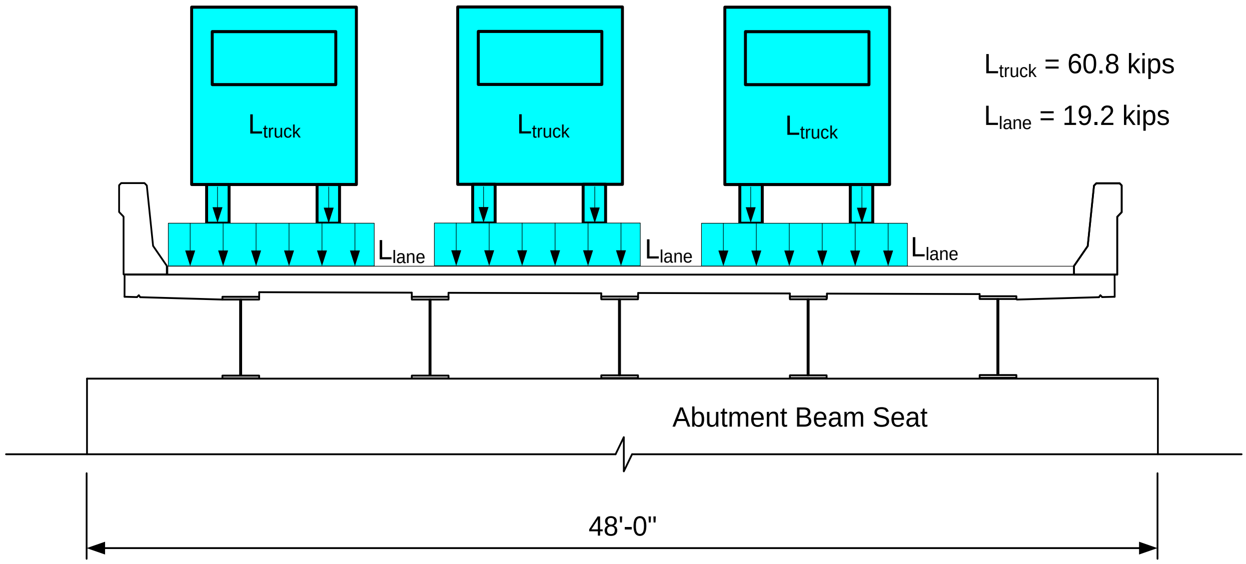


Figure 12.8-3
Live Load on Abutment Beam Seat

For a continuous bridge, the minimum live load applied to the abutment beam seat can be obtained based on the minimum (negative) live load reactions taken from girder design software output.

For footing design, the dynamic load allowance is not included. Therefore, the controlling maximum live loads applied at the beam seat are computed as follows:

$$R_{LL\ footing} = \frac{(3\text{ lanes})(0.85)[60.8\text{ kips} + 19.2\text{ kips}]}{48\text{ feet}} = 4.25\ \frac{K}{ft}$$

12.8.2 Load Modifiers and Load Factors

Table 12.8-1 presents the load modifiers used for abutment and wing wall design.

Description	Load Modifier
Ductility	1.00
Redundancy	1.00
Operational classification	1.00

Table 12.8-1
Load Modifiers Used in Abutment Design

Table 12.8-2 presents load factors used for abutment and wing wall design. Load factors presented in this table are based on the Strength I and Service I limit states. The load factors for WS and WL equal 0.00 for Strength I. Load factors for the Service I limit state for WS and WL are shown in the table below. Only apply these loads in the longitudinal direction.

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, WS	0.00	0.00	0.00
	Superstructure wind load, WS	0.00	0.00	1.00
	Superstructure wind on LL, WL	0.00	0.00	1.00
	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

* Use the minimum load factor for temperature and shrinkage unless checking for deformations.

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 without reinforced concrete approaches, the equivalent height of soil for vehicular loading on abutments shall be based on Table 12.8-3. For abutments with reinforced concrete approaches, one half of the equivalent height of soil shall be used to calculate the horizontal 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

12.8.5 Abutment and Wing Wall Design in Wisconsin

The standard details for abutments and wing walls were developed as an envelope of the loading conditions produced by the standard superstructure types, span lengths and geometric conditions presented in this manual. Prior BOS approval is required and special consideration should be given to designs that are outside of the limits presented in the standard details. The loading conditions, material properties and design methods presented in this chapter should be used for these special designs.

WisDOT policy items:

The resistance of the wing pile to horizontal forces should not be included in the calculations for the wing capacity.

The passive earth resistance can only be developed if there is significant movement of the wing. The soil under the wing may settle or otherwise erode. Therefore, the resistance of the soil friction and the passive earth pressure should not be utilized in resisting the forces on wing walls.

In computing the weight of the approach slab, assume there is settlement under the approach slab and place one-half of the weight of the slab on the abutment. An unfactored dead load value of 1.2 klf shall be used for concrete approach slabs and 2.0 klf for structural approach slabs. An unfactored live load value of 0.900 klf shall be applied to abutment approach slabs when used. Approach reactions shall act along the centroid of the foundation.

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

12.8.6 Horizontal Pile Resistance

The following procedure shall be used to verify the horizontal resistance of the piles for A3 abutments.

Given information:

Horizontal Loads	Unfactored (klf)		Load Factor		Factored Load (klf)
Earth Pressure	5.5	x	1.50	=	8.25
Live Load Surcharge	1.0	x	1.75	=	1.75
Temp. Load from Bearings	0.6	x	0.50	=	0.30
			Total, Hu	=	10.3

Back row pile spacing =	8.0 feet
Front row pile spacing =	5.75 feet
Ultimate Vertical Resistance, 12 3/4" CIP, Pr =	210 kips per pile
Factored Vertical Load on Front Row Pile*	160 kips per pile
Ultimate Horizontal Resistance of back row pile (from Geotech Report), Hr =	14 kips per pile
Ultimate Horizontal Resistance of front row pile (from Geotech Report), Hr =	11 kips per pile

* When calculating the horizontal component of the battered pile, use the actual factored load on the pile resulting from the loading conditions where the horizontal loads are maximized and the vertical loads are minimized.

Calculate horizontal component of the battered pile. The standard pile batter is 1:4.

$$Hr_{battered} = 160\left(\frac{1}{\sqrt{1^2 + 4^2}}\right)$$

$Hr_{battered}$ = 38.8 kips per pile

Calculate ultimate resistance provided by the pile configuration:

$$Hr = \left(\frac{14}{8.0}\right) + \left(\frac{11}{5.75}\right) + \left(\frac{38.8}{5.75}\right)$$

Hr = 10.4 klf

Hr > Hu = 10.3 klf   OK

12.9 Abutment Body Details

There are many different body sections that are utilized for each of the different abutment types. When designing these sections, it is inadvisable to use small and highly reinforced sections. As a general principle, it is better to use a lot of concrete and less reinforcing steel, thus making parts relatively massive and stiff. Adequate horizontal reinforcement and vertical contraction joints are essential to prevent cracking, especially when wing walls are poured monolithically with the abutment body.

The bottom of abutment bodies are normally constructed on a horizontal surface. However, abutments constructed on a horizontal surface may require one end of the body to be much higher than the opposite end due to the vertical geometry of the bridge. This sometimes requires an extremely long and high wing wall. For these extreme cases, the bottom of the abutment body can be stepped.

The berm in front of the body is held level even though the body is stepped. A minimum distance of 2.5 feet between the top of berm and the top of beam seat is allowed. Minimum ground cover as shown in the Standard Detail for Abutments must be maintained.

Stepping the bottom of the body may result in a longer bridge. This is usually more costly than holding the body level and using larger wings and beam seats. Stepped abutments are also more difficult to build. Engineering judgment must be exercised when determining if the bottom of the abutment should be level or stepped. Generally, if a standard wing wall design cannot be used, the bottom of the abutment body should be stepped.

12.9.1 Construction Joints

In a U-shaped abutment with no joint between the wings and the body, traffic tends to compact the fill against the three sides of the abutment. When the temperature drops, the abutment body concrete cannot shrink without tending to squeeze the warmer fill inside. The resistance of the fill usually exceeds the tensile or shearing strength of the body or wing, and cracks result.

If contraction joints are not provided in long abutment bodies, nature usually creates them. To prevent uncontrolled cracking in the body or cracking at the body-wing joint, body pours are limited to a maximum of 50 feet. Expansion joints are required at a maximum of 90 feet, as specified in **LRFD [11.6.1.6]**.

WisDOT exception to AASHTO:

LRFD [11.6.1.6] specifies that contraction joints shall be provided at intervals not exceeding 30 feet for conventional retaining walls and abutments. However, WisDOT has not experienced significant problems with 50 feet and uses a maximum interval of 50 feet.

Shear keys are provided in construction joints to allow the center pour to maintain the beneficial stabilizing effects from the wings. The shear keys enable the end pours, with their counterfort action due to the attached wing, to provide additional stability to the center pour. Reinforcing steel should be extended through the joint.

In general, body construction joints are keyed to hold the parts in line. Water barriers are used to prevent leakage and staining. Steel girder superstructures generally permit a small movement at construction joints without cracking the concrete slab. In the case of concrete slab or prestressed concrete girder construction, a crack will frequently develop in the deck above the abutment construction joint. The designer should consider this when locating the construction joint.

12.9.2 Beam Seats

Because of the bridge deck cross-slopes and/or skewed abutments, it is necessary to provide beam seats of different elevations on the abutment. The tops of these beam seats are poured to the plan elevations and are made level except when elastomeric bearing pads are used and grades are equal to or exceed 1%. For this case, the beam seat should be parallel to the bottom of girder or slab. Construction tolerances make it difficult to obtain the exact beam seat elevation.

When detailing abutments, the differences in elevations between adjacent beam seats are provided by sloping the top of the abutment between level beam seats. For steel girders, the calculation of beam seat elevations and use of shim plates at abutments, to account for thicker flanges substituted for plan flange thickness, is described on the Standard *Plate Girder Details* in Chapter 24.

See the abutment standards for additional reinforcing required when beam seats are 4” higher, or more, than the lowest beam seat.

12.10 Timber Abutments

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 galvanized nails to timber nailing strips which are bolted to the piling, or between the flanges of “HP” piles.

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.

The geotechnical engineer should evaluate approaches for settlement susceptibility and provide recommendations for mitigating settlements prior to approach placement. The bridge designer should determine if a structural approach slab is required and coordinate details with the roadway engineer. Usage of structural approach slabs is currently based on road functional classifications and considerations to traffic volumes (AADT), design speeds, and settlement susceptibility. Structural approach slabs are not intended to mitigate excessive approach settlements.

WisDOT policy item:

Structural approach slabs shall be used on all Interstate and US highway bridges. Structural approach slabs are recommended for bridges carrying traffic volumes greater than 3500 AADT in the future design year. Structural approach slabs are not required on buried structures and culverts. Structural approach slabs should not be used on rehabilitation projects, unless approved otherwise. Other locations can be considered with the approval of the Chief Structural Design Engineer. Design exceptions to structural approach slabs are considered on a project-by-project basis.

Standards for Structural Approach Slab for Type A1 and A3 Abutments and Structural Approach Slab Details for Type A1 and A3 Abutments are available for guidance.

This page intentionally left blank.

Table of Contents

13.1 General

Piers are an integral part of the load path between the superstructure and the foundation. Piers are designed to resist the vertical loads from the superstructure, as well as the horizontal superstructure loads not resisted by the abutments. The magnitude of the superstructure loads applied to each pier shall consider the configuration of the fixed and expansion bearings, the bearing types and the relative stiffness of all of the piers. The analysis to determine the horizontal loads applied at each pier must consider the entire system of piers and abutments and not just the individual pier. The piers shall also resist loads applied directly to them, such as wind loads, ice loads, water pressures and vehicle impact.

Bridges being designed with staged construction, whether new or rehabilitation, shall satisfy the requirements of LRFD (or LFD, if applicable) for each construction stage. Utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by Chief Structures Development Engineer at WisDOT.

WisDOT policy item:

Pier configurations shall be determined by providing the most efficient cast-in-place concrete pier design, unless approved otherwise. See 7.1.4.1.2 for policy guidance. Contact the Bureau of Structures Development Section for further guidance.

13.1.1 Pier Type and Configuration

Many factors are considered when selecting a pier type and configuration. The engineer should consider the superstructure type, the characteristics of the feature crossed, span lengths, bridge width, bearing type and width, skew, required vertical and horizontal clearance, required pier height, aesthetics and economy. For bridges over waterways, the pier location relative to the floodplain and scour sensitive regions shall also be considered.

The connection between the pier and superstructure is usually a fixed or expansion bearing which allows rotation in the longitudinal direction of the superstructure. This has the effect of eliminating longitudinal moment transfer between the superstructure and the pier. In rare cases when the pier is integral with the superstructure, this longitudinal rotation is restrained and moment transfer between the superstructure and the pier occurs. Pier types illustrated in the Standard Details shall be considered to be a pinned connection to the superstructure.

On grades greater than 2 percent, the superstructure tends to move downhill towards the abutment. The low end abutment should be designed as fixed and the expansion joint or joints placed on the uphill side or high end abutment. Consideration should also be given to fixing more piers than a typical bridge on a flat grade.

13.1.2 Bottom of Footing Elevation

The bottom of footing elevation for piers outside of the floodplain is to be a minimum of 4’ below finished ground line unless the footings are founded on solid rock. This requirement is intended to place the bottom of the footing below the frost line.

A minimum thickness of 2’-0” shall be used for spread footings and 2’-6” for pile-supported footings. Spread footings are permitted in streams only if they are founded on rock. Pile cap footings are allowed above the ultimate scour depth elevation if the piling is designed assuming the full scour depth condition.

The bottom of footing elevation for pile cap footings in the floodplain is to be a minimum of 6’ below stable streambed elevation. Stable streambed elevation is the normal low streambed elevation at a given pier location when not under scour conditions. When a pile cap footing in the floodplain is placed on a concrete seal, the bottom of footing is to be a minimum of 4’ below stable streambed elevation. The bottom of concrete seal elevation is to be a minimum of 8’ below stable streambed elevation. These requirements are intended to guard against the effects of scour.

13.1.3 Pier Construction

Except for pile encased piers (see Standard for Pile Encased Pier) and seal concrete for footings, all footing and pier concrete shall be placed in the dry. Successful underwater concreting requires special concrete mixes, additives and placement procedures, and the risk of error is high. A major concern in underwater concreting is that the water in which the concrete is placed will wash away cement and sand, or mix with the concrete, and increase the water-to-cement ratio. It was previously believed that if the lower end of the tremie is kept immersed in concrete during a placement, then the new concrete flows under and is protected by previously placed concrete. However, tests performed at the University of California at Berkeley show that concrete exiting a tremie pipe may exhibit many different flow patterns exposing more concrete to water than expected. A layer of soft, weak and water-laden mortar called laitance may also form within the pour. Slump tests do not measure shear resistance, which is the best predictor of how concrete will flow after exiting a tremie pipe.

Footing excavation adjacent to railroad tracks which falls within the critical zone shown on Standard for Highway Over Railroad Design Requirements requires an approved shoring system. Excavation, shoring and cofferdam costs shall be considered when evaluating estimated costs for pier construction, where applicable. Erosion protection is required for all excavations.

Table of Contents

$\Delta h1:L$ is the ratio of the difference in total vertical settlement between two points along the wall base to the horizontal distance between the two points(L). It should be noted that the tolerance provided in Table 14.4-3 are for guidance purposes only. More stringent tolerances may be required to meet project-specific requirements.

14.4.7.3 Overall Stability

Overall stability of the walls shall be checked at the Service I limit state using appropriate load combinations and resistance factors in accordance with **LRFD [11.6.2.3].** The stability is evaluated using limit state equilibrium methods. The Modified Bishop, Janbu or Spencer method may be used for the analysis. The analyses shall investigate all potential internal, compound and overall shear failure surfaces that penetrate the wall, wall face, bench, back-cut, backfill, and/or foundation zone. The overall stability check is performed by the Geotechnical Engineering Unit for WISDOT designed walls.

14.4.7.4 Internal Stability

Internal stability checks including anchor pullout or soil reinforcement failure and/or structural failure checks are also required as applicable for different wall systems. As an example, see Figure 14.4-11 for internal stability failure of MSE walls. Internal stability checks must be performed at Strength Limits in accordance with **LRFD [11.5.3].**

14.4.7.5 Wall Embedment

The minimum wall footing embedment shall be 1.5 ft below the lowest adjacent grade in front of the wall.

The embedment depth of most wall footings should be established below the depths the foundation soil/rock could be weakened due to the effect of freeze thaw, shrink-swell, scour, scour, erosion, construction excavation. The potential scour elevation shall be established in accordance with 11.2.2.1.1 of the Bridge Manual.

The final footing embedment depth shall be based on the required geotechnical bearing resistance, wall settlement limitations, and all internal, external, and overall (global) wall stability requirements in *AASHTO LRFD* and the *Bridge Manual*.

14.4.7.6 Wall Subsurface Drainage

Retaining wall drainage is necessary to prevent hydrostatic pressure and frost pressure. Inadequate wall sub-drainage can cause premature deterioration, reduced stability and collapse or failure of a retaining wall.

A properly designed wall sub-drainage system is required to control potentially damaging hydrostatic pressures and seepage forces behind and around a wall. A redundancy in the sub-drainage system is required where subsurface drainage is critical for maintaining retaining wall stability. This is accomplished using a pervious granular fill behind the wall.

Pipe underdrain must be provided to drain this fill. Therefore, “Pipe Underdrain Wrapped 6-Inch” is required behind all gravity retaining walls where seepage should be relieved. Gabion walls do not require a pipe drain system as these are porous due to rock fill. It is best to place the pipe underdrain at the top of the wall footing elevation. However, if it is not possible to discharge the water to a lower elevation, the pipe underdrain could be placed higher.

Pipe underdrains and weep holes may discharge water during freezing temperatures. In urban areas, this may create a problem due to the accumulation of flow and ice on sidewalks. Consideration should be given to connect the pipe underdrain to the storm sewer system.

14.4.7.7 Scour

The probable depth of scour shall be determined by subsurface exploration and hydraulic studies if the wall is located in flood prone areas. Refer to 11.2.2.1.1 for guidance related to scour vulnerability and design of walls. All walls with shallow foundations shall be founded below the scour elevation.

14.4.7.8 Corrosion

All metallic components of WISDOT retaining wall systems subjected to corrosion, should be designed to last through the designed life of the walls. Corrosion protection should be designed in accordance with the criteria given in **LRFD [11.10.6].** In addition, **LRFD [11.8.7] thru [11.10]** also include design guidance for corrosion protection on non-gravity cantilever walls, anchored walls and MSE walls respectively.

14.4.7.9 Utilities

Walls that have or may have future utilities in the backfill should minimize the use of soil reinforcement. MSE, soil nail, and anchored walls commonly have conflicts with utilities and should not be used when utilities must remain in or below the reinforced soil zone unless there is no other wall option. Utilities that are encapsulated by wall reinforcement may not be accessible for replacement or maintenance. Utility agreements should specifically address future access if wall reinforcing will affect access.

14.4.7.10 Guardrail and Barrier

Guardrail and barrier shall meet the requirements of the Chapter 30 - Railings, *Facilities Development Manual,* Standard Plans, and *AASHTO LRFD*. In no case shall guardrail be placed through MSE wall or reinforced slope soil reinforcement closer than 3 ft from the back of the wall facing elements. Furthermore, the guard rail posts shall be installed through the soil reinforcement in a manner that prevents ripping, damage and distortion of the soil reinforcement. In addition, the soil reinforcement shall be designed to account for the reduced cross-section resulting from the guardrail post holes.

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.

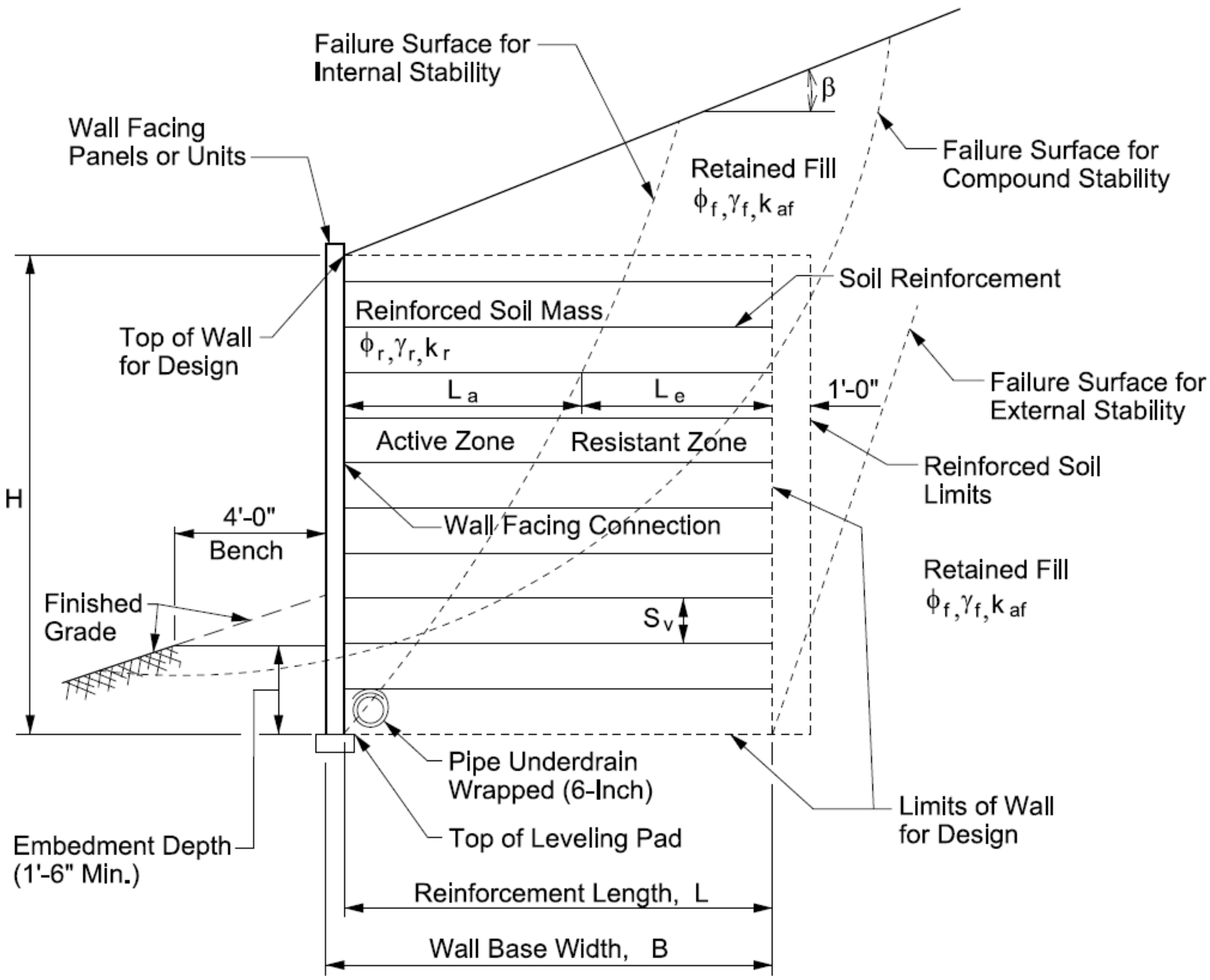


Figure 14.6-1
Structural Components of MSE Walls

14.6.2.1 Reinforced Earthfill Zone

The reinforced backfill to be used to construct the MSE wall shall meet the criteria in the wall specifications. The backfill shall be free from organics, or other deleterious material. It shall not contain foundry sand, bottom ash, blast furnace slag, or other potentially corrosive material. It shall meet the electrochemical criteria given in Table 14.6-1.

Table of Contents

15.1 Grade Separations

In general, there are three types of slope paving used at the abutments of grade separation bridges; cast-in-place concrete, bituminous stabilized crushed aggregate, and select crushed material. Concrete cast-in-place is used in urban areas or where appearance is a prime consideration. Bituminous stabilized crushed aggregate or select crushed material is used in rural areas or where appearance is not as important. Select crushed materials is the preferred slope paving type because of its low costs, durability and ease to repair. Refer to Slope Paving Structures standard details for additional information.

Precast concrete blocks (approximately 4 x 16 x 24 inches) were the standard applications during the late 50's and early 60's. Many blocks settled or washed out of place due to erosion of bedding under the blocks. They are no longer specified except on widening jobs to match existing slope paving.

Table of Contents

Strength I is used for the ultimate capacity of structural members and relates to the normal vehicular use of the bridge without wind.

Strength II is not typically used by WisDOT. However, Wisconsin Standard Permit Vehicle (Wis-SPV) must be checked in accordance with Chapter 45 – Bridge Rating.

Strength III is not typically used as a final-condition design check by WisDOT.

WisDOT policy item:

Strength III is used as a construction check for steel girder bridges with wind load but no live load. When checking this limit state during a deck pour, use a multiplier of 0.3 on the wind speed to account for the unlikelihood that a deck would be poured under extremely windy conditions.

Strength IV is not typically used by WisDOT. Spans > 300 ft. should include this limit state.

Strength V relates to the normal vehicular use of the bridge with wind speed (3-second gust) as specified in **LRFD [3.8]**. This limit state is used in the design of steel structures to check lateral bending stresses in the flanges.

17.2.3.2 Service Limit State

The service limit state shall be applied to restrict stress, deformation and crack width under regular service conditions. The total factored force effect must not exceed the factored resistance.

Service I relates to the normal vehicular use of the bridge. This limit state is used to check general serviceability requirements such as deflections and crack control. This load combination is also used to check compressive stresses in prestressed concrete components.

Service II is intended to control yielding of steel structures and slip of slip-critical connections due to vehicular live loads.

Service III is used to check the tensile stresses in prestressed concrete superstructures with the objective of crack control.

17.2.3.3 Fatigue Limit State

The fatigue limit state shall be applied as a restriction on stress range as a result of a single design truck occurring at the number of expected stress range cycles. The fatigue limit state is intended to limit crack growth under repetitive loads to prevent fracture during the design life of the bridge. The total factored force effect must not exceed the factored resistance.

Fatigue I is related to infinite load-induced fatigue life. This load combination should be checked for longitudinal slab bridge reinforcement and longitudinal continuity reinforcement on prestressed concrete girder and steel girder bridges. Fatigue I is used for steel girder structures to determine whether or not a tensile stress could exist at a particular location. This load combination is also used for any fracture-critical members as well as components and details not meeting the requirements for Fatigue II.

Fatigue II is related to finite load-induced fatigue life. If the projected 75-year single lane Average Daily Truck Traffic is less than or equal to a prescribed value for a given component or detail, that component or detail should be designed for finite life using the Fatigue II load combination.

17.2.3.4 Extreme Event Limit State

The extreme event limit state shall be applied for deck overhang design as specified in Table 17.6-1. For the extreme limit state, the applied loads for deck overhang design are horizontal and vertical vehicular collision forces. These forces are checked at the inside face of the barrier, the design section for the overhang and the design section for the first bay, as described in 17.6.

Extreme Event II is used to design deck reinforcement due to vehicular collision forces.

17.2.4 Design Loads

In LRFD design, structural materials must be able to resist their applied design loads. Two general types of design loads are permanent and transient. Permanent loads include dead load and earth load. Transient loads include live loads, wind, temperature, braking force and centrifugal force.

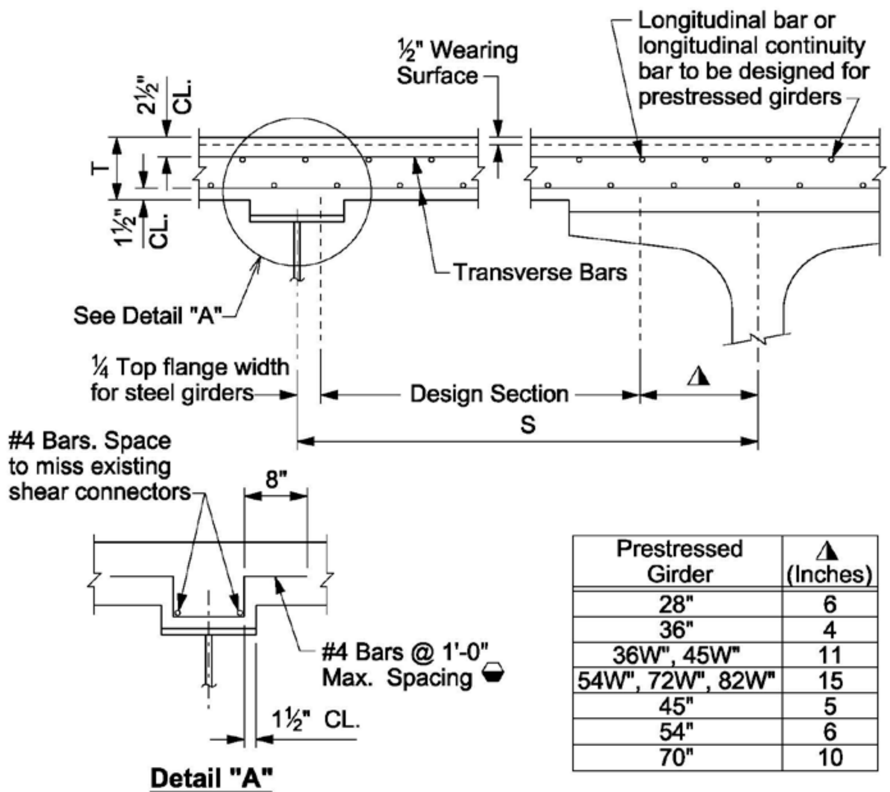
17.2.4.1 Dead Loads

Superstructures must be designed to resist dead load effects. In LRFD, dead load components consist of DC and DW dead loads. DC dead loads include structural components and nonstructural attachments, and DW dead loads include wearing surfaces and utilities. Different load factors are used for DC and DW dead loads, as described in 17.2.5, to account for the differences in the predictability of the loading. In addition, some dead loads are resisted by the non-composite section and other dead loads are resisted by the composite section.

Table 17.2-1 summarizes the various dead load components that are commonly included in beam-on-slab superstructure design. For slab structures, all loads presented in this table are resisted by the slab.

from the centerline of support. For prestressed concrete girders, this distance is equal to the values presented in Figure 17.5-1, along with bar locations and clearances.

Note: Transverse reinforcing steel requirements (bar size and spacing) are determined for both positive moment requirements and negative moment requirements, and the same reinforcing steel is used in both the top and bottom of slab as shown in Table 17.5-1 and Table 17.5-2. Longitudinal reinforcement in Table 17.5-3 and Table 17.5-4 is based on a percentage of the bottom transverse reinforcement required *by actual design calculations* (not a percentage of what is in the tables). **The tables should be used for deck reinforcement, with continuity bars in prestressed girder bridges being the only deck reinforcement requiring calculation.**



Prestressed Girder	▲ (Inches)
28"	6
36"	4
36W", 45W"	11
54W", 72W", 82W"	15
45"	5
54"	6
70"	10

Where the spacing of the existing shear connectors is equal to or less than 1'-0", spacing of #4 hat bars to match.
Where the spacing of the existing shear connectors is more than 1'-0", spacing of #4 hat bars to be 1'-0".

Figure 17.5-1
Transverse Section thru Slab on Girders

For skews of 20° and under, place transverse bars along the skew. For skews greater than 20°, place transverse bars perpendicular to the girders.

Detail "A", as presented in Figure 17.5-1, should be used for decks when shear connectors extend less than 2 inches into the slab on steel girder bridges or 3 inches on prestressed concrete girder bridges.

Several transverse reinforcing steel tables are provided in this chapter. The reinforcing steel in Table 17.5-1 and Table 17.5-2 does not account for deck overhangs. However, the minimum amount of reinforcing steel required in the deck overhangs is presented in various design tables in 17.6.

The reinforcement shown in Table 17.5-1 and Table 17.5-2 is based on both the Strength I requirement and crack control requirement.

Crack control was checked in accordance with **LRFD [5.7.3.4]**. The bar spacing cannot exceed the value from the following formula:

$$s \le \frac{700(\gamma)}{\beta_s f_s} - 2d_c$$

Where:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

γ	=	0.75 for decks
β_s	=	Ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face
f_s	=	Tensile stress in reinforcement at the service limit state (ksi) ≤ 0.6 f_y
d_c	=	Top concrete cover less ½ inch wearing surface plus ½ bar diameter or bottom concrete cover plus ½ bar diameter (inches)
h	=	Slab depth minus ½ inch wearing surface (inches)

WisDOT policy item:

The thickness of the sacrificial ½-inch wearing surface shall not be included in the calculation of d_c.

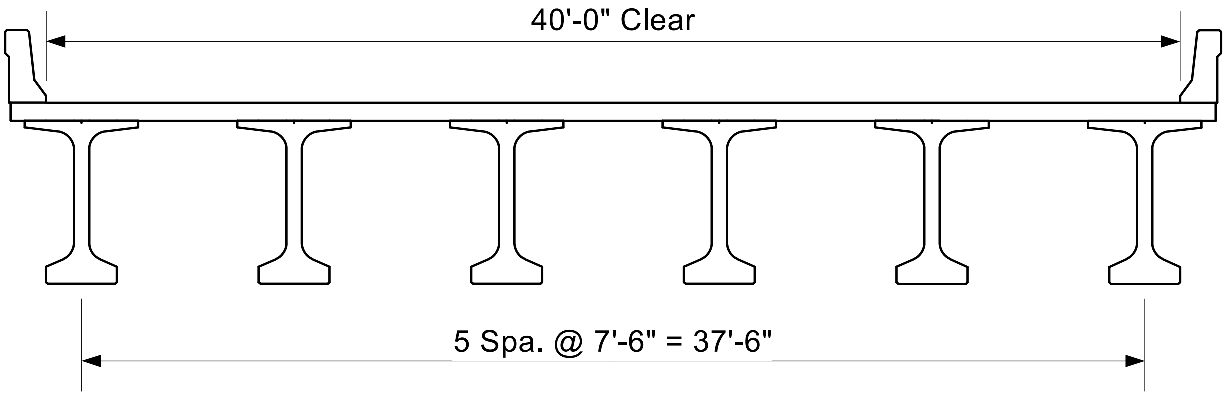
Table 17.5-1 and Table 17.5-2 were developed for specified values of the distance from the centerline of girder to the design section for negative moment. Those specified values – 0, 3, 6, 9, 12 and 18 inches – were selected to match values used in **AASHTO [Table A4-1]**. For a girder in which the distance from the centerline of girder to the design section for negative moment is not included in Table 17.5-1 and Table 17.5-2, the engineer may interpolate between the closest two values in the tables or can use the more conservative of the two values.

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 Seventh Edition - 2016 Interim)***

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

E19-1.2 Modulus of Elasticity of Beam and Deck Material

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

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

$$E_D := E_{deck4}$$

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

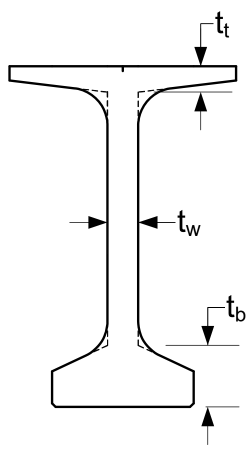
Note that this value of E_B is used for strength, composite section property, and long term deflection (deck and live load) calculations.

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

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

E19-1.3 Section Properties

72W Girder Properties:



$w_{tf} := 48$	in		
$t_t := 5.5$	in		
$t_w := 6.5$	in		
$t_b := 13$	in		
$ht := 72$	in		
$b_w := 30$	width of bottom flange, in		
$A_g := 915$	in^2		
$r_{sq} := 717.5$	in^2	$y_b := -34.87$	in
$I_g := 656426$	in^4	$S_t := 17680$	in^3
$y_t := 37.13$	in	$S_b := -18825$	in^3

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

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

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

E19-1.4 Girder Layout

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

$S := 8.5$ ft

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

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

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

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

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

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

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

E19-1.5 Loads

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

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

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

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

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

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

$w_p = 0.387$ weight of parapet, klf

E19-1.5.1 Dead Loads

Dead load on non-composite (DC):

exterior:

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

interior:

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

* Dead load on composite (DC):

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

* Wearing Surface (DW):

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

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

E19-1.5.2 Live Loads

For Strength 1 and Service 1 and 3:

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

tandem + lane

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

For Fatigue:

LRFD [5.5.3] states that fatigue of the reinforcement need not be checked for fully prestressed components designed to have extreme fiber tensile stress due to Service III Limit State within the tensile stress limit specified in **LRFD [Table 5.9.4.2.2-1]**.

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

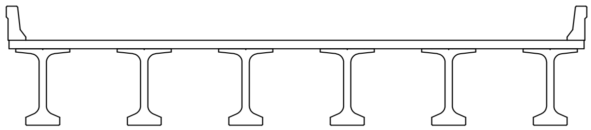
DLA of 15% applied to design truck with a 30 foot axle spacing.

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

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

E19-1.6 Load Distribution to Girders

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



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

For one Design Lane Loaded:

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

For Two or More Design Lanes Loaded:

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

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

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

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

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

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

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

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

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

E19-1.6.1 Distribution Factors for Interior Beams:

One Lane Loaded:

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

$g_{i1} = 0.435$

Two or More Lanes Loaded:

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

$g_{i2} = 0.636$

$$g_i := \max(g_{i1}, g_{i2})$$

$g_i = 0.636$

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

E19-1.6.2 Distribution Factors for Exterior Beams:

Two or More Lanes Loaded:

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

$$w_{parapet} := \frac{w_b - w}{2}$$

Width of parapet overlapping the deck

$w_{parapet} = 1.250$ ft

$$d_e := s_{oh} - w_{parapet}$$

Distance from the exterior web of exterior beam to the interior edge of parapet, ft.

$d_e = 1.250$ ft

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

Check range of applicability for de:

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

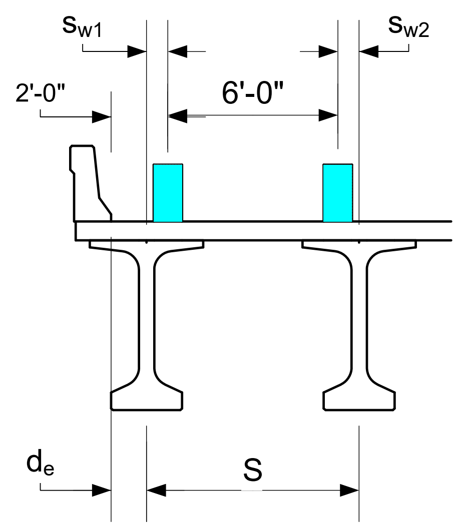
Note: While AASHTO allows the d_e value to be up to 5.5, the deck overhang (from the center of the exterior girder to the edge of the deck) is limited by WisDOT policy as stated in Chapter 17 of the Bridge Manual.

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

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

One Lane Loaded:

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



$s_{w1} := d_e - 2$ Distance from center of exterior girder to outside wheel load, ft. $\boxed{s_{w1} = -0.75}$ ft

$s_{w2} := S + s_{w1} - 6$ Distance from wheel load to first interior girder, ft. $\boxed{s_{w2} = 0.75}$ ft

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

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

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

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

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

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:

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

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 3.4.1-1 & Table 3.4.1-4]**:

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

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

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

$$= 1.0 \cdot \left[1.25 \cdot \left(M_{DLnc} + M_{DLc}\right) + 1.50 \cdot M_{DWc} + 1.75 \cdot M_{LL}\right] \qquad \boxed{M_{str} = 12449} \text{ kip-ft}$$

Service 1 (for compression checks)

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

$$= 1.0 \cdot \left[1.0 \cdot \left(M_{DLnc} + M_{DLc}\right) + 1.0 \cdot M_{DWc} + 1.0 \cdot M_{LL}\right] \qquad \boxed{M_{s1} = 8659} \text{ kip-ft}$$

Service 3 (for tension checks)

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

$$= 1.0 \cdot \left[1.0 \cdot \left(M_{DLnc} + M_{DLc}\right) + 1.0 \cdot M_{DWc} + 0.8 \cdot M_{LL}\right] \qquad \boxed{M_{s3} = 8045} \text{ kip-ft}$$

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

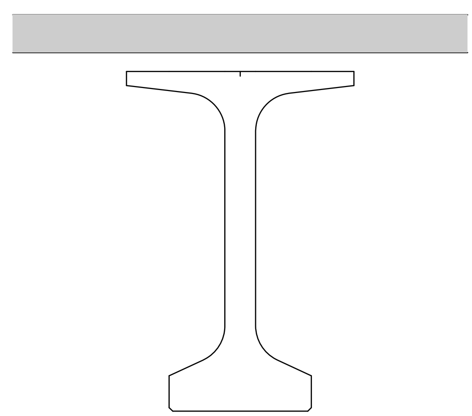
effective slab width; $\boxed{w_{eadj} = 58.46}$ in

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^3

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

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

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

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

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

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

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

Deck:

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

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

E19-1.9 Preliminary Design Information:

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

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

At transfer (Interior Girder):

$$M_{iend} := 0 \quad \text{kip-ft}$$

$$M_g := w_g \cdot \frac{L_g^{\;2}}{8} \qquad \boxed{M_g = 2574} \text{ kip-ft}$$

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

Estimated time dependant losses $\boxed{F_{Delta} := 32.0}$ ksi

Note: The estimated time dependant losses (for low relaxation strands) will be re-calculated using the approximate method in accordance with **LRFD [5.9.5.3]** once th number of strands has been determined.

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

Based on experience, assume $\Delta f_{pES\_est} := 18$ ksi loss from elastic shortening. As an alternate initial estimate, **LRFD [C.5.9.5.2.3a]** suggests assuming a 10% ES loss.

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

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

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

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

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

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

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

$ratio := 1 - \frac{loss_{\%}}{100}$ $\boxed{ratio = 0.753}$

$T = ratio \cdot T_o$

E19-1.10 Preliminary Design Steps

The following steps are utilized to design the prestressing strands:

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

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

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

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

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

E19-1.10.1 Determine Amount of Prestress

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

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

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

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

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

Stress at bottom due to prestressing (after losses):

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

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

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

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

OR:

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

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

desired bottom initial prestress (before losses)

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

The allowable tension, from **LRFD [5.9.4.2.2]**, is:

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

The desired bottom initial prestress (before losses):

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

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

$A_s = 0.217$ in^2

$f'_s := 270000$ psi

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

$$P := A_s \cdot \frac{f_s}{1000}$$ $\boxed{P = 43.943}$ kips

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

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

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

Solution:
Try $ns := 46$ strands, 0.6 inch diameter.
Initial prestress at bottom $f_{bpi} := 5.4858$ ksi,
Eccentricity, $e_s := -30.52$ inches; actual tension should be less than allowed.

E19-1.10.2 Prestress Loss Calculations

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

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

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

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

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

E19-1.10.2.1 Elastic Shortening Loss

at transfer (before ES loss) **LRFD [5.9.5.2]**

$T_{oi} := ns \cdot f_{tr} \cdot A_s$ $\boxed{= 46 \cdot 0.75 \cdot 270 \cdot 0.217 = 2021}$ kips

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

$T_o := \left(1 - \frac{ES_{loss}}{100}\right) \cdot T_{oi}$ $\boxed{T_o = 1842}$ kips

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

$f_{cgp} := \frac{T_o}{A_g} + (T_o \cdot e_s) \cdot \frac{e_s}{I_g} + M_g \cdot 12 \cdot \frac{e_s}{I_g}$ $\boxed{f_{cgp} = 3.190}$ ksi

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

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

$\Delta f_{pES} := \frac{E_p}{E_{ct}} \cdot f_{cgp}$ $\boxed{\Delta f_{pES} = 18.185}$ ksi

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

The initial stress in the strand is:

$$f_i := f_{tr} - \Delta f_{pES}$$ $\boxed{f_i = 184.315}$ ksi

The force in the beam after transfer is:

$$T_o := ns \cdot A_s \cdot f_i$$ $\boxed{T_o = 1840}$ kips

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

$$f_{ttr} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_t} + \frac{M_g \cdot 12}{S_t}$$ $\boxed{f_{ttr} = 0.582}$ ksi

$$f_{btr} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_b} + \frac{M_g \cdot 12}{S_b}$$ $\boxed{f_{btr} = 3.353}$ ksi

temporary allowable stress (compression) **LRFD [5.9.4.1.1]**:

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

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

E19-1.10.2.2 Approximate Estimate of Time Dependant Losses

Calculate the components of the time dependant losses; shrinkage, creep and relaxation, using the approximate method in accordance with **LRFD [5.9.5.3]**.

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

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

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

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

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

$$\Delta f_{pCR} := 10.0 \cdot \frac{f_{tr} \cdot A_s \cdot ns}{A_g} \cdot \gamma_h \cdot \gamma_{st}$$ $\boxed{\Delta f_{pCR} = 13.878}$ ksi

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

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

$\Delta f_{pLT} := \Delta f_{pCR} + \Delta f_{pSR} + \Delta f_{pRE}$ $\boxed{\Delta f_{pLT} = 23.816}$ ksi

The total estimated prestress loss (Approximate Method):

$\Delta f_p := \Delta f_{pES} + \Delta f_{pLT}$ $\boxed{\Delta f_p = 42.001}$ ksi

$$\boxed{\frac{\Delta f_p}{f_{tr}} \cdot 100 = 20.741}$$ % total prestress loss

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

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

$f_{pe} := f_{tr} - \Delta f_p$ $\boxed{f_{pe} = 160.50}$ ksi

$T := ns \cdot A_s \cdot f_{pe}$ $\boxed{T = 1602}$ kips

E19-1.10.3 Design of Strand Drape

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

top:

$$f_{tetr} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_t}$$ $\boxed{f_{tetr} = -1.165}$ ksi

high tension stress

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):

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

bottom:

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

$\boxed{f_{betr} = 4.994}$ ksi

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

high compressive stress

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

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

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

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

$$e_{sendt} := \frac{S_t}{T_o} \cdot \left(0 - \frac{T_o}{A_g}\right)$$

$\boxed{e_{sendt} = -19.32}$ inches or higher

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

$move := e_{sendt} - e_s$ $\boxed{move = 11.20}$ inches upward

Find the required position of the steel centroid to avoid high compression at the bottom of the beam. Set the bottom compression equal to the allowable stress and find where the centroid of $ns = 46$ strands needs to be:

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

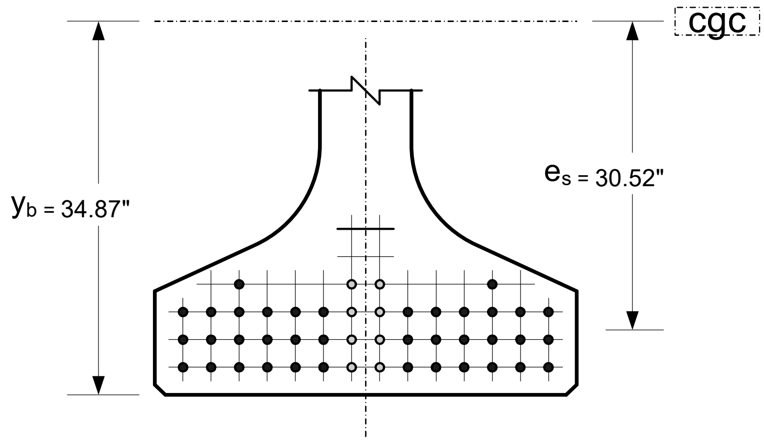
Set equal to allowed: $f_{betr} := f_{ciall}$

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

$\boxed{e_{sendb} = -24.65}$ inches or higher

Top stress condition controls:

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



38 undraped strands
8 draped strands

LRFD [Table 5.12.3-1] requires 2 inches of cover. However, WisDOT uses 2 inches to the center of the strand, and 2 inch spacing between centers.

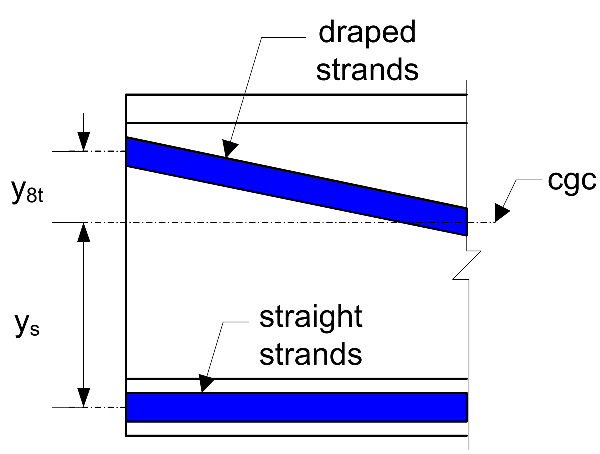
The center $ns_d := 8$ strands will be draped at the end of the girder.

Find the center of gravity of the remaining $ns_s = 38$ straight strands from the bottom of the girder:

$$Y_s := \frac{12 \cdot 2 + 12 \cdot 4 + 12 \cdot 6 + 2 \cdot 8}{ns_s}$$ $\boxed{Y_s = 4.21}$ inches from the bottom of the girder

OR:

$$y_s := y_b + Y_s$$ $\boxed{y_s = -30.66}$ inches from the center of gravity of the girder (cgc)



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

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

$$y_{8t} := \frac{ns \cdot e_{send} - ns_s \cdot y_s}{ns_d} \qquad \boxed{y_{8t} = 34.53}$$ inches above the cgc

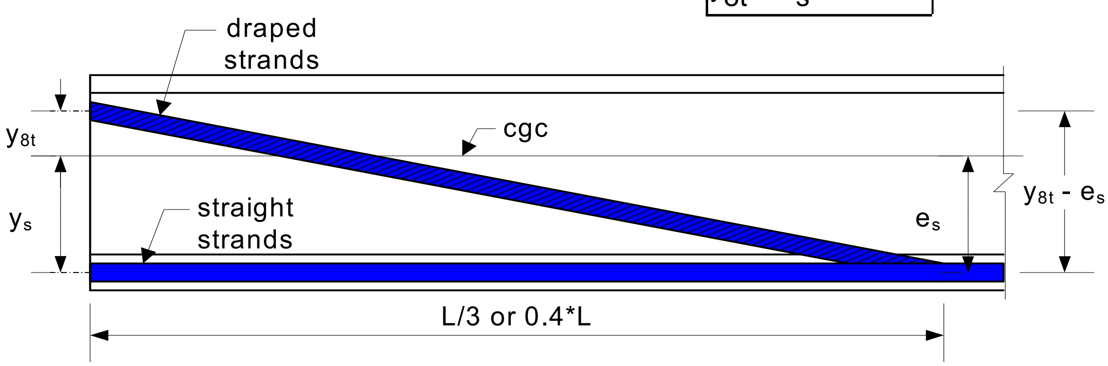
However, $y_t = 37.13$ inches to the top of the beam. If the draped strands are raised $y_{8t} = 34.53$ inches or more above the cgc, the stress will be OK.

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

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

$$\boxed{e_s = -30.52}$$ in

$$\boxed{y_{8t} - e_s = 62.65}$$ in



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

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

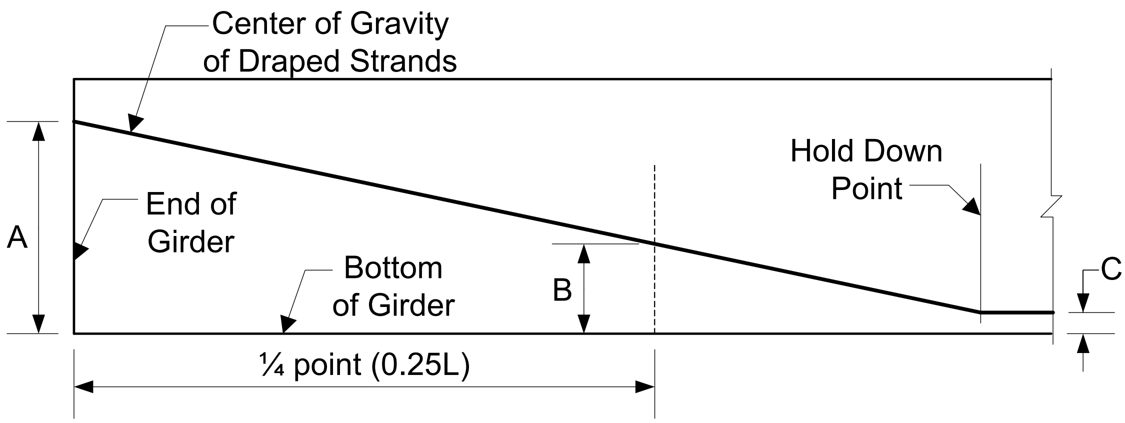
$$e_{8hd} := y_b + 5 \qquad \boxed{e_{8hd} = -29.870}$$ in

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

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

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

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



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

$\boxed{C := 5.00}$ in

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

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

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

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

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

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

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

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

Minimum moment on section = girder moment at the plant

The transfer length may be taken as:

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

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

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

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

$$e_{st} := \frac{ns_s \cdot y_s + 8 \cdot y_{8tt}}{ns}$$ $\boxed{e_{st} = -20.400}$ in

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

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

The girder stresses at the end of the transfer length:

$$f_{tt} := \frac{T_o}{A_g} + \frac{T_o \cdot e_{st}}{S_t} + \frac{M_{gt} \cdot 12}{S_t}$$ $\boxed{f_{tt} = 0.028}$ ksi

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

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

$$f_{bt} := \frac{T_o}{A_g} + \frac{T_o \cdot e_{st}}{S_b} + \frac{M_{gt} \cdot 12}{S_b}$$ $\boxed{f_{bt} = 3.873}$ ksi

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

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

E19-1.10.4 Stress Checks at Critical Sections

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

Data:

$\boxed{T_o = 1840}$ kips $\boxed{T = 1602}$ kips

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

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

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

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

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

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

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

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

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

$\boxed{e_{st} = -20.40}$ inches $\boxed{f_{tiall} = -0.200}$ ksi $\boxed{f_{ciall} = 4.420}$ ksi

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

Top of girder (Service 3):

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

$\boxed{f_{tei} = 0.028}$ ksi

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

Bottom of girder (Service 1):

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

$\boxed{f_{bei} = 3.873}$ ksi

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

Check at the girder and deck at midspan:

$e_s = -30.52$ inches

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

Top of girder stress (Service 3):

$$f_{ti} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_t} + \frac{M_g \cdot 12}{S_t}$$ $f_{ti} = 0.582$ ksi

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

Bottom of girder stress (Service 1):

$$f_{bi} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_b} + \frac{M_g \cdot 12}{S_b}$$ $f_{bi} = 3.353$ ksi

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

Final condition:

Allowable Stresses, **LRFD [5.9.4.2]**:

There are two compressive stress limits: (Service 1) **LRFD [5.9.4.2.1]**

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

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

(Service 3) **LRFD [5.9.4.2.2]** (Moderate Corrosion Condition)

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

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

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

Fatige compressive stress limit:

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

Top of girder stress (Service 1):

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

$\boxed{f_{t1} = 2.465}$ ksi

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

$\boxed{f_{t2} = 3.177}$ ksi

Is f_t less than f_{call}?

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

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

Top of girder stress (Fatigue 1):

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

$\boxed{f_{tfat} = 1.434}$ ksi

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

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

Bottom of girder stress (Service 3):

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

$\boxed{f_b = -0.302}$ ksi

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

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

Top of deck stress (Service 1):

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

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

$$f_{dt} := \frac{(M_{s1} - M_{nc}) \cdot 12}{S_{cgdt}}$$ $\boxed{f_{dt} = 0.800}$ ksi

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

<u>Bottom of deck stress (Service 1):</u>

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

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

Check at hold-down location:

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

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

$$f_{t3i} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_t} + \frac{M_{ghd} \cdot 12}{S_t}$$ $\boxed{f_{t3i} = 0.388}$ ksi

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

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

$$f_{b3i} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_b} + \frac{M_{ghd} \cdot 12}{S_b}$$ $\boxed{f_{b3i} = 3.535}$ ksi

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

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

<u>Top of girder stress (Service 1):</u>

$$f_{t3} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_t} + \frac{M_{nchd} \cdot 12}{S_t} + \frac{M_{1chd} \cdot 12}{S_{cgt}}$$ $\boxed{f_{t3} = 2.710}$ ksi

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

Top of girder stress (Fatigue 1):

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

$\boxed{f_{tfat} = 1.317}$ ksi

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

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

Bottom of girder stress (Service 3):

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

$\boxed{f_{b3} = 0.212}$ ksi

Is f_{b3} greater than f_{tall}?

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

Top of deck stress (Service 1):

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

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

$\boxed{f_{dt3} = 0.715}$ ksi

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

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

Bottom of deck stress (Service 1):

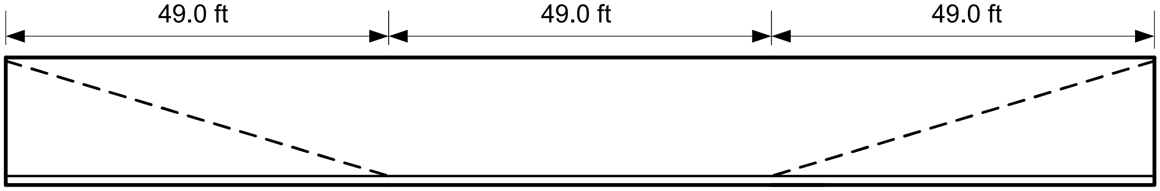
$$f_{db3} := \frac{(M_{1chd}) \cdot 12}{S_{cgdb}}$$

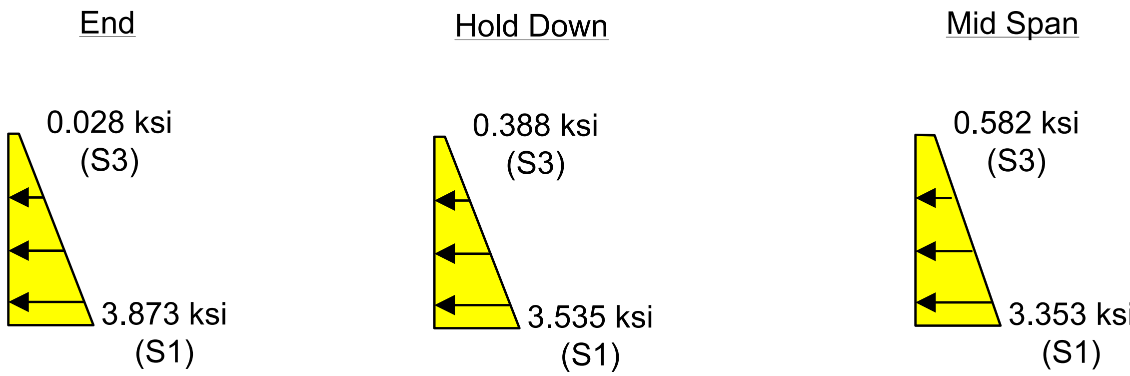
$\boxed{f_{db3} = 0.551}$ ksi

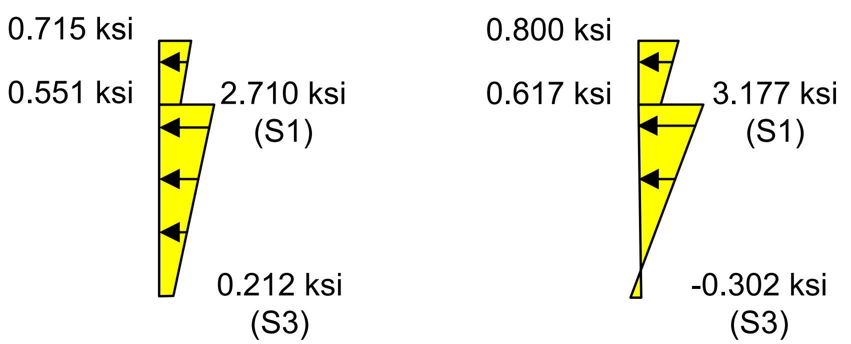
Is f_{db} less than f_{dall}?

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

Summary of Design Stresses:







Initial Allowable:

compression := f_{ciall} = 4.42 ksi

Final Allowable:

$compression_1$:= f_{call1} = 3.6 ksi

$compression_2$:= f_{call2} = 4.8 ksi

compression_fatigue := f_{call_fat} = 3.2 ksi

tension = f_{tall} = −0.537 ksi **All stresses are acceptable!**

E19-1.11 Calculate Jacking Stress

The fabricator is responsible for calculation of the jacking force. See **LRFD [5.9.3]** for equations for low relaxation strands.

E19-1.12 Flexural Strength Capacity at Midspan

Check f_{pe} in accordance with **LRFD [5.7.3.1.1]**:

$\boxed{f_{pe} = 160}$ 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:

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

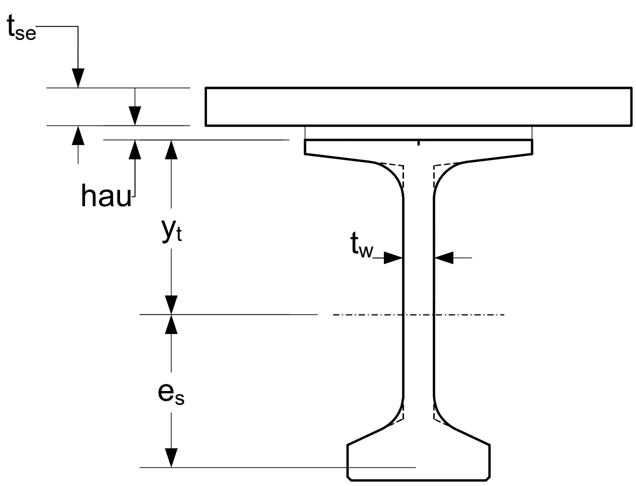
where:

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

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).

Assumed dimensions:



Assume that the compression block is in the deck. Calculate the capacity as if it is a rectangular section (with the compression block in the flange). The neutral axis location,calculated in accordance with **LRFD 5.7.3.1.1** for a rectangular section, is:

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

where:

$A_{ps} := ns \cdot A_s$ $\boxed{A_{ps} = 9.98}$ in^2

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

LRFD [5.7.2.2] $\alpha_1 := 0.85$ (for $f'_{cd} \le 10.0$ ksi)

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

$d_p := y_t + hau + t_{se} - e_s$ $\boxed{d_p = 77.15}$ in

$$c := \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 9.99}$ 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

$$c := \frac{A_{ps} \cdot f_{pu} - \alpha_1 \cdot f'_{cd} \cdot \left(b - w_{tf}\right) \cdot h_f}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot w_{tf} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 10.937}$ 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]; [5.7.3.2.2]**

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

$\boxed{M_n = 15717}$ 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

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

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

$f_{cpe} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_b}$ $\boxed{f_{cpe} = 4.348}$ 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

$M_{cr} := \gamma_3 \cdot \left[S_c \cdot \left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) \cdot \frac{1}{12} - M_{dnc} \cdot \left(\frac{S_c}{S_{nc}} - 1 \right) \right]$ $\boxed{M_{cr} = 10551}$ kip-ft

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

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:

LRFD [5.7.2.2] $\boxed{\alpha_1 = 0.850}$ $\boxed{\beta_1 = 0.850}$

$$c_x := \frac{A_{ps} \cdot f_{pu} - \alpha_1 \cdot f'_{cd} \cdot (b_x - w_{tf}) \cdot h_f}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot w_{tf} + k \cdot A_{ps} \cdot \dfrac{f_{pu}}{d_p}} \qquad \boxed{c_x = 13.51} \text{ in}$$

$$a_x := \beta_1 \cdot c_x \qquad \boxed{a_x = 11.49} \text{ in}$$

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:

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

$$\boxed{M_{n\_x} = 15515} \text{ kip-ft}$$

$$M_{r\_x} := \phi_f \cdot M_{n\_x} \qquad \boxed{M_{r\_x} = 15515} \text{ kip-ft}$$

$\boxed{1.33M_{strx} = 14874}$ 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:

$$g_{vi1} := 0.36 + \frac{S}{25}$$ $\boxed{g_{vi1} = 0.660}$

Two or more lanes loaded:

$$g_{vi2} := 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2$$ $\boxed{g_{vi2} = 0.779}$

$$g_{vi} := \max(g_{vi1}, g_{vi2})$$ $\boxed{g_{vi} = 0.779}$

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 distance from the centerline of the exterior beam to the inside edge of the parapet, $d_e = 1.25$ feet.

$$e_v := 0.6 + \frac{d_e}{10}$$ $\boxed{e_v = 0.725}$

$$g_{vx2} := e_v \cdot g_{vi}$$ $\boxed{g_{vx2} = 0.565}$

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}..

$$g_{vx1} := g_{x1} \quad = e \cdot g_i$$ $\boxed{g_{vx1} = 0.600}$

$g_{vx} := \max(g_{vx1}, g_{vx2})$ $\boxed{g_{vx} = 0.600}$

Apply the shear magnification factor in accordance with **LRFD [4.6.2.2.3c]**.

$$skew_{correction} := 1.0 + 0.2 \cdot \left(\frac{12L \cdot t_s^{\,3}}{K_g}\right)^{0.3} \cdot \tan\left(skew \cdot \frac{\pi}{180}\right)$$

$\boxed{L = 146.00}$

$\boxed{t_s = 8.00}$

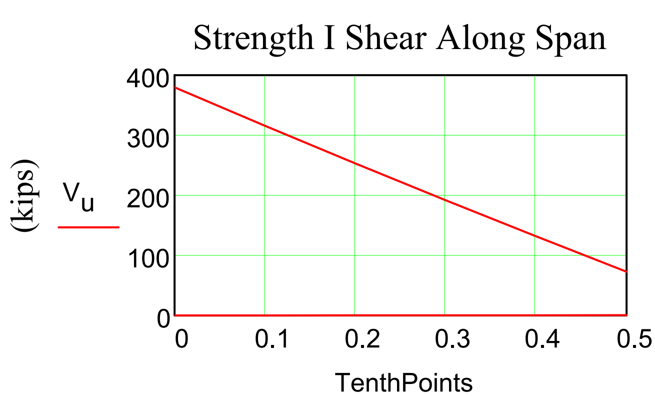
$\boxed{K_g = 3600866}$

$\boxed{skew = 20.000}$ $\boxed{skew_{correction} = 1.048}$

$g_{vx} := g_{vx} \cdot skew_{correction}$ $\boxed{g_{vx} = 0.629}$

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:

$$d_v := -e_s + y_t + hau + t_{se} - \frac{a}{2} \qquad \boxed{d_v = 72.50} \text{ in}$$

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:

$$L_{crit} := \left(\frac{w_{brg}}{2} + d_v\right)\cdot\frac{1}{12} + 0.5 \qquad \boxed{L_{crit} = 6.25} \text{ ft}$$

Calculate the eccentricity of the strand group at the critical section.

$$y_{8t\_crit} := y_{8t} - \frac{slope}{100}\cdot L_{crit}\cdot 12 \qquad \boxed{y_{8t\_crit} = 24.22} \text{ in}$$

$$e_{s\_crit} := \frac{ns_s\cdot y_s + ns_d\cdot y_{8t\_crit}}{ns_s + ns_d} \qquad \boxed{e_{s\_crit} = -21.11} \text{ in}$$

Calculation of compression stress block based on revised eccentricity:

$$d_{p\_crit} := y_t + hau + t_{se} - e_{s\_crit} \qquad \boxed{d_{p\_crit} = 67.74} \text{ in}$$

$$A_{ps\_crit} := (ns_d + ns_s)\cdot A_s \qquad \boxed{A_{ps\_crit} = 9.98} \text{ in}^2$$

Also, the value of f_{pu}, should be revised if the critical section is located less than the development length from the end of the beam. The development length for a prestressing strand is calculated in accordance with **LRFD [5.11.4.2]**:

$K := 1.6$ for prestressed members with a depth greater than 24 inches

$\boxed{d_b = 0.600}$ in

$$l_d := K \cdot \left(f_{ps} - \frac{2}{3} \cdot f_{pe}\right) \cdot d_b \qquad \boxed{l_d = 146.2} \text{ in}$$

The transfer length may be taken as: $l_{tr} := 60 \cdot d_b$ $\boxed{l_{tr} = 36.00}$ in

Since $L_{crit} = 6.250$ feet is between the transfer length and the development length, the design stress in the prestressing strand is calculated as follows:

$$f_{pu\_crit} := f_{pe} + \frac{L_{crit} \cdot 12 - l_{tr}}{l_d - l_{tr}} \cdot \left(f_{ps} - f_{pe}\right) \qquad \boxed{f_{pu\_crit} = 195} \text{ ksi}$$

For rectangular section behavior:

LRFD [5.7.2.2] $\boxed{\alpha_1 = 0.850}$ $\boxed{\beta_1 = 0.850}$

$$c := \frac{A_{ps\_crit} \cdot f_{pu\_crit}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps\_crit} \cdot \frac{f_{pu\_crit}}{d_{p\_crit}}} \qquad \boxed{c = 7.276} \text{ in}$$

$$a_{crit} := \beta_1 \cdot c \qquad \boxed{a_{crit} = 6.184} \text{ in}$$

Calculation of shear depth based on refined calculations of e_s and a:

$$d_{v\_crit} := -e_{s\_crit} + y_t + hau + t_{se} - \frac{a_{crit}}{2} \qquad \boxed{d_{v\_crit} = 64.65} \text{ in}$$

This value matches the assumed value of d_v above. OK!

The nominal shear resistance of the section is calculated as follows, **LRFD [5.8.3.3]**:

$$V_n = \min\left(V_c + V_s + V_p, 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p\right)$$

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, $L_{crit} = 6.25$ feet from the end of the girder at the abutment.

$V_d = 141$ kips

$V_i = 136$ kips

$M_{dnc} = 740$ kip-ft

$M_{max} = 837$ kip-ft

However, the equations below require the value of M_{max} to be in kip-in:

$M_{max} = 10048$ kip-in

$f_r = -0.20 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := -0.20 \cdot \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = -0.566}$ ksi

$\boxed{T = 1602}$ kips

$$f_{cpe} := \frac{T}{A_g} + \frac{T \cdot e_{s\_crit}}{S_b}$$ $\boxed{f_{cpe} = 3.548}$ ksi

$\boxed{M_{dnc} = 740}$ kip-ft

$\boxed{M_{max} = 10048}$ kip-in

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

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

$$M_{cre} := S_c \cdot \left(f_r - f_{cpe} - \frac{12M_{dnc}}{S_{nc}}\right)$$ $\boxed{M_{cre} = 89892}$ kip-in

Calculate V_{ci}, **LRFD [5.8.3.4.3]** λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$V_{ci1} := 0.06 \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v$ $\boxed{V_{ci1} = 71.7}$ kips

$$V_{ci2} := 0.02 \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v + V_d + \frac{V_i \cdot M_{cre}}{M_{max}}$$ $\boxed{V_{ci2} = 1384.0}$ kips

$V_{ci} := \max(V_{ci1}, V_{ci2})$ $\boxed{V_{ci} = 1384.0}$ kips

$$f_t := \frac{T}{A_g} + \frac{T \cdot e_{s\_crit}}{S_t} + \frac{M_{dnc} \cdot 12}{S_t}$$ $\boxed{f_t = 0.340}$ ksi

$$f_b := \frac{T}{A_g} + \frac{T \cdot e_{s\_crit}}{S_b} + \frac{M_{dnc} \cdot 12}{S_b}$$ $\boxed{f_b = 3.076}$ ksi

$\boxed{y_{cgb} = -48.76}$ in

$\boxed{ht = 72.00}$ in

$$f_{pc} := f_b - y_{cgb} \cdot \frac{f_t - f_b}{ht}$$ $\boxed{f_{pc} = 1.223}$ ksi

$$V_{p\_cw} := ns_d \cdot A_s \cdot f_{pe} \cdot \frac{slope}{100}$$ $\boxed{V_{p\_cw} = 29.4}$ kips

Calculate V_{cw}, **LRFD [5.8.3.4.3]** λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$V_{cw} := \left(0.06 \cdot \lambda \sqrt{f'_c} + 0.30 \cdot f_{pc}\right) \cdot b_v \cdot d_v + V_{p\_cw}$ $\boxed{V_{cw} = 256.1}$ kips

$V_c := \min\left(V_{ci}, V_{cw}\right)$ $\boxed{V_c = 256.1}$ kips

Calculate the required shear resistance:

$\phi_v := 0.9$ **LRFD [5.5.4.2]**

$V_{u\_crit} = \gamma st_{DC} \cdot \left(V_{DCnc} + V_{DCc}\right) + \gamma st_{DW} \cdot V_{DWc} + \gamma st_{LL} \cdot Vu_{LL}$ where,

$V_{DCnc} = 123.357$ kips $V_{DCc} = 8.675$ kips $V_{DWc} = 8.967$ kips $Vu_{LL} = 100.502$ kips

$V_{u\_crit} = 354.368$ kips $V_n := \frac{V_{u\_crit}}{\phi_v}$ $\boxed{V_n = 393.7}$ kips

Transverse Reinforcing Design at Critical Section:

The required steel capacity:

$V_s := V_n - V_c - V_p$ $\boxed{V_s = 137.7}$ kips

$A_v := 0.40$ in^2 for #4 rebar

$f_y := 60$ ksi

$\boxed{d_v = 65.00}$ in

$$\cot\theta := \begin{cases} 1 & \text{if } V_{ci} < V_{cw} \\ \min\left(1.0 + 3 \cdot \frac{f_{pc}}{\sqrt{f'_c}}, 1.8\right) & \text{otherwise} \end{cases}$$

$\boxed{\cot\theta = 1.800}$

$V_s = A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s}$ **LRFD Eq 5.8.3.3-4** reduced per **C5.8.3.3-1** when α = 90 degrees.

$s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{V_s}$ $\boxed{s = 20.399}$ in

Check Maximum Spacing, **LRFD [5.8.2.7]**:

$v_u := \frac{V_{u\_crit}}{\phi_v \cdot b_v \cdot d_v}$ $\boxed{v_u = 0.932}$ ksi

Max. stirrup spacing per WisDOT policy item is 18" $\boxed{0.125 \cdot f'_c = 1.000}$ ksi

$$s_{max1} := \begin{cases} \min(0.8 \cdot d_v, 18) & \text{if } v_u < 0.125 \cdot f'_c \\ \min(0.4 \cdot d_v, 12) & \text{if } v_u \geq 0.125 \cdot f'_c \end{cases} \qquad \boxed{s_{max1} = 18.00} \text{ in}$$

Check Minimum Reinforcing, **LRFD [5.8.2.5]**:

$$s_{max2} := \frac{A_v \cdot f_y}{0.0316 \cdot \lambda \sqrt{f'_c} \cdot b_v}$$

λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$$\boxed{s_{max2} = 41.31} \text{ in}$$

$$s_{max} := \min(s_{max1}, s_{max2}) \qquad \boxed{s_{max} = 18.00} \text{ in}$$

Therefore use a maximum spacing of $s := 18$ inches.

$$V_s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s} \qquad \boxed{V_s = 156} \text{ kips}$$

Check V_n requirements:

$$Vn_1 := V_c + V_s + V_p \qquad \boxed{Vn_1 = 412} \text{ kips}$$

$$Vn_2 := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p \qquad \boxed{Vn_2 = 845} \text{ kips}$$

$$V_n := \min(Vn_1, Vn_2) \qquad \boxed{V_n = 412} \text{ kips}$$

$$V_r := \phi_v \cdot V_n \qquad \boxed{V_r = 370.88} \text{ kips}$$

$$\boxed{V_{u\_crit} = 354.37} \text{ kips}$$

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: $\boxed{V_{u\_crit} = 354}$ $\boxed{0.5 \cdot \phi_v \cdot (V_c + V_p) = 115}$

From calculations similar to those shown above,

At hold down point: $\boxed{V_{u\_hd} = 172}$ $\boxed{0.5 \cdot \phi_v \cdot (V_{c\_hd} + V_p) = 64}$

At mid-span: $\boxed{V_{u\_mid} = 73}$ $\boxed{0.5 \cdot \phi_v \cdot (V_{c\_mid} + V_p) = 36}$

Therefore, use web reinforcing over the entire beam.

Resulting Shear Design:

Use #4 U shaped stirrups at 18-inch spacing between the typical end sections. Unless a large savings in rebar can be realized, use a single stirrup spacing between the standard end sections.

E19-1.14 Longitudinal Tension Flange Capacity:

The 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:

$$T_{ps} := \frac{M_{max}}{d_v \cdot \phi_f} + \left(\frac{V_{u\_crit}}{\phi_v} - 0.5 \cdot V_s - V_{p\_cw} \right) \cdot \cot\theta \qquad \boxed{T_{ps} = 670} \text{ kips}$$

actual capacity of the straight strands:

$$\boxed{ns_s \cdot A_s \cdot f_{pu\_crit} = 1612} \text{ kips}$$

Is the capacity of the straight strands greater than T_{ps}? $\boxed{check = \text{"OK"}}$

Check the tension Capacity at the edge of the bearing:

The strand is anchored $l_{px} := 10$ inches. The transfer and development lengths for a prestressing strand are calculated in accordance with **LRFD [5.11.4.2]**:

$$\boxed{l_{tr} = 36.00} \text{ in}$$

$$\boxed{l_d = 146.2} \text{ in}$$

Since l_{px} is less than the transfer length, the design stress in the prestressing strand is calculated as follows:

The assumed crack plane crosses the centroid of the straight strands at

$$l_{px'} := l_{px} + Y_s \cdot \cot\theta \qquad \boxed{Y_s = 4.21} \text{ in} \qquad \boxed{l_{px'} = 17.58} \text{ in}$$

$$f_{pb} := \frac{f_{pe} \cdot l_{px'}}{60 \cdot d_b} \qquad \boxed{f_{pb} = 78.37} \text{ ksi}$$

Tendon capacity of the straight strands: $\boxed{ns_s \cdot A_s \cdot f_{pb} = 646}$ kips

The values of V_u, V_s, V_p and θ may be taken at the location of the critical section.

Over the length d_v, the average spacing of the stirrups is:

$$s_{ave} := \frac{6 \cdot 4.25 + 6 \cdot 5.5}{12}$$ $\boxed{s_{ave} = 4.88}$ in

$$V_s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s_{ave}}$$ $\boxed{V_s = 576}$ kips

The vertical component of the draped strands is: $\boxed{V_{p\_cw} = 29}$ kips

The factored shear force at the critical section is: $\boxed{V_{u\_crit} = 354}$ kips

Minimum capacity required at the front of the bearing:

$$T_{breqd} := \left(\frac{V_{u\_crit}}{\phi_v} - 0.5 \cdot V_s - V_{p\_cw}\right) \cdot \cot\theta$$ $\boxed{T_{breqd} = 137}$ kips

Is the capacity of the straight strands greater than T_{breqd}? $\boxed{check = \text{"OK"}}$

E19-1.15 Composite Action - Design for Interface Shear Transfer

The total shear to be transferred to the flange between the end of the beam and mid-span is equal to the compression force in the compression block of the flange and haunch in strength condition. For slab on girder bridges, the shear interface force is calculated in accordance with **LRFD [5.8.4.2]**.

$b_{vi} := 18$ in width of top flange available to bond to the deck

$\boxed{d_v = 65.00}$ in

$$v_{ui} := \frac{V_{u\_crit}}{b_{vi} \cdot d_v}$$ $\boxed{v_{ui} = 0.303}$ ksi

$$V_{ui} := v_{ui} \cdot 12 \cdot b_{vi}$$ $\boxed{V_{ui} = 65.4}$ kips/ft

$$V_n = c \cdot A_{cv} + \mu \cdot \left(A_{vf} \cdot f_y + P_c\right)$$ **LRFD [5.8.4.1]**

The nominal shear resistance, V_n, used in design shall not be greater than the lesser of:

$$V_{n1} = K_1 \cdot f'_{cd} \cdot A_{cv} \quad \text{or} \quad V_{n2} = K_2 \cdot A_{cv}$$

$c := 0.28$ ksi

$\mu := 1.0$

$K_1 := 0.3$

$K_2 := 1.8$

$A_{cv} := b_{vi} \cdot 12$ Area of concrete considered to be engaged in interface shear transfer. $\boxed{A_{cv} = 216}$ in^2/ft

For an exterior girder, P_c is the weight of the deck, haunch, parapet and FWS.

$$P_{cd} := \frac{w_c \frac{t_s}{12}}{2 \cdot S} \cdot (S + s_{oh})^2 \qquad \boxed{P_{cd} = 0.667} \text{ klf}$$

$$P_{ch} := \frac{hau \cdot w_{tf}}{12^2} \cdot w_c \qquad \boxed{P_{ch} = 0.100} \text{ klf}$$

$$P_{cp} := w_p \qquad \boxed{P_{cp} = 0.129} \text{ klf}$$

$$P_{cfws} := w_{ws} \qquad \boxed{P_{cfws} = 0.133} \text{ klf}$$

$$P_c := P_{cd} + P_{ch} + P_{cp} + P_{cfws} \qquad \boxed{P_c = 1.029} \text{ klf}$$

From earlier calculations, the maximum #4 stirrup spacing used is $s = 18.0$ inches.

$$A_{vf} := \frac{A_v}{s} \cdot 12 \qquad \boxed{A_{vf} = 0.267} \text{ in}^2\text{/ft}$$

$$V_n := c \cdot A_{cv} + \mu \cdot (A_{vf} \cdot f_y + P_c) \qquad \boxed{V_n = 77.5} \text{ kips/ft}$$

$$V_{n1} := K_1 \cdot f'_{cd} \cdot A_{cv} \qquad \boxed{V_{n1} = 259.2} \text{ kips/ft}$$

$$V_{n2} := K_2 \cdot A_{cv} \qquad \boxed{V_{n2} = 388.8} \text{ kips/ft}$$

$$V_n := \min(V_n, V_{n1}, V_{n2}) \qquad \boxed{V_n = 77.5} \text{ kips/ft}$$

$$V_r := \phi_v \cdot V_n \qquad \boxed{V_r = 69.8} \text{ kips/ft}$$

$$\boxed{V_{ui} = 65.4} \text{ kips/ft}$$

Is V_r greater than V_{ui}? $\boxed{check = \text{"OK"}}$

Solution:
#4 stirrups spaced at $s = 18.0$ inches is adequate to develop the required interface shear

resistance for the entire length of the girder.

E19-1.16 Deflection Calculations

Check the Live Load deflection with the vehicle loading as specified in **LRFD [3.6.1.3.2]**; design truck alone or 25% of the design truck + the lane load.

The deflection shall be limited to L/800.

The moment of inertia of the entire bridge shall be used.

$$\Delta_{limit} := \frac{L \cdot 12}{800} \qquad \boxed{\Delta_{limit} = 2.190} \text{ inches}$$

$I_{cg} = 1203475.476$

$ng = 6$ number of girders

$$I_{bridge} := I_{cg} \cdot ng \qquad \boxed{I_{bridge} = 7220853} \text{ in}^4$$

From CBA analysis with 3 lanes loaded, the truck deflection controlled:

$$\Delta_{truck} := 0.648 \text{ in}$$

Applying the multiple presence factor from **LRFD Table [3.6.1.1.2-1]** for 3 lanes loaded:

$$\Delta := 0.85 \cdot \Delta_{truck} \qquad \boxed{\Delta = 0.551} \text{ in}$$

Is the actual deflection less than the allowable limit, $\Delta < \Delta$ limit? $\boxed{check = \text{"OK"}}$

E19-1.17 Camber Calculations

Moment due to straight strands:

Number of straight strands: $\boxed{ns_S = 38}$

Eccentricity of the straight strands: $y_s = -30.66$ in

$P_{i\_s} := ns_s \cdot A_s \cdot (f_{tr} - \Delta f_{pES})$ $P_{i\_s} = 1520$ kips

$M_1 := P_{i\_s} \cdot |y_s|$ $M_1 = 46598$ kip-in

Upward deflection due to straight strands:

Length of the girder: $L_g = 147$ ft

Modulus of Elasticity of the girder at release: $E_{ct} = 4999$ ksi

Moment of inertia of the girder: $I_g = 656426$ in^4

$$\Delta_s := \frac{M_1 \cdot L_g^{\,2}}{8 \cdot E_{ct} \cdot I_g} \cdot 12^2$$ $\Delta_s = 5.523$ in

Moment due to draped strands:

$P_{i\_d} := ns_d \cdot A_s \cdot (f_{tr} - \Delta f_{pES})$ $P_{i\_d} = 319.971$ kips

$A = 67.000$ in

$C = 5.000$ in

$M_2 := P_{i\_d} \cdot (A - C)$ $M_2 = 19838.175$ kip-in

$M_3 := P_{i\_d} \cdot (A - |y_b|)$ $M_3 = 10280.654$ kip-in

Upward deflection due to draped strands:

$$\Delta_d := \frac{L_g^{\,2}}{8 \cdot E_{ct} \cdot I_g} \cdot \left(\frac{23}{27} \cdot M_2 - M_3\right) \cdot 12^2$$ $\Delta_d = 0.784$ in

Total upward deflection due to prestress:

$\Delta_{PS} := \Delta_s + \Delta_d$ $\Delta_{PS} = 6.308$ in

Downward deflection due to beam self weight at release:

$$\Delta_{gi} := \frac{5 \cdot w_g \cdot L^4}{384 \cdot E_{ct} \cdot I_g} \cdot 12^3$$ $\Delta_{gi} = 2.969$ in

Anticipated prestress camber at release:

$\Delta_i := \Delta_{PS} - \Delta_{gi}$ $\boxed{\Delta_i = 3.339}$ in

The downward deflection due to the dead load of the deck and diaphragms:

Calculate the additional non-composite dead loads for an interior girder:

$w_{nc} := w_{dlii} - w_g$ $\boxed{w_{nc} = 0.881}$ klf

Modulus of Elasticity of the beam at final strength $\boxed{E_B = 6351}$ ksi

$$\Delta_{nc} := \frac{5 \cdot w_{nc} \cdot L^4}{384 \cdot E_B \cdot I_g} \cdot 12^3$$ $\boxed{\Delta_{nc} = 2.161}$ in

The downward deflection due to the dead load of the parapets is calculated as follows. Note that the deflections due to future wearing surface loads are not considered.

Calculate the composite dead loads for an interior girder:

$w_{ws} := 0$ klf

$w_c := w_p + w_{ws}$ $\boxed{w_c = 0.129}$ klf

$$\Delta_c := \frac{5 \cdot w_c \cdot L^4}{384 \cdot E_B \cdot I_{cg}} \cdot 12^3$$ $\boxed{\Delta_c = 0.173}$ in

The total downward deflection due to dead loads acting on an interior girder:

$\Delta_{DL} := \Delta_{nc} + \Delta_c$ $\boxed{\Delta_{DL} = 2.334}$ in

The residual camber for an interior girder:

$RC := \Delta_i - \Delta_{DL}$ $\boxed{RC = 1.005}$ in

This page intentionally left blank.

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 Seventh Edition - 2016 Interim. Note: Example has not been updated to current Bridge Manual guidance and should be used for informational purposes only)***

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:

$$E_{beam5} := 5500 \cdot \frac{\sqrt{f'_c \cdot 1000}}{\sqrt{6000}}$$ $\boxed{E_{beam5} = 5021}$ ksi

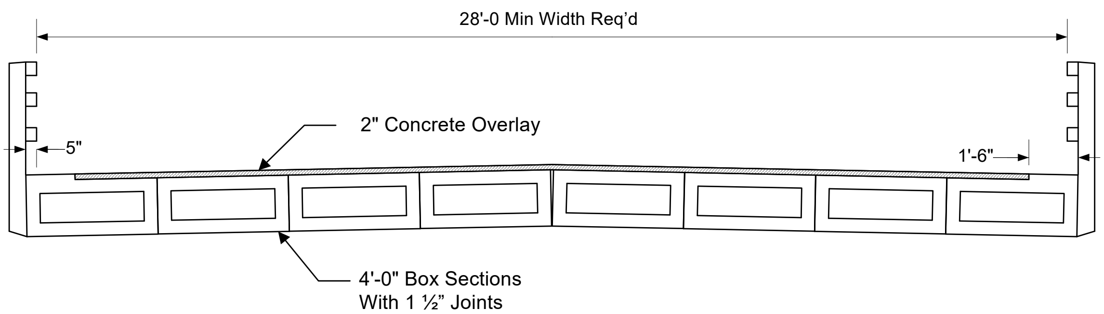
$E_B := E_{beam5}$

The modulus of elasticity at the time of release is calculated in accordance with **LRFD [C5.4.2.4]**.

$$E_{beam4.25} := 33000 \cdot K_1 \cdot w_c^{1.5} \cdot \sqrt{f'_{ci}}$$ $\boxed{E_{beam4.25} = 3952}$ ksi

$E_{ct} := E_{beam4.25}$

Based on the preliminary data, Section 19.3.9 of this chapter and Table 19.3-3, select a 4'-0" wide pretensioned box section having a depth of 1'-9" (Section 3), as shown on Bridge Manual Standard 19.15. The actual total deck width provided is calculated below.



$n_{beams} := 8$

$n_{joints} := n_{beams} - 1$ $\boxed{n_{joints} = 7}$

$W_s := 4$ Width of section, ft

$W_j := 1.5$ Width of joints, inches

Overall width of the bridge, ft

$$W_b := n_{beams} \cdot W_s + n_{joints} \cdot \frac{W_j}{12}$$ $\boxed{W_b = 32.875}$ feet

Clear width of the bridge, ft

$W_{b\_clear} := W_b - 2 \cdot Wh_{rail}$ $\boxed{W_{b\_clear} = 32.035}$ feet

$W_{curb} := 1.5$ Width of curb on exterior girder (for steel rails), feet

$S := W_s + \frac{W_j}{12}$ Effective spacing of sections $\boxed{S = 4.125}$ feet

<u>Section Properties, 4 ft x 1'-9" deep Box, Section 3</u>

$D_s := 1.75$ Depth of section, ft

$A := 595$ Area of the box girder, in^2

$t_w := 5$ Thickness of each vertical element, in

$r_{sq} := 55.175$ in^2

$y_t := 10.5$ in

$y_b := -10.5$ in

$S_t := 3137$ Section modulus, in^3

$S_b := -3137$ Section modulus, in^3

$I := 32942$ Moment of inertia, in^4

$J := 68601$ St. Venant's torsional inertia, in^4

E19-3.2 Live Load Distribution

The live load distribution for adjacent box beams is calculated in accordance with **LRFD [4.6.2.2.2]**. Note that if the section does not fall within the applicability ranges, the lever rule shall be used to determine the distribution factor.

E19-3.2.1 Distribution for Moment

For interior beams, the live load moment distribution factor is calculated as indicated in **LRFD [Table 4.6.2.2.2b-1]** for cross section type "g" if connected only enough to prevent relative vertical displacement. This distribution factor applies regardless of the number of lanes loaded.

$K := \sqrt{\frac{(1+\mu)\cdot I}{J}}$ $\boxed{K = 0.759}$

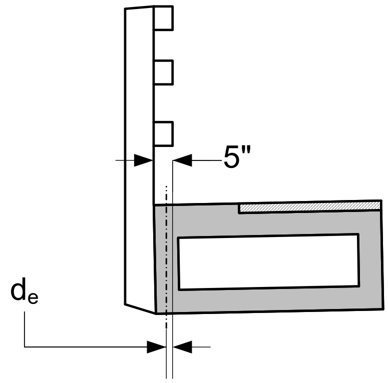
$C := \min\left[K\cdot\left(\frac{W_b}{L}\right), K\right]$ $\boxed{C = 0.567}$

When C is less than 5:

$D := 11.5 - N_L + 1.4\cdot N_L\cdot(1 - 0.2\cdot C)^2$ $\boxed{D = 11.701}$

$g_{int\_m} := \frac{S}{D}$ $\boxed{g_{int\_m} = 0.353}$

For exterior beams, the live load moment distribution factor is calculated as indicated in **LRFD [Table 4.6.2.2.2d-1]** for cross section type "g".



$$d_e := \frac{5}{12} \cdot \frac{1}{2} - Wh_{rail}$$

Distance from the center of the exterior web to the face of traffic barrier, ft.

$\boxed{d_e = -0.212}$ feet

For one design lane loaded:

$$e_1 := \max\left(1.125 + \frac{d_e}{30}, 1\right)$$ $\boxed{e_1 = 1.118}$

$$g_{ext1} := g_{int\_m} \cdot e_1$$ $\boxed{g_{ext1} = 0.394}$

For two or more design lanes loaded:

$$e_2 := \max\left(1.04 + \frac{d_e}{25}, 1\right)$$ $\boxed{e_2 = 1.032}$

$$g_{ext2} := g_{int\_m} \cdot e_2$$ $\boxed{g_{ext2} = 0.364}$

Use the maximum value from the above calculations to determine the controlling exterior girder distribution factor for moment.

$$\boxed{g_{ext\_m} := \max(g_{ext1}, g_{ext2})}$$ $\boxed{g_{ext\_m} = 0.394}$

The distribution factor for fatigue is the single lane distribution factor with the multi-presence factor, $m := 1.2$, removed:

$$g_f := \frac{g_{ext1}}{1.2}$$ $\boxed{g_f = 0.328}$

E19-3.2.2 Distribution for Shear

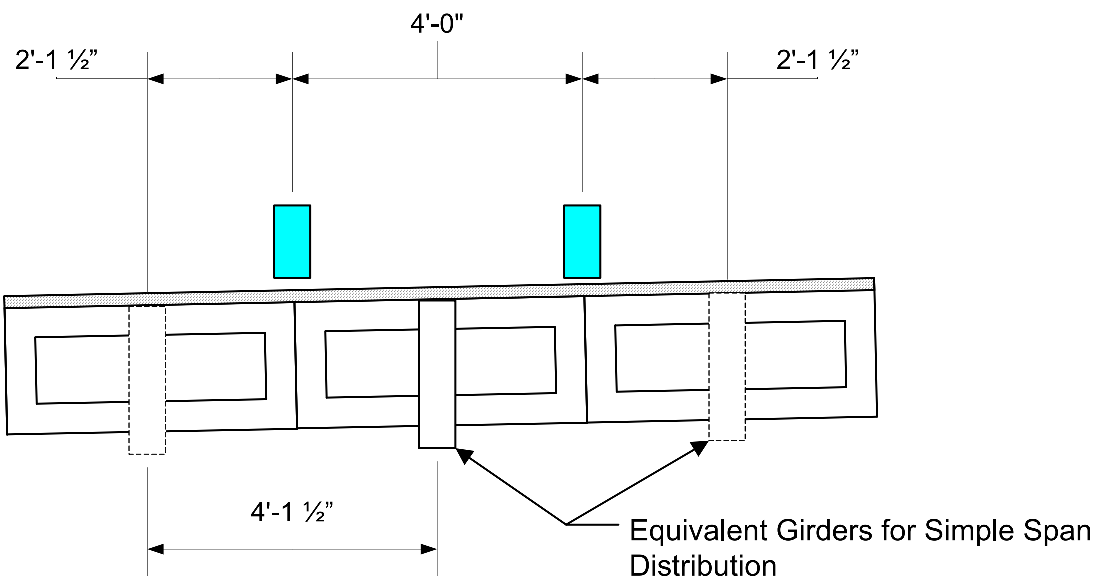
Interior Girder

This section does not fall in the range of applicability for shear distribution for interior girders of bridge type "g". I = 32942 in^4 and the limit is 40000 < I < 610,000, per **LRFD [Table 4.6.2.2.3a-1]**. Therefore, use the lever rule.

For the single lane loaded, only one wheel can be located on the box section. With the single lane multi presence factor, the interior girder shear distribution factor is:

$g_{int\_v1} := 0.5 \cdot 1.2$ $\boxed{g_{int\_v1} = 0.600}$

For two or more lanes loaded, center adjacent vehicles over the beam. One load from each vehicle acts on the beam.



$g_{int\_v2} := 0.5 \cdot \frac{2.125}{4.125} \cdot 2$ $\boxed{g_{int\_v2} = 0.515}$

$g_{int\_v} := \max(g_{int\_v1}, g_{int\_v2})$ $\boxed{g_{int\_v} = 0.600}$

Exterior Girder

For the exterior girder, the range of applicability of **LRFD [T-4.6.2.2.3b-1]** for bridge type "g" is satisfied.

For the single lane loaded:

$e_{v1} := \max\left(1.25 + \frac{d_e}{20}, 1.0\right)$ $\boxed{e_{v1} = 1.239}$

$g_{ext\_v1} := e_{v1} \cdot g_{int\_v1}$ $\boxed{g_{ext\_v1} = 0.744}$

For two or more lanes loaded:

$b := W_s \cdot 12$ $\boxed{b = 48}$ inches

$$e_{v2} := \max\left[1 + \left(\frac{d_e + \frac{b}{12} - 2.0}{40}\right)^{0.5}, 1.0\right] \qquad \boxed{e_{v2} = 1.211}$$

$$g_{ext\_v2} := e_{v2} \cdot g_{int\_v2} \qquad \boxed{g_{ext\_v2} = 0.624}$$

$$g_{ext\_v} := \max\left(g_{ext\_v1}, g_{ext\_v2}\right) \qquad \boxed{g_{ext\_v} = 0.744}$$

E19-3.3 Live Load Moments

The HL-93 live load moment per lane on a 44 foot span is controlled by the design tandem plus lane. The maximum value at mid-span, including a dynamic load allowance of 33%, is $M_{LL\_lane} := 835.84$ kip-ft per lane.

$$M_{LLint} := M_{LL\_lane} \cdot g_{int\_m} \qquad \boxed{M_{LLint} = 294.7} \quad \text{kip-ft}$$

$$M_{LLext} := M_{LL\_lane} \cdot g_{ext\_m} \qquad \boxed{M_{LLext} = 329.4} \quad \text{kip-ft}$$

The Fatige live load moment per lane on a 44 foot span at mid-span, including a dynamic load allowance of 15%, is $M_{LLfat\_lane} := 442.4$ kip-ft per lane.

$$M_{LLfat} := M_{LLfat\_lane} \cdot g_f \qquad \boxed{M_{LLfat} = 145.3} \quad \text{kip-ft}$$

E19-3.4 Dead Loads

Interior Box Girders

Box Girder $w_g := \frac{A}{12^2} \cdot w_c$ $\qquad \boxed{w_g = 0.620}$ klf

Internal Concrete Diaphragm (at center of span)

$$w_{diaph} := 1.17 \cdot \left(D_s - \frac{10}{12}\right) \cdot \left(W_s - \frac{10}{12}\right) \cdot w_c \qquad \boxed{w_{diaph} = 0.509} \quad \text{kips}$$

Equivalent uniform load: $w_{d\_mid} := 2 \cdot \frac{w_{diaph}}{L}$ $\qquad \boxed{w_{d\_mid} = 0.023}$ klf

Internal Concrete Diaphragm (at ends of span)

$$w_{diaph\_end} := 2.83 \cdot \left(D_s - \frac{10}{12}\right) \cdot \left(W_s - \frac{10}{12}\right) \cdot w_c \qquad \boxed{w_{diaph\_end} = 1.232} \text{ kips}$$

Equivalent uniform load:

$$w_{d\_end} := 8 \cdot \frac{w_{diaph\_end} \cdot 1.17}{L^2} \qquad \boxed{w_{d\_end} = 0.006} \text{ klf}$$

$$w_d := w_{d\_mid} + w_{d\_end} \qquad \boxed{w_d = 0.029} \text{ klf}$$

For the interior girders, all dead loads applied after the post tensioning has been completed are distributed equally to all of the girders.

Overlay

$$w_o := \frac{\frac{t_{overlay}}{12} \cdot \left(W_b - W_{curb} \cdot 2\right) \cdot w_c}{n_{beams}} \qquad \boxed{w_o = 0.093} \text{ klf}$$

Joint Grout

$$w_j := \frac{W_j}{12} \cdot \left(D_s + \frac{t_{overlay}}{12}\right) \cdot w_c \cdot \frac{n_{joints}}{n_{beams}} \qquad \boxed{w_j = 0.031} \text{ klf}$$

"M" Rail

$$w_r := \frac{2 \cdot w_{rail}}{n_{beams}} \qquad \boxed{w_r = 0.019} \text{ klf}$$

Future Wearing Surface

$$w_{fws} := \frac{W_b \cdot 0.020}{n_{beams}} \qquad \boxed{w_{fws} = 0.082} \text{ klf}$$

$$w_{DCint} := w_g + w_d + w_o + w_j + w_r \qquad \boxed{w_{DCint} = 0.792} \text{ klf}$$

$$w_{DWint} := w_{fws} \qquad \boxed{w_{DWint} = 0.082} \text{ klf}$$

Exterior Box Girders

Box Girder $w_{g\_ext} := \frac{A + 2 \cdot W_{curb} \cdot 12}{12^2} \cdot w_c$ $\boxed{w_{g\_ext} = 0.657}$ klf

Internal Concrete Diaphragms $\boxed{w_d = 0.029}$ klf

For the exterior girders, all dead loads applied directly to the girder are applied.

Overlay $w_{o\_ext} := \frac{t_{overlay}}{12} \cdot (S - W_{curb}) \cdot w_c$ $\boxed{w_{o\_ext} = 0.066}$ klf

Joint Grout $w_{j\_ext} := \frac{1}{2} \cdot \frac{W_j}{12} \cdot \left(D_s + \frac{t_{overlay}}{12}\right) \cdot w_c$ $\boxed{w_{j\_ext} = 0.018}$ klf

Type M Rail $w_{r\_ext} := w_{rail}$ $\boxed{w_{r\_ext} = 0.075}$ klf

Future Wearing Surface

$w_{fws\_ext} := S \cdot 0.020$ $\boxed{w_{fws\_ext} = 0.083}$ klf

$w_{DCext} := w_{g\_ext} + w_d + w_{o\_ext} + w_{j\_ext} + w_{r\_ext}$

$\boxed{w_{DCext} = 0.845}$ klf

$w_{DWext} := w_{fws\_ext}$ $\boxed{w_{DWext} = 0.083}$ klf

E19-3.5 Dead Load Moments

Moment of the girder and internal diaphragms alone, based on total girder length.

$M_{gi} := (w_g + w_d) \cdot \frac{L_g^2}{8}$ $\boxed{M_{gi} = 160.6}$ kip-ft

$M_{gext} := (w_{g\_ext} + w_d) \cdot \frac{L_g^2}{8}$ $\boxed{M_{gext} = 169.9}$ kip-ft

Interior Girder

$M_{DCint} := w_{DCint} \cdot \frac{L^2}{8}$ $\boxed{M_{DCint} = 191.8}$ kip-ft

$M_{DWint} := w_{DWint} \cdot \frac{L^2}{8}$ $\boxed{M_{DWint} = 19.9}$ kip-ft

Exterior Girder

$M_{DCext} := w_{DCext} \cdot \frac{L^2}{8}$ $\boxed{M_{DCext} = 204.5}$ kip-ft

$M_{DWext} := w_{DWext} \cdot \frac{L^2}{8}$ $\boxed{M_{DWext} = 20.0}$ kip-ft

E19-3.6 Design Moments

Calculate the total moments on the interior and exterior girders to determine which girder will control the design.

$M_{T\_int} := M_{DCint} + M_{DWint} + M_{LLint}$ $\boxed{M_{T\_int} = 506.3}$ kip-ft

$M_{T\_ext} := M_{DCext} + M_{DWext} + M_{LLext}$ $\boxed{M_{T\_ext} = 553.9}$ kip-ft

Since the Dead Load moments are very close and the exterior Live Load moments are greater than the interior moments, the exterior girder controls for this design example. Note: an interior box girder section design will not be provided in this example. However, the interior girder shall not have less load carrying capacity then the exterior girder.

$M_{DC} := M_{DCext}$ $\boxed{M_{DC} = 204.5}$ kip-ft

$M_{DW} := M_{DWext}$ $\boxed{M_{DW} = 20}$ kip-ft

$M_{LL} := M_{LLext}$ $\boxed{M_{LL} = 329.4}$ kip-ft

$M_{LLf} := M_{LLfat}$ $\boxed{M_{LLf} = 145.3}$ kip-ft

E19-3.7 Load Factors

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

	DC	DW	LL
Strength 1	$\gamma st_{DC} := 1.25$	$\gamma st_{DW} := 1.50$	$\gamma st_{LL} := 1.75$
Service 1	$\gamma s1_{DC} := 1.0$	$\gamma s1_{DW} := 1.0$	$\gamma s1_{LL} := 1.0$
Service 3	$\gamma s3_{DC} := 1.0$	$\gamma s3_{DW} := 1.0$	$\gamma s3_{LL} := 0.8$
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

$$M_{str} := \eta \cdot (\gamma st_{DC} \cdot M_{DC} + \gamma st_{DW} \cdot M_{DW} + \gamma st_{LL} \cdot M_{LL})$$

$$= 1.0 \cdot (1.25 \cdot M_{DC} + 1.50 \cdot M_{DW} + 1.75 \cdot M_{LL}) \qquad \boxed{M_{str} = 862} \quad \text{kip-ft}$$

Service 1 (for compression checks)

$$M_{s1} := \eta \cdot (\gamma s1_{DC} \cdot M_{DC} + \gamma s1_{DW} \cdot M_{DW} + \gamma s1_{LL} \cdot M_{LL})$$

$$= 1.0 \cdot (1.0 \cdot M_{DC} + 1.0 \cdot M_{DW} + 1.0 \cdot M_{LL}) \qquad \boxed{M_{s1} = 554} \quad \text{kip-ft}$$

Service 3 (for tension checks)

$$M_{s3} := \eta \cdot (\gamma s3_{DC} \cdot M_{DC} + \gamma s3_{DW} \cdot M_{DW} + \gamma s3_{LL} \cdot M_{LL})$$

$$= 1.0 \cdot (1.0 \cdot M_{DC} + 1.0 \cdot M_{DW} + 0.8 \cdot M_{LL}) \qquad \boxed{M_{s3} = 488} \quad \text{kip-ft}$$

Fatigue 1 (for compression checks)

$$M_{f1} := \eta \cdot \left[\frac{1}{2} \cdot (M_{DC} + M_{DW}) + \gamma f_{LL} \cdot M_{LLf}\right]$$

$$= 1.0 \cdot \left[\frac{1}{2} \cdot (M_{DC} + M_{DW}) + 1.5 \cdot M_{LLf}\right] \qquad \boxed{M_{f1} = 330} \quad \text{kip-ft}$$

E19-3.9 Allowable Stress

Allowable stresses are determined for 2 stages 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 service.

E19-3.9.1 Temporary Allowable Stresses

The temporary allowable stress (compression) **LRFD [5.9.4.1.1]**:

$f_{ciall} := 0.65 \cdot f'_{ci}$ $\boxed{f_{ciall} = 2.763}$ 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):

$f_{tiall} := -\min\left(0.0948 \cdot \lambda \sqrt{f'_{ci}}, 0.2\right)$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_{tiall} = -0.195}$ ksi

If bonded reinforcement is present in the top flange, the temporary allowable tension stress is calculated as follows:

$f_{tiall\_bond} := -0.24 \cdot \lambda \sqrt{f'_{ci}}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_{tiall\_bond} = -0.495}$ ksi

E19-3.9.2 Final Condition Allowable Stresses

Allowable Stresses,**LRFD [5.9.4.2.1]**:

There are two compressive service stress limits:

$f_{call1} := 0.45 \cdot f'_c$ PS + DL $\boxed{f_{call1} = 2.250}$ ksi

$f_{call2} := 0.60 \cdot f'_c$ LL + PS + DL $\boxed{f_{call2} = 3.000}$ ksi

There is one tension service stress limit **LRFD [5.9.4.2.2]**:

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

$f_{tall} := -0.19 \cdot \sqrt{f'_c}$ LL + PS + DL $|f_{tall}| \le 0.6$ ksi $\boxed{f_{tall} = -0.425}$ ksi

There is one compressive fatigue stress limit **LRFD [5.5.3.1]**:

$f_{call\_f} := 0.40 \cdot f'_c$ LLf + 1/2(PS + DL) $\boxed{f_{call\_f} = 2.000}$ 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 losses.

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 (before losses and while in service (after losses).

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 loac (at center span) after losses.

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

For this example, the 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 service the prestress has decreased (due to CR, SH, RE):

Estimated time dependant losses $F_{Delta} \equiv 30$ ksi

Note: The estimated time dependant losses (based on experience for low relaxation strands) will be re-calculated using the approximate method in accordance with **LRFD [5.9.5.3]** once the number of strands has been determined.

Assume an initial strand stress; $f_{tr} := 0.75 \cdot f_{pu}$ $f_{tr} = 202.5$ 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.

$$ES_{loss} := \frac{\Delta f_{pES\_est}}{f_{tr}} \cdot 100 \qquad ES_{loss} = 4.494 \ \%$$

$$f_i := f_{tr} - \Delta f_{pES\_est} \qquad f_i = 193.4 \ \text{ksi}$$

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

$$loss := F_{Delta} + \Delta f_{pES\_est} \qquad loss = 39.1 \ \text{ksi}$$

$$loss_{\%} := \frac{loss}{f_{tr}} \cdot 100 \qquad loss_{\%} = 19.309 \ \text{\% (estimated)}$$

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

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

$$ratio := 1 - \frac{loss_{\%}}{100} \qquad ratio = 0.807$$

$$T = ratio \cdot T_o$$

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

OR:

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

$$f_{bpi} := \frac{f_{bp}}{1 - \frac{loss_{\%}}{100}}$$

$\boxed{f_{bpi} = 2.313}$ ksi

desired bottom initial prestress

E19-3.10.1.2 Determine Number of Strands

$A_s = 0.153$ in²

$f_{pu} = 270$ 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

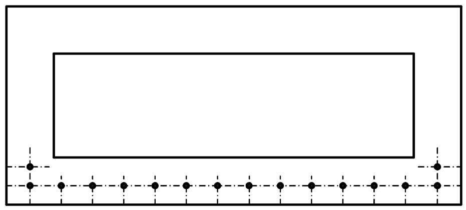
$e_s := e_b$ $\boxed{e_s = -8.5}$ inches

$$N_{req} := \frac{f_{bpi} \cdot A}{P} \cdot \frac{1}{1 + e_s \cdot \frac{y_b}{r_{sq}}}$$

$\boxed{N_{req} = 17}$ 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

$$\Delta f_{pES} := \frac{E_p}{E_{ct}} \cdot f_{cgp}$$ $\boxed{\Delta f_{pES} = 9.118}$ 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:

$$f_i := f_{tr} - \Delta f_{pES}$$ $\boxed{f_i = 193.382}$ ksi

The force in the beam after transfer is:

$$T_o := N \cdot A_s \cdot f_i$$ $\boxed{T_o = 474}$ 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.

$$f_{ttr} := \frac{T_o}{A} + \frac{T_o \cdot e_s}{S_t} + \frac{M_{gi} \cdot 12}{S_t}$$ $\boxed{f_{ttr} = 0.200}$ ksi

$$f_{btr} := \frac{T_o}{A} + \frac{T_o \cdot e_s}{S_b} + \frac{M_{gi} \cdot 12}{S_b}$$ $\boxed{f_{btr} = 1.392}$ ksi

temporary allowable stress (tension) $\boxed{f_{tiall} = -0.195}$ ksi

temporary allowable stress (compression) $\boxed{f_{ciall} = 2.763}$ ksi

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

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

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, $H := 72$ %.

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

$\gamma_{st} := \dfrac{5}{1 + f'_{ci}}$ $\quad\boxed{\gamma_{st} = 0.952}$

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

$\Delta f_{pCR} := 10.0 \cdot \dfrac{f_{tr} \cdot A_s \cdot N}{A} \cdot \gamma_h \cdot \gamma_{st}$ $\quad\boxed{\Delta f_{pCR} = 7.781}$ ksi

$\Delta f_{pSR} := 12.0 \cdot \gamma_h \cdot \gamma_{st}$ $\quad\boxed{\Delta f_{pSR} = 11.200}$ ksi

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

$\Delta f_{pLT} := \Delta f_{pCR} + \Delta f_{pSR} + \Delta f_{pRE}$ $\quad\boxed{\Delta f_{pLT} = 21.381}$ ksi

The total estimated prestress loss (Approximate Method):

$\Delta f_p := \Delta f_{pES} + \Delta f_{pLT}$ $\quad\boxed{\Delta f_p = 30.499}$ 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 $loss_{\%} = 19.3$.

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

$f_{pe} := f_{tr} - \Delta f_p$ $\quad\boxed{f_{pe} = 172.00}$ ksi

$T := N \cdot A_s \cdot f_{pe}$ $\quad\boxed{T = 421}$ kips

E19-3.10.3 Check Stresses at Critical Locations

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

Minimum moment on section = girder moment at the plant

$$M_{gz} = \frac{w_g}{2} \cdot \left(L_g \cdot z - z^2\right)$$

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:

$l_{tr} := 60 \cdot d_s$ $\boxed{l_{tr} = 30.00}$ in

$x := \dfrac{l_{tr}}{12}$ $\boxed{x = 2.50}$ feet

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

$$M_{gt} := \frac{w_{g\_ext}}{2} \cdot \left(L_g \cdot x - x^2\right) + \left(\frac{w_{diaph} \cdot x}{2} + w_{diaph\_end} \cdot x\right)$$

$\boxed{M_{gt} = 38}$ kip-ft

The girder stresses at the end of the transfer length:

$$f_{tt} := \frac{T_o}{A} + \frac{T_o \cdot e_s}{S_t} + \frac{M_{gt} \cdot 12}{S_t}$$

$\boxed{f_{tt} = -0.303}$ ksi

$\boxed{f_{tiall} = -0.195}$ ksi

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

If bonded reinforcement is provided in the top flange, the allowable stress is:

$f_{tiall\_bond} = -0.495$ ksi

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

$$f_{bt} := \frac{T_o}{A} + \frac{T_o \cdot e_s}{S_b} + \frac{M_{gt} \cdot 12}{S_b}$$

$\boxed{f_{bt} = 1.896}$ ksi

$\boxed{f_{ciall} = 2.763}$ 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):

$$f_{t1} := \frac{T}{A} + \frac{T \cdot e_s}{S_t} + \frac{\left(M_{DC} + M_{DW}\right) \cdot 12}{S_t}$$ PS + DL

$\boxed{f_{t1} = 0.459}$ ksi

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

$$f_{t2} := \frac{T}{A} + \frac{T \cdot e_s}{S_t} + \frac{M_{s1} \cdot 12}{S_t}$$ LL + PS + DL $\boxed{f_{t2} = 1.719}$ ksi

$\boxed{check = "OK"}$

Bottom of girder stress (Compression - Service 1):

$$f_{b1} := \frac{T}{A} + \frac{T \cdot e_s}{S_b} + \frac{(M_{DC} + M_{DW}) \cdot 12}{S_b}$$ PS + DL $\boxed{f_{b1} = 0.958}$ ksi

$\boxed{check = "OK"}$

Bottom of girder stress (Tension - Service 3):

$$f_b := \frac{T}{A} + \frac{T \cdot e_s}{S_b} + \frac{M_{s3} \cdot 12}{S_b}$$ $\boxed{f_b = -0.051}$ ksi

$\boxed{check = "OK"}$

Top of girder stress (Compression - Fatigue 1):

$$f_{tf1} := \frac{1}{2} \cdot \left[\frac{T}{A} + \frac{T \cdot e_s}{S_t} + \frac{(M_{DC} + M_{DW}) \cdot 12}{S_t} \right] + \frac{M_{LLf} \cdot 12}{S_t}$$ 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

$\boxed{f_{call2} = 3.000}$ 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:

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

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] $\alpha_1 := 0.85$ (for $f'_c \leq 10.0$ ksi)

$\beta_1 := \max\left[0.85 - \left(f'_c - 4\right) \cdot 0.05, 0.65\right]$ $\boxed{\beta_1 = 0.800}$

$d_p := y_t - e_s$ $\boxed{d_p = 18.75}$ in

$$c := \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 3.82}$ 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

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

$f_r := 0.24 \cdot \sqrt{f'_c}$ $\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.537}$ 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

$$M_{cr} := \gamma_3 \cdot \left[S_c \cdot \left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) \cdot \frac{1}{12} \right]$$ $\boxed{M_{cr} = 747}$ kip-ft

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

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:

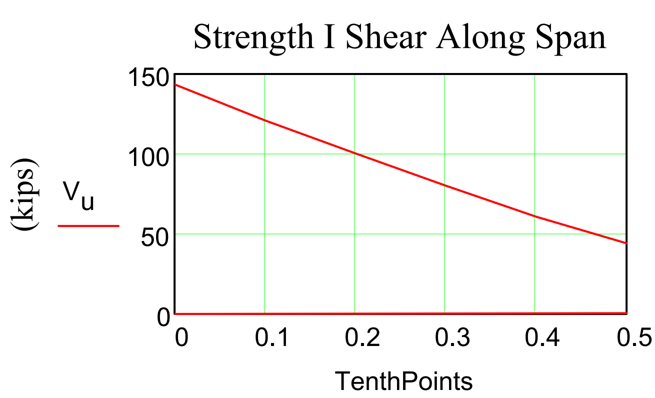
Interior Girder $\boxed{w_{DCint} = 0.792}$ klf

$\boxed{w_{DWint} = 0.082}$ 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 poin 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:

$L_{crit} := \left(w_{brg} + d_v\right) \cdot \frac{1}{12}$ $\boxed{L_{crit} = 2.10}$ 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:

$f_{pu\_crit} := f_{pe} \cdot \dfrac{L_{crit} \cdot 12}{l_{tr}}$ $\boxed{f_{pu\_crit} = 145}$ ksi

$T_{crit} := N \cdot A_s \cdot f_{pu\_crit}$ $\boxed{T_{crit} = 354}$ kips

For rectangular section behavior:

$\boxed{\alpha_1 = 0.850}$ $\boxed{\beta_1 = 0.800}$

$c_{crit} := \dfrac{A_{ps} \cdot f_{pu\_crit}}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \dfrac{f_{pu\_crit}}{d_p}}$ $\boxed{c_{crit} = 2.102}$ in

$a_{crit} := \beta_1 \cdot c_{crit}$ $\boxed{a_{crit} = 1.682}$ in

Calculation of shear depth based on refined calculations of a:

$d_{v\_crit} := -e_s + y_t - \dfrac{a_{crit}}{2}$ $\boxed{d_{v\_crit} = 17.91}$ 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:

$L_{crit} := \left(w_{brg} + d_v\right) \cdot \dfrac{1}{12}$ $\boxed{L_{crit} = 2.159}$ 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

$f_r = -0.20 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := -0.20 \cdot \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = -0.447}$ 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

$$M_{cre} := S_c \cdot \left(f_r - f_{cpe} - \frac{12 M_{dnc}}{S_{nc}} \right)$$

$\boxed{M_{cre} = 5746}$ kip-in

Calculate V_{ci} , **LRFD [5.8.3.4.3]** λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$V_{ci1} := 0.06 \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v$ $\boxed{V_{ci1} = 24.0}$ kips

$V_{ci2} := 0.02 \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v + V_d + \frac{V_i \cdot M_{cre}}{M_{max}}$ $\boxed{V_{ci2} = 495.9}$ 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

Calculate V_{cw}, **LRFD [5.8.3.4.3]** λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$V_{cw} := \left(0.06 \cdot \lambda \sqrt{f'_c} + 0.30 \cdot f_{pc}\right) \cdot b_v \cdot d_v + V_{p\_cw}$ $\boxed{V_{cw} = 56.0}$ 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]**:

$$s_{max2} := \frac{A_v \cdot f_y}{0.0316 \cdot \lambda \sqrt{f'_c} \cdot b_v}$$

λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$\boxed{s_{max2} = 33.97}$ 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

$V_{u\_crit} = 132.85$ kips

Is V_{u_crit} less than V_r? $check = \text{"OK"}$

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: $V_{u\_crit} = 133$ $0.5 \cdot \phi_v \cdot (V_c + V_p) = 25$

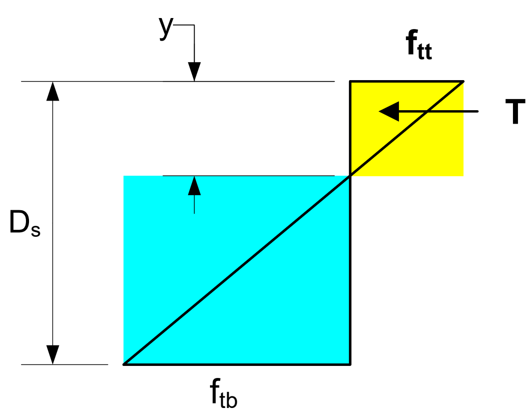
Therefore, use web reinforcing over the entire beam.

Resulting Shear Design:

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]**



$f_{tt} = -0.303$ ksi

$f_{bt} = 1.896$ ksi

$D_s = 1.75$ feet

$b = 48$ inches

$$Y := \frac{f_{tt} \cdot D_s \cdot 12}{f_{tt} - f_{bt}}$$ $Y = 2.898$ inches

$$T := \frac{|f_{tt}| \cdot b \cdot Y}{2}$$ $T = 21.101$ kips

$f_y = 60$

$$A_{reqd} := \frac{T}{0.5 \cdot f_y}$$ $A_{reqd} = 0.703$ 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:

$$T_{ps} := \frac{M_{max}}{d_v \cdot \phi_f} + \left(\left| \frac{V_{u\_crit}}{\phi_v} - V_{p\_cw} \right| - 0.5 \cdot V_s \right) \cdot \cot\theta \qquad \boxed{T_{ps} = 241} \quad \text{kips}$$

actual capacity of the straight strands:

$$\boxed{N \cdot A_s \cdot f_{pu\_crit} = 354} \quad \text{kips}$$

Is the capacity of the straight strands greater than T_{ps}? $\boxed{\text{check} = \text{"OK"}}$

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]**:

$$\boxed{l_{tr} = 30.00} \quad \text{in}$$

$$\boxed{l_d = 70.0} \quad \text{in}$$

Since l_{px} is less than the transfer length, the design stress in the prestressing strand is calculated as follows:

The assumed crack plane crosses the centroid of the straight strands at

$$Y_s := \left| y_b - e_s \right| \qquad \boxed{Y_s = 2.25} \quad \text{in}$$

$$l_{px'} := l_{px} + Y_s \cdot \cot\theta \qquad \boxed{l_{px'} = 12.05} \quad \text{in}$$

$$f_{pb} := \frac{f_{pe} \cdot l_{px'}}{60 \cdot d_s} \qquad \boxed{f_{pb} = 69.07} \quad \text{ksi}$$

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

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

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

$$\Delta_s := \frac{M_1 \cdot L_g^{\,2}}{8 \cdot E_{ct} \cdot I} \cdot 12^2$$ $\boxed{\Delta_s = 1.07}$ 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:

$$\Delta_{gi} := \frac{5 \cdot \left(w_g + w_d\right) \cdot L_g^{\,4}}{384 \cdot E_{ct} \cdot I} \cdot 12^3$$ $\boxed{\Delta_{gi} = 0.44}$ 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

$$\Delta_{nc} := \frac{5 \cdot w_{nc} \cdot L^4}{384 \cdot E_B \cdot I} \cdot 12^3 \qquad \boxed{\Delta_{nc} = 0.123} \quad \text{in}$$

The residual camber for an exterior girder:

$$RC := \Delta_i - \Delta_{nc} \qquad \boxed{RC = 0.507} \quad \text{in}$$

This page intentionally left blank.

Table of Contents

24.4.1.2 Traffic Live Load

For information about LRFD traffic live load, see 17.2.4.2.

24.4.1.3 Pedestrian Live Load

For information about LRFD pedestrian live load, see 17.2.4.4.

24.4.1.4 Temperature

Steel girder bridges are designed for a coefficient of linear expansion equal to .0000065/°F at a temperature range from -30 to 120°F. Refer to Chapter 28 – Expansion Devices for expansion joint requirements, and refer to Chapter 27 – Bearings for the effect of temperature forces on bearings.

24.4.1.5 Wind

For information about LRFD wind load, see Chapter 17 – Superstructures – General, including the WisDOT Policy item in 17.2.3.1 regarding wind speeds during a deck pour. In addition, see 24.6.16 for wind effects on girder flanges and 24.6.22 for design of bracing.

24.4.2 Minimum Depth-to-Span Ratio

Traditional minimum depths for constant depth superstructures are provided in **LRFD [Table 2.5.2.6.3-1]**. For steel simple-span superstructures, the minimum overall depth of the composite girder (concrete slab plus steel girder) is 0.040L and the minimum depth of the I-beam portion of the composite girder is 0.033L. For steel continuous-span superstructures, the minimum overall depth of the composite girder (concrete slab plus steel girder) is 0.032L and the minimum depth of the I-beam portion of the composite girder is 0.027L. For trusses, the minimum depth is 0.100L.

For a given span length, a preliminary, approximate steel girder web depth can be determined by referring to Table 24.4-1. This table is based on previous design methods and should therefore be used for preliminary purposes only. However, it remains a useful tool for approximating an estimated range of web depths for a given span length. Recommended web depths are given for parallel flanged steel girders. The girder spacings and web depths were determined from an economic study, deflection criteria and load-carrying capacity of girders for a previous design method.

From a known girder spacing, the effective span is computed as shown in Figure 17.5-1. From the effective span, the slab depth and required slab reinforcement are determined from tables in Chapter 17 – Superstructures - General, as well as the additional slab reinforcement required due to slab overhang.

10' Girder Spacing, 9" Deck		12' Girder Spacing, 10" Deck	
Span Lengths (Ft.)	Web Depth (In.)	Span Lengths (Ft.)	Web Depth (In.)
90 – 115	48	90 – 103	48
116 – 131	54	104 – 119	54
132 – 140	60	120 – 127	60
141 – 149	66	128 – 135	66
150 – 163	72	136 – 146	72
164 – 171	78	147 – 153	78
172 – 180	84	154 – 163	84
181 – 190	90	164 – 170	90
191 – 199	96	171 – 177	96
200 – 207	102	178 – 184	102
208 – 215	108	185 – 192	108

Table 24.4-1
Parallel Flange Girder Recommended Depths
For 2-Span Bridges with Equal Span Lengths)

24.4.3 Live Load Deflections

WisDOT requirements for allowable live load deflection are described in 17.2.12, and the computation of actual live load deflection is explained in 17.2.13.

Limiting the live load deflection ensures a minimum degree of stiffness in the steel girders and helps when constructing the bridge. This is especially important when using higher-strength high-performance steels which can result in shallower and more flexible girders, particularly on curved and/or skewed bridges.

24.4.4 Uplift and Pouring Diagram

Permanent hold-down devices are used to attach the superstructure to the substructure at the bearing when any combination of loading using Strength I loading combination (see **LRFD [C3.4.1]**) produces uplift. Also, permanent hold-down devices are required on alternate girders that cross over streams with less than 2' clearance for a 100-year flood where expansion bearings are used. These devices are required to prevent the girder from moving off the bearings during extreme flood conditions.

Uplift generally occurs under live loading on continuous spans when the span ratio is greater than 1 to 1.75. However, a span ratio of 1.75 should be avoided. Under extreme span ratios,

Table of Contents

30.1 Crash-Tested Bridge Railings and FHWA Policy

Notice: All contracts with a letting date after December 31, 2019 must use bridge rails and transitions meeting the 2016 Edition of MASH criteria for new permanent installation and full replacement.

WisDOT policy item:

For all Interstate structures, the 42SS parapet shall be used. For all STH and USH structures with a posted speed >= 45 mph, the 42SS parapet shall be used.

The timeline for implementation of the above policy is:

- All contracts with a letting date after December 31, 2019.
(This is an absolute, regardless of when the design was started.)

- All preliminary designs starting after October 1, 2017
(Even if the let is anticipated to be prior to December 31, 2019.)

Contact BOS should the 42" height adversely affect sight distance, a minimum 0.5% grade for drainage cannot be achieved, or for other non-typical situations.

Crash test procedures for full-scale testing of guardrails were first published in 1962 in the *Highway Research Correlation Services Circular 482*. This was a one-page document that specified vehicle mass, impact speed, and approach angle for crash testing and was aimed at creating uniformity to traffic barrier research between several national research agencies.

In 1974, the National Cooperative Highway Research Program (NCHRP) published their final report based on NCHRP Project 22-2, which was initiated to address outstanding questions that were not covered in *Circular 482*. The final report, NCHRP Report 153 – *"Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances,"* was widely accepted following publication; however, it was recognized that periodic updating would be required.

NCHRP Project 22-2(4) was initiated in 1979 to address major changes to reflect current technologies of that time and NCHRP Report 230, *"Recommended Procedures for the Safety Performance Evaluation of Highway Safety Appurtenances,"* was published in 1980. This document became the primary reference for full-scale crash testing of highway safety appurtenances in the U.S. through 1993.

In 1986, the Federal Highway Administration (FHWA) issued a policy memorandum that stated highway bridges on the National Highway System (NHS) and the Interstate Highway System (IHS) must use crash-tested railings in order to receive federal funding.

The American Association of State Highway and Transportation Officials (AASHTO) recognized that the evolution of roadside safety concepts, technology, and practices necessitated an update to NCHRP Report 230 approximately 7 years after its adoption. NCHRP Report 350, *"Recommended Procedures for the Safety Performance Evaluation of Highway Features,"* represented a major update to the previously adopted report. The updates were based on significant changes in the vehicle fleet, the emergence of many new barrier designs, increased interest in matching safety performance to levels of roadway utilization, new policies requiring the use of safety belts, and advances in computer simulation and other evaluation methods.

NCHRP Report 350 differs from NCHRP Report 230 in the following ways: it is presented in all-metric documentation, it provides a wider range of test procedures to permit safety performance evaluations for a wider range of barriers, it uses a pickup truck as the standard test vehicle in place of a passenger car, it defines other supplemental test vehicles, it includes a broader range of tests to provide a uniform basis for establishing warrants for the application of roadside safety hardware that consider the levels of use of the roadway facility, it includes guidelines for selection of the critical impact point for crash tests on redirecting-type safety hardware, it provides information related to enhanced measurement techniques related to occupant risk, and it reflects a critical review of methods and technologies for safety-performance evaluation.

In May of 1997, a memorandum from Dwight A. Horne, the FHWA Chief of the Federal-Aid and Design Division, on the subject of “Crash Testing of Bridge Railings” was published. This memorandum identified 68 crash-tested bridge rails, consolidated earlier listings, and established tentative equivalency ratings that related previous NCHRP Report 230 testing to NCHRP Report 350 test levels.

In 2009, AASHTO published the *Manual for Assessing Safety Hardware* (MASH). MASH is an update to, and supersedes, NCHRP Report 350 for the purposes of evaluating new safety hardware devices. AASHTO and FHWA jointly adopted an implementation plan for MASH that stated that all highway safety hardware accepted prior to the adoption of MASH – using criteria contained in NCHRP Report 350 – may remain in place and may continue to be manufactured and installed. In addition, highway safety hardware accepted using NCHRP Report 350 criteria is not required to be retested using MASH criteria. However, new highway safety hardware not previously evaluated must utilize MASH for testing and evaluation. MASH represents an update to crash testing requirements based primarily on changes in the vehicle fleet.

All bridge railings as detailed in the Wisconsin LRFD Bridge Standard Detail Drawings in Chapter 30 are approved for use on WisDOT projects. In order to use railings other than Bureau of Structures Standards, the railings must conform to MASH or must be crash tested rails which are available from the FHWA office. Any railing not in the Standards must be approved by the Bureau of Structures. Any railings that are not crash tested must be reviewed by FHWA when they are used on a bridge, culvert, retaining wall, etc.

WisDOT and FHWA policy states that railings that meet the criteria for Test Level 3 (TL-3) or greater shall be used on NHS roadways and all functional classes of Wisconsin structures (Interstate Highways, United States Highways, State Trunk Highways, County Trunk Highways, and Local Roadways) where the design speed exceeds 45 mph. Railings that meet Test Level 2 (TL-2) criteria may be used on non-NHS roadways where the design speed is 45 mph or less.

There may be unique situations that may require the use of a MASH crash-tested railing of a different Test Level; a railing design using an older crash test methodology; or a modified railing system based on computer modeling, component testing, and or expert opinion. These unique situations will require an exception to be granted by the Bureau of Project Development and/or the Bureau of Structures. It is recommended that coordination of these unique situations occur early in the design process.

Table of Contents

36.1 Design Method

36.1.1 Design Requirements

All new box culverts are to be designed using *AASHTO LRFD Bridge Design Specifications*, hereafter referred to as *AASHTO LRFD*.

36.1.2 Rating Requirements

The current version of *AASHTO Manual for Bridge Evaluation (LRFR)* covers rating of concrete box culverts. Refer to 45.8 for additional guidance on load rating various types of culverts.

36.1.3 Standard Permit Design Check

New structures are also to be checked for strength for the 190 kip Wisconsin Standard Permit Vehicle (Wis-SPV), with a single lane loaded, multiple presence factor equal to 1.0, and a live load factor (γ_{LL}) as shown in Table 45.3-3. See 45.12 for the configuration of the Wis-SPV. The structure should have a minimum capacity to carry a gross vehicle load of 190 kips, while also supporting the future wearing surface (where applicable – future wearing surface loads are only applied to box culverts with no fill). When applicable, this truck will be designated as a Single Trip Permit Vehicle. It will have no escorts restricting the presence of other traffic on the culvert, no lane position restrictions imposed and no restrictions on speed to reduce the dynamic load allowance, IM. The maximum Wisconsin Standard Permit Vehicle load that the structure can resist, calculated including current wearing surface loads, is shown on the plans.

36.7.3 Type E

Type E is used primarily in urban areas where a sidewalk runs over the culvert and it is necessary to have a parapet and railing along the sidewalk. For Type E the wingwalls run parallel to the roadway just like the abutment wingwalls of most bridges. It is also used where Right of Way (R/W) is a problem and the aprons would extend beyond the R/W for other types. Wingwall lengths for Type E wings are based on a minimum channel side slope of 1.5 to 1.

36.7.4 Wingwall Design

Culvert wingwalls are designed using a 1 foot surcharge height, a unit weight of backfill of 0.120 kcf and a coefficient of lateral earth pressure of 0.5, as discussed in 36.4.3. When the wingwalls are parallel to the direction of traffic and where vehicular loads are within ½ the wall height from the back face of the wall, design using a surcharge height representing vehicular load per **LRFD [Table 3.11.6.4-2]**. Load and Resistance Factor Design is used, and the load factor for lateral earth pressure of γ_{EH} = 1.69 is used, based on past design experience. The lateral earth pressure was conservatively selected to keep wingwall deflection and cracking to acceptable levels. Many wingwalls that were designed for lower horizontal pressures have experienced excessive deflections and cracking at the footing. This may expose the bar steel to the water that flows through the culvert and if the water is of a corrosive nature, corrosion of the bar steel will occur. This phenomena has led to complete failure of some wingwalls throughout the State.

For wing heights of 7 feet or less determine the area of steel required by using the maximum wall height and use the same bar size and spacing along the entire wingwall length. The minimum amount of steel used is #4 bars at 12 inch spacing. Wingwall thickness is made equal to the barrel wall thickness.

For wing heights over 7 feet the wall length is divided into two or more segments to determine the area of steel required. Use the same bar size and spacing throughout each segment, as determined by using the maximum wall height in the segment.

Wingwalls must satisfy Strength I Limit State for flexure and shear, and Service I Limit State for crack control, minimum reinforcement, and reinforcement spacing. Adequate shrinkage and temperature reinforcement shall be provided.

36.8 Box Culvert Camber

Camber of culverts is a design compensation for anticipated settlement of foundation soil beneath the culvert. Responsibility for the recommendation and calculation of camber belongs to the Regional Soils Engineer. Severe settlement problems with accompanying large camber are to be checked with the Geotechnical Section.

Both total and differential settlement need to be considered to determine the amount of box camber required to avoid adverse profile sag and undesirable separation at culvert joints per **LRFD [12.6.2.2]**. If the estimated settlement is excessive, contingency measures will need to be considered, such as preloading with embankment surcharge, undercutting and subgrade stabilization. To evaluate differential settlement, it will be necessary to calculate settlement at more than one point along the length of the box culvert.

36.8.1 Computation of Settlement

Settlement should be evaluated at the Service Limit state in accordance with **LRFD [12.6.2.2]** and **LRFD [10.6.2]**, and consider instantaneous elastic consolidation and secondary components. Elastic settlement is the instantaneous deformation of the soil mass that occurs as the soil is loaded. Consolidation settlement is the gradual compression of the soil skeleton when excess pore pressure is forced out of the voids in the soil. Secondary settlement, or creep, occurs as a result of plastic deformation of the soil skeleton under constant effective stress. Secondary settlement is typically not significant for box culvert design, except where there is an increase in effective stress within organic soil, such as peat. If secondary settlement is a concern, it should be estimated in accordance with **LRFD [10.6.2.4]**.

Total settlement, including elastic, consolidation and secondary components may be taken in accordance with **LRFD [10.6.2.4.1]** as:

$$S_t = S_e + S_c + S_s$$

Where:

S_t = Total settlement (ft)

S_e = Elastic settlement (ft)

S_c = Primary consolidation settlement (ft)

S_s = Secondary settlement (ft)

To compute settlement, the subsurface soil profile should be subdivided into layers based on stratigraphy to a depth of about 3 times the box width. The maximum layer thickness should be 10 feet.

Primary consolidation settlement for normally-consolidated soil is computed using the following equation in accordance with **LRFD [10.6.2.4.3]**:

Table of Contents

structures, pin and hanger systems, and pinned connections are inspected on a five-year cycle now.

40.2.4 Funding Eligibility and Asset Management

Nationally, MAP-21 (2012) and the FAST Act (2015) have moved structures asset management to a more data-driven approach. Funding restrictions with regards to Sufficiency Rating, Structural Deficiency, and Functional Obsolescence have been removed or significantly revised. In place of these past restrictions, MAP-21 requires the development and approval of a statewide Transportation Asset Management Plan (TAMP). A key part of the WisDOT TAMP is the Wisconsin Structures Asset Management System (WiSAMS).

WiSAMS is being developed as a planning tool, which analyzes current structure inspection data, projects future deteriorated structure condition, and applies the Bridge Preservation Policy Guide (BPPG) to recommend appropriate structure work actions at the optimal time. WiSAMS is a tool for regional and statewide programming, and is not designed as an in-depth scoping tool. WiSAMS may provide an estimate of the appropriate work action, but an in-depth evaluation of the actual structure condition and appropriate scope of work (SSR) and consideration of other non-structural project factors (e.g. cost and functionality) is still required.

In Wisconsin, the Local Bridge Program, through State Statute 84.18 and Administrative Rule Trans 213, is still tied to historic FHWA classifications of Sufficiency Rating, Structural Deficiency, and Functional Obsolescence.

40.3 Bridge Replacements

Bridge preservation and rehabilitation is preferred over bridge replacement if the final structure provides adequate serviceability. Ideal bridge preservation strategy is explained in the WisDOT Bridge Preservation Policy Guide (BPPG). This guide should be followed as closely as possible, considering estimated project costs and funding constraints.

See FDM 11-40-1.5 for policies regarding necessary bridge width* and structural capacity.

* If lane widening is planned as part of the 3R project, the usable bridge width should be compared to the planned width of the approaches after they are widened.

40.4 Rehabilitation Considerations

As a structure ages, rehabilitation is a necessary part of ensuring some level of acceptable serviceability; however, structure preservation as explained in the Bridge Preservation Policy Guide (BPPG) should be followed as closely as possible, considering estimated project costs and funding constraints.

The first consideration for any bridge rehabilitation decision is whether its geometry and load carrying capacity are adequate to safely carry present and projected traffic. Information which is helpful in determining structure adequacy includes structure inspection history, inventory data, traffic projections, maintenance history, capacity and route designations. The methods of rehabilitation are based on the type of structures, existing condition or rating information, the preliminary details of rehabilitation, and traffic control costs. These are important factors in considering rehabilitation options such as either a deck protection system or a deck replacement.

WisDOT policy item:

Rate the bridge using LFR provided it was designed using ASD or LFD. There are instances, however, where the LRFR rating of an existing bridge is beneficial (e.g. There is no M/M_u reduction to shear capacity using LRFR, which can affect longer-span steel bridges). Please contact the Bureau of Structures Development Section if using LRFR to rate bridges designed using ASD or LFD. Bridges designed using LRFD must be rated using LRFR.

The Regions are to evaluate bridge deficiencies when the bridge is placed in the program to ensure that rehabilitation will remove all structural deficiencies. Bureau of Structures (BOS) concurrence with all proposed bridge rehabilitation is required. See FDM 11-40-1.5 for policies regarding bridge rehabilitation.

Deck removal on prestressed girders is a concern as the contractors tend to use jackhammers to remove the deck. Contractors have damaged the top girder flanges using this process either by using too large a hammer or carelessness. With the wide-flange sections this concern is amplified. It is therefore suggested that the contractor saw cut the slab to be removed longitudinally close to the shear connectors. With the previously applied bond breaker, the slab should break free and then the contractor can clear the concrete around the shear connectors. Saw cutting needs to be closely monitored as contractors have sawn through steel girder flanges by not watching their blade depth.

In the rehabilitation or widening of bridge decks, it is often necessary to place concrete while traffic uses adjacent lanes. There has been considerable concern over the effects of traffic-induced vibrations on the quality of the concrete and its bond to existing concrete and its embedded reinforcing steel. Wisconsin bridge construction experience indicates that there are many cases where problems have occurred during deck pours with adjacent traffic. Consideration should be given to prohibiting traffic during the deck pour until concrete has taken its initial set.

The most effective way to reduce the amplitude of traffic-induced vibrations is to maintain a smooth structure approach and riding surface. Wisconsin has experienced some cracking in the concrete overlays and parapets during rehabilitation construction. The apparent cause of cracking is related to traffic impact due to rough structure approaches. Particular attention should be given to providing a smooth transition on temporary approaches and over expansion joints on all bridge decks during rehabilitation. Other causes of cracking are related to shrinkage, temperature, wind induced vibrations, concrete quality and improper curing.

Options for deck rehabilitation are as follows:

1. Asphalt Patch
2. Asphalt or Polymer Modified Asphaltic Overlay
3. Concrete or Modified Concrete Patch
4. Waterproof Membrane with an Asphalt Overlay (currently not used)
5. Concrete Overlay - Grade E Low Slump Concrete, or Micro-Silica Modified Concrete

Consider the following criteria for rehabilitation of highway bridges:

1. Interstate Bridges as Stand Alone Project
    a. Deck condition equal 4 or 5 and;
    b. Wear course or wear surface less than or equal to 3.
    c. No roadway work scheduled for at least 3 years.
2. Interstate Bridge with Roadway Work
    a. No previous work in last 10 years or;
    b. Deck Condition less than or equal 4.
    c. Wear course or wear surface less than or equal to 4.
3. Rehab not needed on Interstate Bridges if:
    a. Deck rehab work less than 10 years old.
    b. Deck condition greater than 4.
    c. Wear surface or wear course greater than or equal 4.
4. All Bridges

40.6 Deck Replacements

Depending on the structure age or site conditions, the condition of original deck or deck overlay, a complete deck replacement may be the most cost effective solution and extend the life of the bridge by 40 years or more. Epoxy coated rebars are required on bridge deck replacements under the same criteria as for new bridges. The new deck and parapet or railing shall be designed per the most recent edition of the WisDOT Bridge Manual, including continuity bars and overhang steel. The top flange of steel girders should be painted.

The following condition or rating criteria are the minimum requirements on STN bridges (does not include local roadways over STN routes) eligible for deck replacements:

Item	Existing Condition	Condition after Construction
Deck Condition	≤ 4	≥ 8
Inventory Rating	---	≥ HS15*
Superstructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Substructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Horizontal and Vertical Alignment Condition	> 3	---
Shoulder Width	6 ft	6 ft

Table 40.6-1
Condition Requirements for Deck Replacements

*Rating evaluation is based on the criteria found in Chapter 45-Bridge Rating. An exception to requiring a minimum Inventory Rating of HS15 is made for continuous steel girder bridges, with the requirement for such bridges being a minimum Inventory Rating of HS10. For all steel girder bridges, assessment of fatigue issues as well as paint condition (and lead paint concerns) should be included in the decision as to whether a deck replacement or a superstructure/bridge replacement is the best option.

For any bridge not meeting the conditions for deck replacement, a superstructure, or likely a complete bridge replacement is recommended. For all Interstate Highway deck replacements, the bridges are to have a minimum Inventory Rating of HS20 after the deck is replaced.

WisDOT policy item:

Contact the Bureau of Structures Development Section Ratings Unit if a deck replacement for an Interstate Highway bridge would result in an Inventory Rating less than HS20.

For slab superstructure replacements on concrete pier columns, it is reported that the columns often get displaced and/or cracked during removal of the existing slab. Plans should be detailed showing full removal of these pier columns to the existing footing and replacement with current standard details showing a concrete cap with pier columns or shaft.

See the *Facilities Development Manual* and *FDM SDD 14b7* for anchorage/offset requirements for temporary barrier used in staged construction. Where temporary bridge barriers are being used, the designer should attempt to meet the required offsets so that the barrier does not require anchorage which would necessitate drilling holes in the new deck.

In general, the substructure need not be analyzed for additional dead load provided the new deck is comparable to the existing. Exceptions include additional sidewalk or raised medians that would significantly alter the dead load of the superstructure.

For prestressed girder deck replacements, replace existing intermediate concrete diaphragms with new steel diaphragms at existing diaphragm locations (i.e. don't add intermediate lines of diaphragms). See Chapter 19 Standard Details and Steel Diaphragm Insert Sheets for additional information.

For deck replacement projects that change global continuity of the structure, the existing superstructure and substructure elements shall be evaluated using LFD criteria. One example of this condition is an existing, multi-simple span structure with expansion joints located at the pier locations. From a maintenance perspective, it is advantageous to remove the joints from the bridge deck and in order to do so; the continuity of the deck must be made continuous over the piers.

40.7 Rehabilitation Girder Sections

WisDOT BOS has retired several girder shapes from standard use on new structures. The 45", 54" and 70" girder sections shall be used primarily for bridge rehabilitation projects such as girder replacements or widening.

These sections may also be used on a limited basis on new curved structures when the overhang requirements cannot be met with the wide-flange girder sections. These sections employ draped strand patterns with undraped alternates where feasible. Undraped strand patterns, when practical, should be specified on the designs. See the Standard Details for the girder sections' draped and undraped strand patterns.

The 45", 54", and 70" girders in Chapter 40-Bridge Rehabilitation standards have been updated to include the proper non-prestressed reinforcement so that these sections are LRFD compliant. Table 40.7-1 provides span lengths versus interior girder spacings for HL-93 live loading on single-span and multiple-span structures for prestressed I-girder rehabilitation sections. Girder spacing and span lengths are based on the following criteria:

- Interior girders with low relaxation strands at $0.75f_{pu}$,
- A concrete haunch of 2-1/2",
- Slab thicknesses from Chapter 17-Superstructure - General,
- A future wearing surface of 20 psf,
- A line load of 0.300 klf is applied to the girder to account for superimposed dead loads,
- 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, and
- Required f'_c girder at initial prestress < 6,800 psi

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. 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, consideration shall be given to replacing the entire deck 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-Prestressed Concrete sections designed to LRFD or the sections from Chapter 40-Bridge Rehabilitation 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 **LRFD [3.6.5]** (600 kip loading) as a widening is considered rehabilitation. 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.

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). Also, additional costs will occur if bearings, floor drains, expansion joints, railings, or other structural elements require replacement.

Approval is required from BOS for all superstructure replacement projects. In order for a superstructure replacement to be allowed, the substructure must meet the criteria outlined below. This justifies the cost of a new superstructure by ensuring a uniform level of reliability for the entire structure.

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 designed 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.

Table of Contents

45.3.3.3 Deck

Reinforced concrete decks on redundant, multi-girder bridges are not typically load rated. A load rating would only be required in cases of significant deterioration, damage, or to investigate particularly heavy wheel or axle loads. A deck designed using an antiquated design load (H-10, H-15, etc.) may also warrant a load rating.

Other deck types (timber, filled corrugated steel) generally have lower capacity than reinforced concrete decks. This should be taken under consideration when load rating a structure with one of these deck types. Other deck types may also be more susceptible to damage or deterioration.

It is the responsibility of the load rating engineer to determine if a load rating for the deck is required.

45.3.4 Data Collection

Proper and complete data collection is essential for the accurate load rating of a bridge. It is the responsibility of the load rating engineer to gather all essential data and to assess its reliability. When assumptions are used, they should be noted and justified.

45.3.4.1 Existing Plans

Existing design plans are used to determine original design loads, bridge geometry, member section properties, and member material properties. It is important to review all existing plans; original plans as well as plans for any rehabilitation projects (deck replacements, overlays, etc.). If possible, as-built plans should be consulted as well. These plans reflect any changes made to the design plans during construction. Repair plans that document past repairs to the structure may also be available and should be reviewed, if they exist.

If no plans exist or if existing plans are illegible, field measurements may be required to determine bridge geometries and member section properties. Assumptions may have to be made on material properties. Direction on material assumptions is addressed in 45.5.2.

45.3.4.2 Shop Drawings and Fabrication Plans

Shop drawings and fabrication plans can be an extremely valuable source of information when performing a load rating. Shop drawings and fabrication plans are probably the most accurate documentation of what members and materials were actually used during construction, and may contain information not found in the design plans.

WisDOT has an inventory of shop drawings and fabrication plans, but they do not exist for every existing bridge. If the load rating engineer feels shop drawings and/or fabrication plans are required in order to accurately perform the load rating, contact the Bureau of Structures Rating Unit for assistance.

45.3.4.3 Inspection Reports

When rating an existing bridge, it is critical to review inspection reports, particularly the most recent report. Any notes regarding deterioration, particularly deterioration in primary load-carrying members, should be paid particular attention. It is the responsibility of the load rating engineer to evaluate any recorded deterioration and determine how to properly model that deterioration in a load rating analysis. Reviewing historical inspection reports can offer insight as to the rate of growth of any reported deterioration. Inspection reports can also be used to verify existing overburden.

Inspections of bridges on the State Trunk Highway Network are performed by trained personnel from the Regional maintenance sections utilizing guidelines established in the latest edition of the *WisDOT Structure Inspection Manual*. Engineers from the Bureau of Structures may assist in the inspection of bridges with unique structural problems or when it is suspected that a reduction in load capacity is warranted. To comply with the National Bridge Inspection Standards (NBIS), it is required that all bridges be routinely inspected at intervals not to exceed two years. More frequent inspections are performed for bridges which are posted for load capacity or when it is warranted based on their condition. In addition, special inspections such as underwater diving or fracture critical are performed when applicable. Inspectors enter inspection information into the Highway Structures Information System (HSIS), an on-line bridge management system developed by internally by WisDOT. For more information on HSIS, see 45.3.5. For questions on inspection-related issues, please contact the Bureau of Structures Maintenance Section.

45.3.4.4 Other Records

Other records may exist that can offer additional information or insight into bridge design, construction, or rehabilitation. In some cases, these records may override information found in design plans. It is the responsibility of the load rating engineer to gather all pertinent information and decide how to use that information. Examples of records that may exist include:

- Standard plans – generic design plans that were sometimes used for concrete t-girder structures, concrete slab structures, steel truss structures, and steel through-girder structures.
- Correspondences
- Material test reports
- Mill reports
- Non-destructive test reports
- Photographs
- Repair records
- Historic rating analysis

Once a bridge has been removed, records are removed from HSIS. However, if the bridge was removed after 2003, information may still be available by contacting the Bureau of Structures Bridge Management Unit.

45.3.5 Highway Structure Information System (HSIS)

The Highway Structure Information System (HSIS) is an on-line database used to store a wide variety of bridge information. Data stored in HSIS is used to create the National Bridge Inventory (NBI) file that is submitted annually to FHWA. Much of this data can be useful for the load rating engineer when performing a rating. HSIS is also the central source for documents such as plans and maintenance records. Other information, such as design calculations, rating calculations, fabrication drawings, and items mentioned in 45.3.4.4 may also be found in HSIS. For more information on HSIS, see the WisDOT Bureau of Structures web page or contact the Bureau of Structures Bridge Management Unit.

45.3.6 Load Rating Methodologies – Overview

There are two primary methods of load rating bridge structures that are currently utilized by WisDOT. Both methods are detailed in the AASHTO MBE. They are as follows:

- Load and Resistance Factor Rating (LRFR)
- Load Factor Rating (LFR)

Load and Resistance Factor Rating is the most current rating methodology and has been the standard for new bridges in Wisconsin since approximately 2007. LRFR employs the same basic principles as LFR for the load factors, but also utilizes multipliers on the capacity side of the rating equation, called resistance factors, to account for uncertainties in member condition, material properties, etc. This method is covered in 45.3.7, and a detailed description of this method can also be found in **MBE [6A]**.

Load Factor Rating (LFR) has been used since the early 1990s to load rate bridges in Wisconsin. The factor of safety for LFR-based rating comes from assigning multipliers, called load factors, to both dead and live loads. A detailed description of this method can be found in 45.3.8 and also in **MBE [6B]**.

Allowable Stress Rating (ASR) is a third method of load rating structures. ASR was the predominant load rating methodology prior to the implementation of LFR. It is not commonly used for modern load rating, though it is still permitted to be used for select superstructure types (See 45.3.2). The basic philosophy behind this method assigns an appropriate factor of safety to the limiting stress of the material being analyzed. The maximum stress in the member produced by actual loadings is then checked for sufficiency. A more detailed description of this method can be found in 45.3.9 below and also in **MBE [6B]**.

45.3.7 Load and Resistance Factor Rating (LRFR)

The basic rating equation for LRFR, per **MBE [Equation 6A.4.2.1-1]**, is:

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)}$$

For the Strength Limit States (primary limit state when load rating using LRFR):

$$C = \phi_C \phi_S \phi R_n$$

Where the following lower limit shall apply:

$$\phi_C \phi_S \geq 0.85$$

Where:

RF	=	Rating factor
C	=	Capacity
R_n	=	Nominal member resistance
DC	=	Dead-load effect due to structural components and attachments
DW	=	Dead-load effect due to the wearing surface and utilities
P	=	Permanent loads other than dead loads
LL	=	Live load effects
IM	=	Dynamic load allowance
γ_{DC}	=	LRFR load factor for structural components and attachments
γ_{DW}	=	LRFR load factor for wearing surfaces and utilities
γ_P	=	LRFR load factor for permanent loads other than dead loads = 1.0
γ_{LL}	=	LRFR evaluation live load factor
ϕ_c	=	Condition factor
ϕ_s	=	System factor
ϕ	=	LRFR resistance factor

The LRFR methodology is comprised of three distinct procedures:

- Design Load Rating (first level evaluation) – Used for verification during the design phase, a design load rating is performed on both new and existing structures alike. See 45.3.7.6 for more information.
- Legal Load Rating (second level evaluation) – If required, the legal load rating is used to determine whether or not the bridge in question can safely carry legal-weight traffic; whether or not a load posting is required. See 45.3.7.7 for more information.

- Permit Load Rating (third level evaluation) – The permit load rating is used to determine whether or not over-legal weight vehicles may travel across a bridge. See 45.3.7.8 for more information.

The results of each procedure serve specific uses (as noted above) and also guide the need for further evaluations to verify bridge safety or serviceability. A flow chart outlining this approach is shown in Figure 45.3-1. The procedures are structured to be performed in a sequential manner, as needed, starting with the design load rating. Load rating for AASHTO legal loads is only required when a bridge fails the design load rating (RF < 1.0) at the operating level.

Note that when designing a new structure, it is required that the rating factor be greater than one for the HL-93 vehicle at the inventory level (note also that new designs shall include a dead load allotment for a future wearing surface); therefore, a legal load rating will never be required on a newly designed structure.

Similarly, only bridges that pass the legal load rating at the operating level (RF ≥ 1.0) can be evaluated utilizing the permit load rating procedures. See 45.11 for more information on over-weight permitting.

45.3.7.1 Limit States

The concept of limit states is discussed in detail in the AASHTO LRFD design code (**LRFD [3.4.1]**). The application of limit states to the design of Wisconsin bridges is discussed in 17.2.3.

Service limit states are utilized to limit stresses, deformations, and crack widths under regular service conditions. Satisfying service limits during the design-phase is critical in order for the structure in question to realize its full intended design-life. WisDOT policy regarding load rating using service limit states is as follows:

<u>Steel Superstructures</u>

- The Service II limit state shall be satisfied (inventory rating > 1.0) during design.
- For design or legal load ratings for in-service bridges, the Service II rating shall be checked at the inventory and operating level.
- The Service II limit state should be considered for permit load rating at the discretion of the load rating engineer.

<u>Reinforced Concrete Superstructures</u>

- WisDOT does not consider the Service I limit state during design.
- For design or legal load ratings of new or in-service bridges, the Service I rating is not required.

- The Service I limit state should be considered for permit load rating at the discretion of the load rating engineer.

Prestressed Concrete Superstructures

- The Service III limit state shall be satisfied (inventory rating > 1.0) during the design phase for a new bridge.
- For design load ratings of an in-service bridge, the Service III limit state shall be checked at the inventory level. The Service III limit state should be considered for legal load rating at the discretion of the load rating engineer. The Service III limit state is not required for a permit load rating.
- For design or legal load ratings of new or in-service bridges, the Service I limit state is not required. The Service I limit state should be considered for permit load rating at the discretion of the load rating engineer.

See Table 45.3-1 for live load factors to use for each limit state. Service limit states checks that are considered optional are shaded.

Table of Contents

E45-1 Reinforced Concrete Slab Rating Example - LRFR

The 3-span continuous haunched slab structure shown in the Design Example from Chapter 18 is rated below. This same basic procedure is applicable for flat slab structures. For LRFR, the Bureau of Structures rates concrete slab structures for the Design Load (HL-93) and for Permit Vehicle Loads on an Interior Strip. The Permit Vehicle may be the Wisconsin Standard Permit Vehicle (Wis-SPV) or an actual Single-Trip Permit Vehicle. This bridge was analyzed using a slab width equal to one foot.

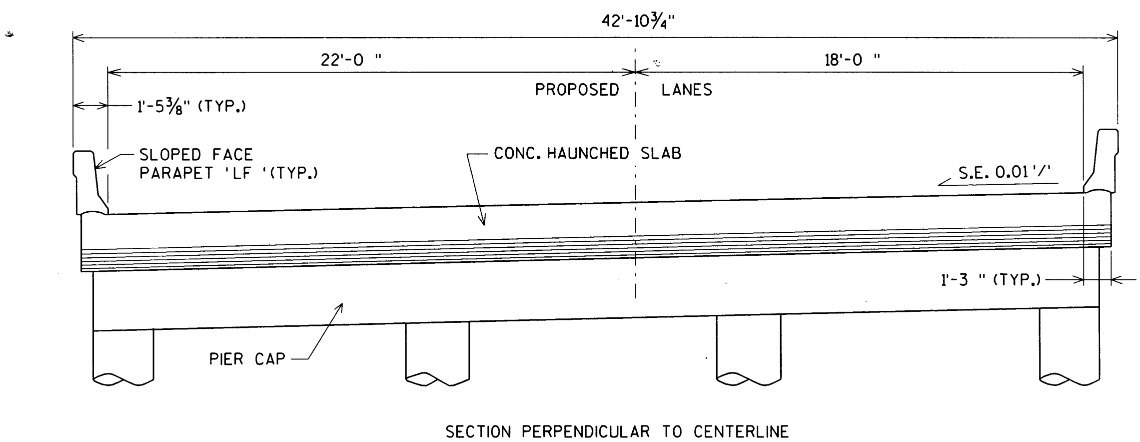


Figure E45-1.1

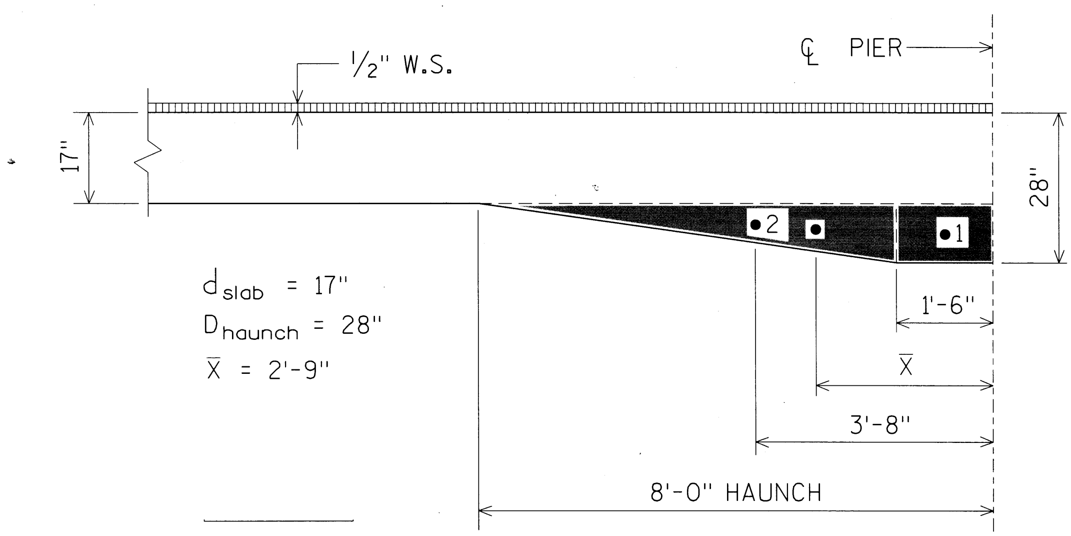


Figure E45-1.2

E45-1.1 Design Criteria

Geometry:

$L_1 := 38.0$	ft	Span 1 Length
$L_2 := 51.0$	ft	Span 2 Length
$L_3 := 38.0$	ft	Span 3 Length
$slab_{width} := 42.5$	ft	out to out width of slab
$skew := 6$	deg	skew angle (RHF)
$w_{roadway} := 40.0$	ft	clear roadway width
$cover_{top} := 2.5$	in	concrete cover on top bars (includes 1/2in wearing surface)
$cover_{bot} := 1.5$	in	concrete cover on bottom bars
$d_{slab} := 17$	in	slab depth (not including 1/2in wearing surface)
$D_{haunch} := 28$	in	haunch depth (not including 1/2in wearing surface)
$A_{st\_0.4L} := 1.71$	$\frac{in^2}{ft}$	Area of longitudinal bottom steel at 0.4L (# 9's at 7in centers)
$A_{st\_pier} := 1.88$	$\frac{in^2}{ft}$	Area of longitudinal top steel at Pier (# 8's at 5in centers)

Material Properties:

$f'_c := 4$	ksi	concrete compressive strength
$f_y := 60$	ksi	yield strength of reinforcement
$E_c := 3800$	ksi	modulus of elasticity of concrete
$E_s := 29000$	ksi	modulus of elasticity of reinforcement
$n := 8$		E_s / E_c (modular ratio)

Weights:

$w_c := 150$	pcf	concrete unit weight
$w_{LF} := 387$	plf	weight of Type LF parapet (each)

E45-1.2 Analysis of an Interior Strip - one foot width

Use Strength Limit States to rate the concrete slab bridge. **MBE [6A.4.2.2]**

The influence of ADTT and skew on force effects are ignored for slab bridges (See 18.3.2.2).

E45-1.2.1 Dead Loads (DC, DW)

The slab dead load, DC_{slab}, and the section properties of the slab, do not include the 1/2 inch wearing surface. But the 1/2 inch wearing surface load, DC_{WS}, of 6 psf must be included in the analysis of the slab. For a one foot slab width:

$DC_{ws} := 6$ 1/2 inch wearing surface load, plf

The parapet dead load is uniformly distributed over the full width of the slab when analyzing an Interior Strip. For a one foot slab width:

$$DC_{para} := 2 \cdot \frac{w_{LF}}{slab_{width}} \qquad \boxed{DC_{para} = 18} \quad \text{plf}$$

The unfactored dead load moments, M_{DC}, due to slab dead load (DC_{slab}), parapet dead load (DC_{para}), and the 1/2 inch wearing surface (DC_{ws}) are shown in Chapter 18 Example (Table E18.4).

The structure was designed for a possible future wearing surface, DW_{FWS}, of 20 psf.

$DW_{FWS} := 20$ Possible wearing surface, plf

E45-1.2.2 Live Load Distribution (Interior Strip)

Live loads are distributed over an equivalent width, E, as calculated below.
The live loads to be placed on these widths are axle loads (i.e., two lines of wheels) and the full lane load. The equivalent distribution width applies for both live load moment and shear.

Single - Lane Loading: $E = 10.0 + 5.0 \cdot (L_1 \cdot W_1)^{0.5}$ in

Multi - Lane Loading: $E = 84.0 + 1.44 \cdot (L_1 \cdot W_1)^{0.5} \le 12.0 \cdot \frac{W}{N_L}$ in

Where:

L_1 = modified span length taken equal to the lesser of the actual span or 60ft (L_1 in ft)

W_1 = modified edge to edge width of bridge taken to be equal to the lesser of the actual width or 60ft for multi-lane loading, or 30ft for single-lane loading (W_1 in ft)

W = physical edge to edge width of bridge (W in ft)

N_L = number of design lanes as specified in **LRFD [3.6.1.1.1]**

For single-lane loading:

(Span 1, 3) $E := 10.0 + 5.0 \cdot (38 \cdot 30)^{0.5}$ $\boxed{E = 178.819}$ in

(Span 2) $E := 10.0 + 5.0 \cdot (51 \cdot 30)^{0.5}$ $\boxed{E = 205.576}$ in

For multi-lane loading:

$$12.0 \cdot \frac{W}{N_L} = 12.0 \cdot \frac{42.5}{3} = 170 \quad \text{in}$$

(Span 1, 3) $E := 84.0 + 1.44 \cdot (38 \cdot 42.5)^{0.5}$ $\boxed{E = 141.869}$ in <170" O.K.

(Span 2) $E := 84.0 + 1.44 \cdot (51 \cdot 42.5)^{0.5}$ $\boxed{E = 151.041}$ in <170" O.K.

E45-1.2.3 Nominal Flexural Resistance: (M_n)

The depth of the compressive stress block, (a) is (See 18.3.3.2.1):

$$a = \frac{A_s \cdot f_s}{\alpha_1 \cdot f'_c \cdot b}$$

where:

A_s = area of developed reinforcement at section (in^2)

f_s = stress in reinforcement (ksi)

$f'_c = 4$ ksi

$b := 12$ in

$\alpha_1 := 0.85$ (for $f'_c \leq 10.0$ ksi) **LRFD [5.7.2.2]**

As shown throughout the Chapter 18 Example, when f_s is assumed to be equal to f_y, and is used to calculate (a), the value of c/d_s will be < 0.6 (for f_y = 60 ksi) per **LRFD [5.7.2.1]** Therefore the assumption that the reinforcement will yield ($f_s = f_y$) is correct. The value for (c) and (d_s) are calculated as:

$$c = \frac{a}{\beta_1}$$

$\beta_1 := 0.85$

d_s = slab depth(excl. 1/2" wearing surface) - bar clearance - 1/2 bar diameter

For rectangular sections, the nominal moment resistance, M_n, (tension reinforcement only) equals:

$$M_n = A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$$

Minimum Reinforcement Check

All sections throughout the bridge meet minimum reinforcement requirements, because this was checked in the chapter 18 Design example. Therefore, no adjustment to nominal resistance (M_n) or moment capacity is required. **MBE [6A.5.6]**

E45-1.2.4 General Load - Rating Equation (for flexure)

$$RF = \frac{C - (\gamma_{DC}) \cdot (M_{DC}) - (\gamma_{DW}) \cdot (M_{DW})}{\gamma_L \cdot (M_{LL\_IM})}$$ **MBE [6A.4.2.1]**

For the Strength Limit State:

$$C = (\phi_c)(\phi_s)(\phi) \cdot R_n$$

where:

$R_n = M_n$ (for flexure)

$(\phi_c)(\phi_s) \geq 0.85$

Factors affecting Capacity (C):

Resistance Factor (ϕ), for Strength Limit State **MBE [6.5.3]**

$\phi := 0.9$ for flexure (all reinforced concrete section in the Chapter 18 Example were found to be tension-controlled sections as defined in **LRFD [5.7.2.1]**).

Condition Factor (ϕ_c) per Chapter 45.3.2.4

$\phi_c := 1.0$

System Factor (ϕ_s) Per Chapter 45.3.2.5

$\phi_s := 1.0$ for a slab bridge

E45-1.2.5 Design Load (HL-93) Rating

Use Strength I Limit State to find the Inventory and Operating Ratings **MBE [6A.4.2.2, 6A.5.4.1]**

Equivalent Strip Width (E) and Distribution Factor (DF):

Use the smaller equivalent width (single or multi-lane), when (HL-93) live load is to be distributed, for Strength I Limit State. Multi-lane loading values will control for this bridge.

The distribution factor, DF, is computed for a slab width equal to one foot.

$$DF = \frac{1}{E}$$ (where E is in feet)

The multiple presence factor, m, has been included in the equations for distribution width, E, and therefore is not used to adjust the distribution factor, DF, **LRFD [3.6.1.1.2]**.

Spans 1 & 3:

DF = 1/(141"/12) = 0.0851 lanes / ft-slab

Span 2:

DF = 1/(151"/12) = 0.0795 lanes / ft-slab

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.
Therefore use: $DF := 0.0851$ lanes / ft-slab for all spans.

Dynamic Load Allowance (IM)

$IM := 33$ % **MBE [6A.4.4.3]**

Live Loads (LL)

The live load combinations used for Strength I Limit State are shown in the Chapter 18 Example in Table E18.2 and E18.3. The unfactored moments due to Design Lane, Design Tandem, Design Truck and 90%{Double Design Truck + Design Lanes] are shown in Chapter 18 Example (Table E18.4).

Rating for Flexure

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

Load Factors

$\gamma_{DC} := 1.25$ Chapter 45 Table 45.3-1

$\gamma_{DW} := 1.50$ WisDOT policy is to always use 1.50; Chapter 45 Table 45.3-1

$\gamma_{Li} := 1.75$ (Inventory Rating) Chapter 45 Table 45.3-1

$\gamma_{Lo} := 1.35$ (Operating Rating) Chapter 45 Table 45.3-1

The Design Load Rating was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location, for this example, is in span 1 at the 0.4 pt.

Span 1 (0.4 pt.)

Inventory:

$$RF_i = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_{Li}\cdot(M_{LL\_IM})}$$

$A_{st\_0.4L} = 1.71\ \frac{in^2}{ft}$ and $\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi) **LRFD [5.7.2.2]**

$d_s := 17.0 - cover_{bot} - \frac{1.128}{2}$ $\boxed{d_s = 14.94}$ in

$a := \frac{A_{st\_0.4L}\cdot f_y}{\alpha_1\cdot f'_c\cdot b}$ $\boxed{a = 2.51}$ in

$M_n := A_{st\_0.4L}\cdot f_y\cdot\left(d_s - \frac{a}{2}\right)$ $\boxed{M_n = 1403.4}$ kip – in

$\boxed{M_n = 117.0}$ kip – ft

$M_{DC} := 18.1$ kip – ft (from Chapter 18 Example, Table E18.4)

$M_{DW} := 0.0$ kip – ft (additional wearing surface not for HL-93 rating runs)

The positive live load moment shall be the largest caused by the following (from Chapter 18 Example, Table E18.4):

Design Tandem (+IM) + Design Lane: (37.5 kip-ft + 7.9 kip-ft) = 45.4 kip-ft
Design Truck (+IM) + Design Lane: (35.4 kip-ft + 7.9 kip-ft) = 43.3 kip-ft

Therefore:

$M_{LL\_IM} := 45.4$ kip – ft

Inventory:

$$RF_i := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_{Li}\cdot(M_{LL\_IM})}$$

$\boxed{RF_i = 1.04}$

Operating:

$$RF_o := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_{Lo}\cdot(M_{LL\_IM})}$$

$\boxed{RF_o = 1.35}$

Rating for Shear:

Slab bridge designed for dead load and (HL-93) live load moments in conformance with **LRFD [4.6.2.3]** may be considered satisfactory in shear **LRFD [5.14.4.1]**. This bridge was designed using this procedure, therefore a shear rating is not required.

The Rating Factors, RF, for Inventory and Operating Rating are shown on the plans and also on the load rating summary sheet.

E45-1.2.6 Permit Vehicle Load Ratings

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed (per 45.6).

The bridge shall be analyzed for this vehicle considering both single-lane and multi-lane distribution. Also, the vehicle will be analyzed assuming it is mixing with other traffic on the bridge and that full dynamic load allowance is utilized. Future wearing surface will not be considered.

Since this example is rating a newly designed bridge, an additional check is required. The designer shall ensure that the results of a single-lane analysis utilizing the future wearing surface are greater than 190 kips MVW.

Use Strength II Limit State to find the Permit Vehicle Load Rating **MBE[6A.4.2.2, 6A.5.4.2.1]**.

E45-1.2.6.1 Wis-SPV Permit Rating with Single Lane Distribution w/ FWS

Equivalent Strip Width (E) and Distribution Factor (DF)

The equivalent width from single-lane loading is used, when Permit Vehicle live load is to be distributed, for Strength II Limit State **MBE [6A.4.5.4.2]**.

Calculate the distribution factor, DF, and divide it by (1.20) to remove the effects of the multiple presence factor (m), which are present in the equation for equivalent width (E) **MBE [6A.3.2, C6A.4.5.4.2b]**.

The distribution factor, DF, is computed for a slab width equal to one foot.

$$DF = \frac{1}{E \cdot (1.20)} \qquad \text{(where E is in feet)}$$

Spans 1 &3:

DF = 1/(178"/12)(1.20) = 0.0562 lanes / ft-slab

Span 2:

DF = 1/(205"/12)(1.20) = 0.0488 lanes / ft-slab

Look at the distribution factor calculated for each span and select the largest value.
This single value is to be applied along the entire length of the bridge.

Therefore use: $DF := 0.0562$ lanes / ft-slab for all spans.

Dynamic Load Allowance (IM)

$IM = 33 \quad \%$ **MBE [6A.4.5.5]**

Rating for Flexure

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

Load Factors

$\gamma_{DC} := 1.25$ Chapter 45 Table 45.3-1

$\gamma_{DW} := 1.50$ WisDOT policy is to always use 1.50; Chapter 45 Table 45.3-1

$\gamma_L := 1.20$ WisDOT Policy is to designate the (Wis_SPV) as a "Single-Trip" vehicle with no escorts. Current policy is to select the value for γ_L from Chapter 45 Table 45.3-3

The Maximum Permit Vehicle Load was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location is the C/L of Pier.

At C/L of Pier

Permit Vehicle:

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

$A_{st\_pier} := 1.88 \dfrac{in^2}{ft}$ and $\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi) **LRFD [5.7.2.2]**

$d_s := 28.0 - (cover_{top} - 0.5) - \dfrac{1.00}{2}$ $\boxed{d_s = 25.5}$ in

$a := \dfrac{A_{st\_pier}\cdot f_y}{\alpha_1\cdot f'_c\cdot b}$ $\boxed{a = 2.76}$ in

$M_n := A_{st\_pier}\cdot f_y\cdot\left(d_s - \dfrac{a}{2}\right)$ $\boxed{M_n = 2720.5}$ kip – in

$\boxed{M_n = 226.7}$ kip – ft

$M_{DC} := 59.2\ kip - ft$ (from Chapter 18 Example, Table E18.4)

$M_{DW} := 1.5\ kip - ft$

The live load moment at the C/L of Pier due to the Wisconsin Standard Permit Vehicle (Wis-SPV) having a gross vehicle load of 190 kips and utilizing single lane distribution is:

$M_{LL\_IM} := 65.2 \quad \text{kip} - \text{ft}$

Permit:

$$RF_{permit} := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

$$\boxed{RF_{permit} = 1.63}$$

The maximum Wisconsin Standard Permit Vehicle (Wis_SPV) load is:

$RF_{permit}\cdot(190) = 310 \quad \text{kips}$ which is > 190k, Check OK

This same procedure used for the (Wis-SPV) can also be used when evaluating the bridge for an actual "Single-Trip Permit" vehicle.

Rating for Shear:

WisDOT does not rate Permit Vehicles on slab bridges based on shear.

E45-1.2.6.2 Wis-SPV Permit Rating with Single Lane Distribution w/o FWS

Equivalent Strip Width (E) and Distribution Factor (DF)

The equivalent width from single-lane loading is used, when Permit Vehicle live load is to be distributed, for Strength II Limit State **MBE [6A.4.5.4.2]**.

Calculate the distribution factor, DF, and divide it by (1.20) to remove the effects of the multiple presence factor (m), which are present in the equation for equivalent width (E) **MBE [6A.3.2, C6A.4.5.4.2b]**.

The distribution factor, DF, is computed for a slab width equal to one foot.

$$DF = \frac{1}{E\cdot(1.20)}$$ (where E is in feet)

Spans 1 &3:

DF = 1/(178"/12)(1.20) = 0.0562 lanes / ft-slab

Span 2:

DF = 1/(205"/12)(1.20) = 0.0488 lanes / ft-slab

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Therefore use: $DF := 0.0562$ lanes / ft-slab for all spans.

Dynamic Load Allowance (IM)

$IM = 33$ % **MBE [6A.4.5.5]**

Rating for Flexure

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

Load Factors

$\gamma_{DC} := 1.25$ Chapter 45 Table 45.3-1

$\gamma_L := 1.20$ WisDOT Policy is to designate the (Wis_SPV) as a "Single-Trip" vehicle with no escorts. Current policy is to select the value for γ_L from Chapter 45 Table 45.3-3

The Maximum Permit Vehicle Load was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location is the C/L of Pier.

At C/L of Pier

Permit Vehicle:

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

$A_{st\_pier} := 1.88 \frac{in^2}{ft}$ and $\alpha_1 := 0.85$ (for $f'_C \le 10.0$ ksi) **LRFD [5.7.2.2]**

$d_s := 28.0 - (cover_{top} - 0.5) - \frac{1.00}{2}$ $\boxed{d_s = 25.5}$ in

$a := \frac{A_{st\_pier}\cdot f_y}{\alpha_1\cdot f'_c\cdot b}$ $\boxed{a = 2.76}$ in

$M_n := A_{st\_pier}\cdot f_y\cdot\left(d_s - \frac{a}{2}\right)$ $\boxed{M_n = 2720.5}$ kip – in

$\boxed{M_n = 226.7}$ kip – ft

$M_{DC} := 59.2\,kip - ft$ (from Chapter 18 Example, Table E18.4)

The live load moment at the C/L of Pier due to the Wisconsin Standard Permit Vehicle (Wis-SPV) having a gross vehicle load of 190 kips and utilizing single lane distribution is:

$M_{LL\_IM} := 65.2$ kip – ft

Permit:

$$RF_{permit} := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC})}{\gamma_L\cdot(M_{LL\_IM})}$$

$\boxed{RF_{permit} = 1.66}$

The maximum Wisconsin Standard Permit Vehicle (Wis_SPV) load is:

$RF_{permit}\cdot(190) = 316$ kips

This same procedure used for the (Wis-SPV) can also be used when evaluating the bridge for an actual "Single-Trip Permit" vehicle.

E45-1.2.6.3 Wis-SPV Permit Rating with Multi Lane Distribution w/o FWS

Rating for Flexure

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC})}{\gamma_L\cdot(M_{LL\_IM})}$$

The capacity of the bridge to carry the Permit Vehicle Load was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location is at the C/L of Pier.

Load Factors

$\gamma_{DC} := 1.25$ Chapter 45 Table 45.3-1

$\gamma_{DW} := 1.50$ WisDOT policy is to always use 1.50; Chapter 45 Table 45.3-1

$\gamma_L := 1.30$ WisDOT Policy when analyzing the Wis-SPV as an "Annual Permit" vehicle with no escorts

At C/L of Pier

Permit Vehicle:

$$RF_{permit} = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC})}{\gamma_L\cdot(M_{LL\_IM})}$$

$M_n = 226.7$ kip – ft (as shown previously)

$M_{DC} = 59.2$ kip – ft (as shown previously)

The live load moment at the C/L of Pier due to the Wisconsin Permit Vehicle (Wis_SPV) having a gross vehicle load of 190 kips and a DF of 0.0851 lanes/ft-slab:

$M_{LL\_IM} := 98.7$ kip – ft

$$RF_{permit} := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC})}{\gamma_L\cdot(M_{LL\_IM})}$$

$\boxed{RF_{permit} = 1.01}$

The Wisconsin Standard Permit Vehicle (Wis_SPV) load that can be carried by the bridge is:

$RF_{permit}\cdot(190) = 193$ kips

E45-1.3 Summary of Rating

Slab - Interior Strip								
Limit State		Design Load Rating		Legal Load Rating	Permit Load Rating (kips)		
		Inventory	Operating		Single DF w/ FWS	Single DF w/o FWS	Multi DF w/o FWS
Strength I	Flexure	1.04	1.34	N/A	310	316	193
Service I		N/A	N/A	N/A	Optional	Optional	Optional

Table of Contents

E45-4 Steel Girder Rating Example - LRFR

This example shows rating calculations conforming to the *AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* as supplemented by the *WisDOT Bridge Manual (July 2008)*. This example will rate the design example E24-1 contained in the WisDOT Bridge Manual. **(Note: Example has not been updated for example E24-1 January 2016 updates)**

E45-4.1 Preliminary Data

An interior plate girder will be rated for this example. The girder was designed to be composite throughout. There is no overburden on the structure. In addition, inspection reports reveal no loss of section to any of the main load carrying members.

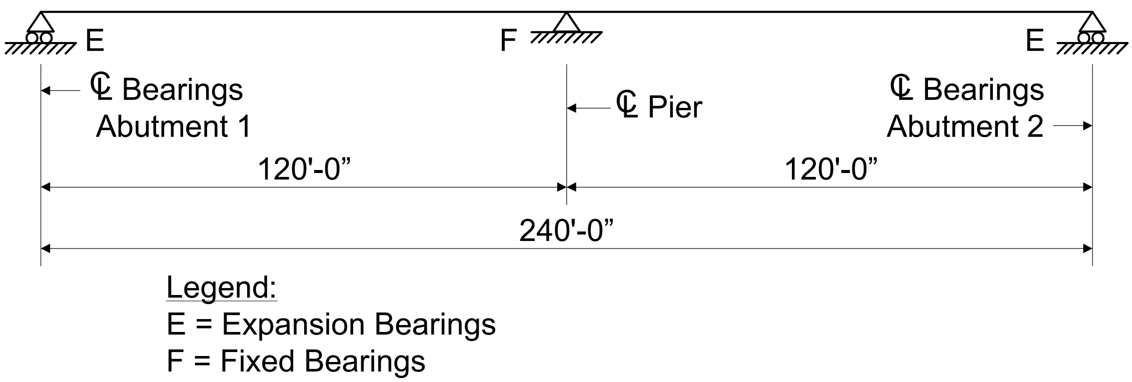


Figure E45-4.1-1
Span Configuration

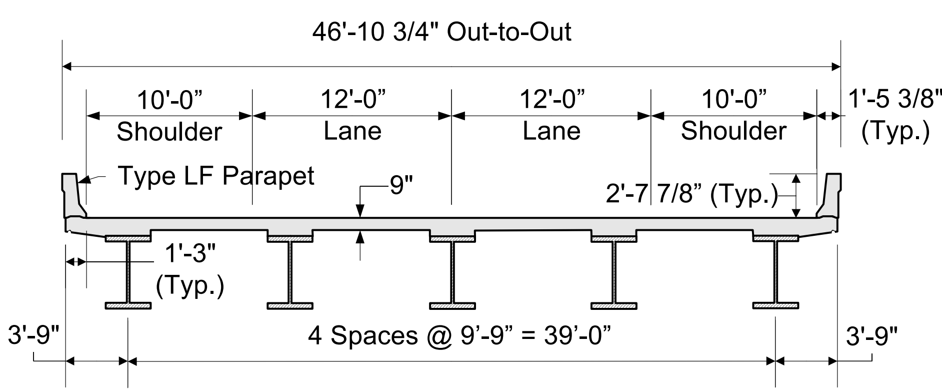


Figure E45-4.1-2
Superstructure Cross Section

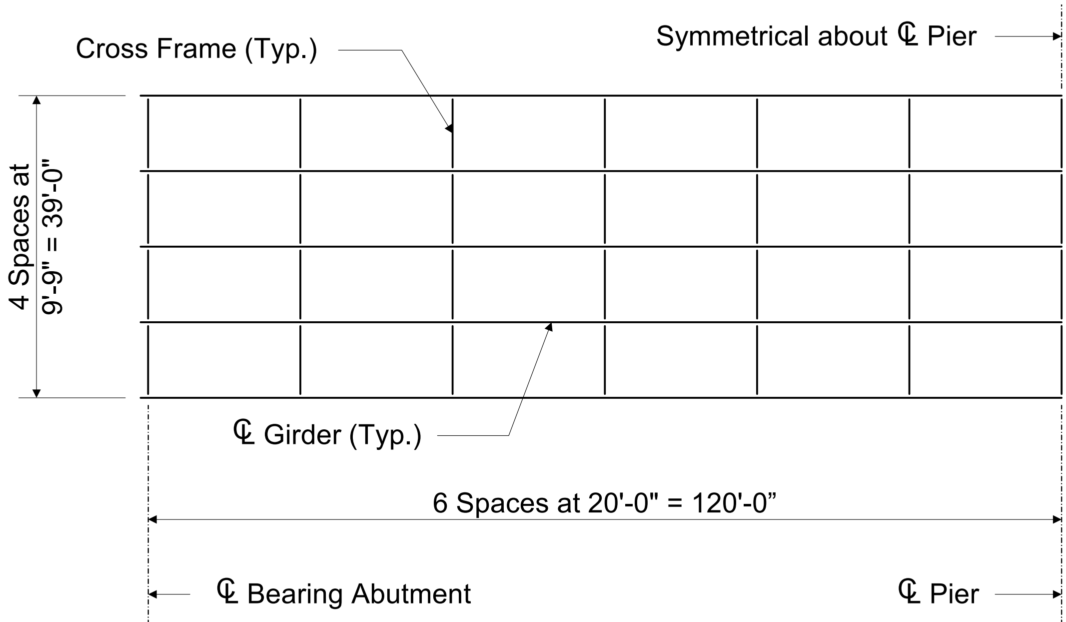


Figure E45-4.1-3
Framing Plan

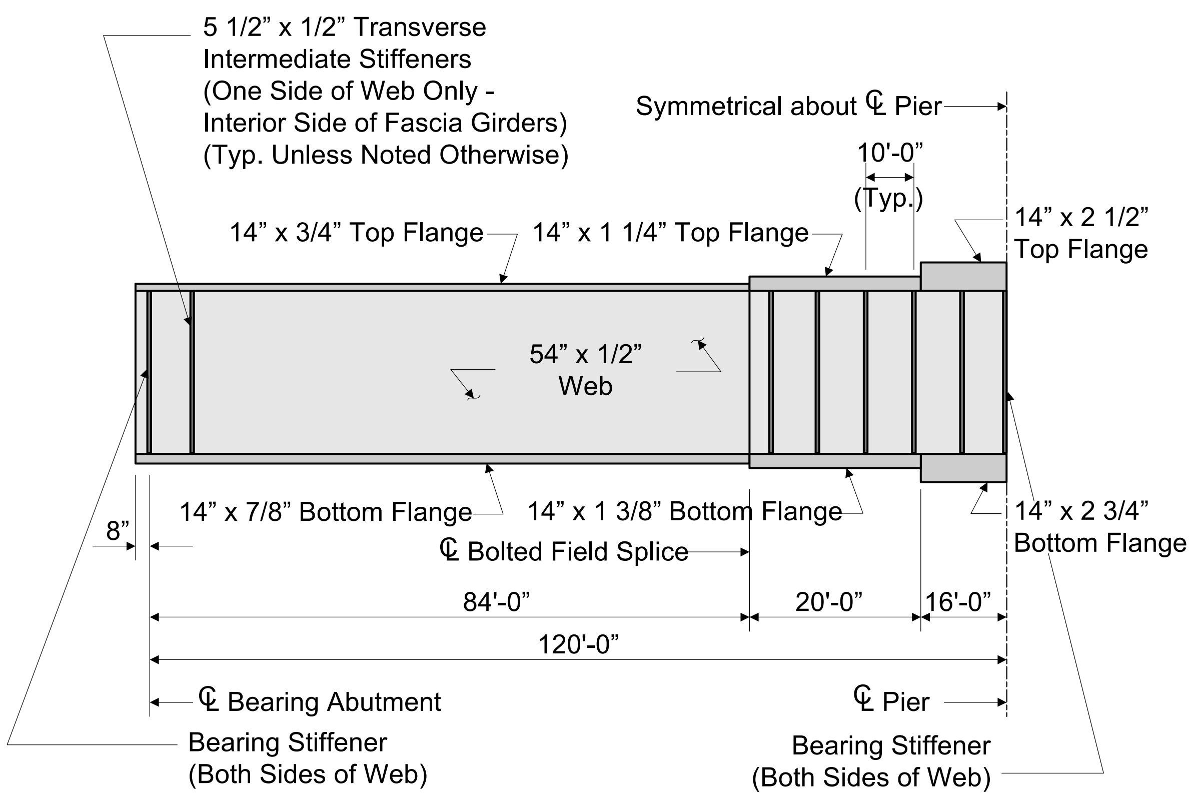


Figure E45-4.1-4
Interior Plate Girder Elevation

$N_{spans} := 2$	Number of spans	
$L := 120$	ft	span length
$Skew := 0$	deg	skew angle
$N_b := 5$	number of girders	
$S := 9.75$	ft	girder spacing
$S_{overhang} := 3.75$	ft	deck overhang
$L_b := 240$	in	cross-frame spacing
$F_{yw} := 50$	ksi	web yield strength
$F_{yf} := 50$	ksi	flange yield strength
$f'_c := 4.0$	ksi	concrete 28-day compressive strength
$f_y := 60$	ksi	reinforcement strength
$E_s := 29000$	ksi	modulus of elasticity
$t_{deck} := 9.0$	in	total deck thickness
$t_s := 8.5$	in	effective deck thickness when 1/2" future wearing surface is removed from total deck thickness
$w_s := 0.490$	kcf	steel density **LRFD[Table 3.5.1-1]**
$w_c := 0.150$	kcf	concrete density **LRFD[Table 3.5.1-1 & C3.5.1]**
$w_{misc} := 0.030$	kip/ft	additional miscellaneous dead load (per girder) per 17.2.4.1
$w_{par} := 0.387$	kip/ft	parapet weight (each)
$w_{fws} := 0.00$	kcf	future wearing surface is not used in rating analysis
$w_{deck} := 46.5$	ft	deck width
$w_{roadway} := 44.0$	ft	roadway width
$d_{haunch} := 3.75$	in	haunch depth (from top of web for design) (for construction, the haunch is measured from the top of the top flange)

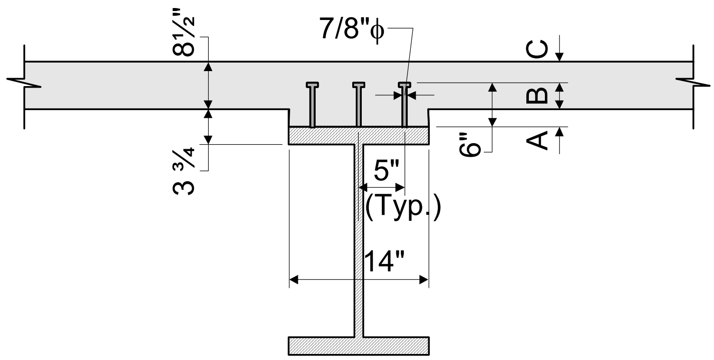


Figure E45-4.1-5
Composite Cross Section at Location of Maximum Positive Moment (0.4L)
(Note: 1/2" Intergral Wearing Surface has been removed for structural calcs.)

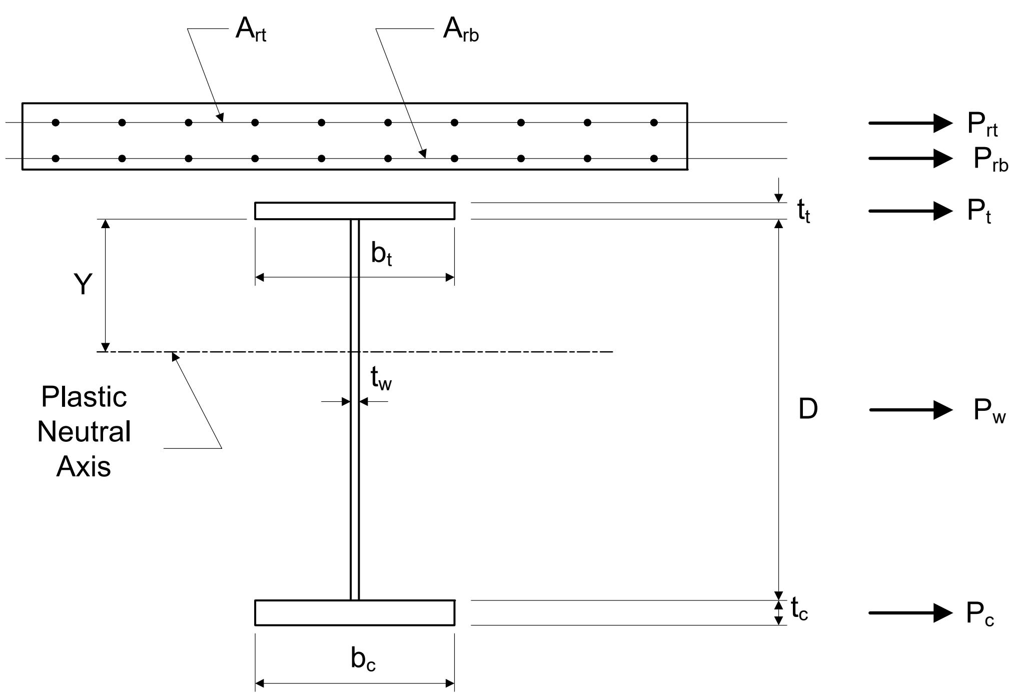


Figure E45-4.1-6
Composite Cross Section at Location of Maximum Negative Moment over Pier

$D := 54$ in

$t_w := 0.5$ in

E45-4.2 Compute Section Properties

Since the superstructure is composite, several sets of section properties must be computed **LRFD [6.10.1.1]**. The initial dead loads (or the noncomposite dead loads) are applied to the girder-only section. For permanent loads assumed to be applied to the long-term composite section, the long-term modular ratio of 3n is used to transform the concrete deck area **LRFD [6.10.1.1.1b]**. For transient loads assumed applied to the short-term composite section, the short-term modular ratio of n is used to transform the concrete deck area.

The modular ratio, n, is computed as follows:

$$n = \frac{E_s}{E_c}$$

Where:

E_s = Modulus of elasticity of steel (ksi)

E_c = Modulus of elasticity of concrete (ksi)

$E_s = 29000$ ksi **LRFD [6.4.1]**

$E_c = 33000 \cdot K_1 \cdot \left(w_c^{1.5}\right) \cdot \sqrt{f'_c}$ **LRFD [C5.4.2.4]**

Where:

K_1 = Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test, and as approved by the authority of jurisdiction

w_c = Unit weight of concrete (kcf)

f'_c = Specified compressive strength of concrete (ksi)

$w_c = 0.15$ kcf **LRFD [Table 3.5.1-1 & C3.5.1]**

$f'_c = 4.00$ ksi **LRFD [Table 5.4.2.1-1 & 5.4.2.1]**

$K_1 := 1.0$ **LRFD [5.4.2.4]**

$E_c := 33000 \cdot K_1 \cdot \left(w_c^{1.5}\right) \cdot \sqrt{f'_c}$ $\boxed{E_c = 3834}$ ksi

$n := \frac{E_s}{E_c}$ $\boxed{n = 7.6}$ **LRFD [6.10.1.1.1b]**

Therefore, use: $n := 8$

The effective flange width is computed as follows .

For interior beams, the effective flange width is calculated as per **LRFD [4.6.2.6]**:

1. 12.0 times the average thickness of the slab, plus the greater of web thickness or one-half the width of the top flange of the girder:

$$b_{eff2} := \frac{12 \cdot t_s + \frac{14}{2}}{12}$$

This is no longer a valid criteria, however it has been left in place to avoid changing the entire example at this time.

$\boxed{b_{eff2} = 9.08}$ ft

2. The average spacing of adjacent beams:

$$b_{eff3} := S$$

$\boxed{b_{eff3} = 9.75}$ ft

Therefore, the effective flange width is:

$$b_{effflange} := \min\left(b_{eff2}, b_{eff3}\right)$$

$\boxed{b_{effflange} = 9.08}$ ft

or

$\boxed{b_{effflange} \cdot 12 = 109.00}$ in

For this design example, the slab haunch is 3.75 inches throughout the length of the bridge. That is, the bottom of the slab is located 3.75 inches above the top of the web The area of the haunch is conservatively not considered in the section properties for this example.

Based on the plate sizes shown in Figure E45-4.1-4, the noncomposite and composite section properties for the positive moment region are computed as shown in the following table. The distance to the centroid is measured from the bottom of the girder.

Positive Moment Region Section Properties						
Section	**Area, A (Inches^2)**	**Centroid, d (Inches)**	**A*d (Inches^3)**	**I_o (Inches^4)**	**A*y^2 (Inches^4)**	**I_{total} (Inches^4)**
Girder only:						
Top flange	10.50	55.25	580.1	0.5	8441.1	8441.6
Web	27.00	27.88	752.6	6561.0	25.8	6586.8
Bottom flange	12.25	0.44	5.4	0.8	8576.1	8576.9
Total	49.75	26.90	1338.1	6562.3	17043.0	23605.3
Composite (3n):						
Girder	49.75	26.90	1338.1	23605.3	12293.9	35899.2
Slab	38.60	62.88	2427.2	232.4	15843.4	16075.8
Total	88.35	42.62	3765.3	23837.7	28137.3	51975.0
Composite (n):						
Girder	49.75	26.90	1338.1	23605.3	31511.0	55116.2
Slab	115.81	62.88	7281.7	697.3	13536.3	14233.6
Total	165.56	52.06	8619.8	24302.5	45047.3	69349.8
Section	**y_{botgdr} (Inches)**	**y_{topgdr} (Inches)**	**$y_{topslab}$ (Inches)**	**S_{botgdr} (Inches^3)**	**S_{topgdr} (Inches^3)**	**$S_{topslab}$ (Inches^3)**
Girder only	26.90	28.73	---	877.6	821.7	---
Composite (3n)	42.62	13.01	24.51	1219.6	3995.5	2120.7
Composite (n)	52.06	3.56	15.06	1332.0	19474.0	4604.5

Table E45-4.2-1
Positive Moment Region Section Properties

Similarly, the noncomposite and composite section properties for the negative moment region are computed as shown in the following table. The distance to the centroid is measured from the bottom of the girder **LRFD [6.6.1.2.1, 6.10.5.1, 6.10.4.2.1]**.

For the strength limit state, since the deck concrete is in tension in the negative moment region, the deck reinforcing steel contributes to the composite section properties and the deck concrete does not. However, per 45.6.3, only the top longitudinal mat of steel is used for rating purposes. Per the design example, the amount of longitudinal steel within the effective slab area is 6.39 in^2. This number will be used for the calculations below.

Negative Moment Region Section Properties						
Section	Area, A (Inches^2)	Centroid, d (Inches)	A*d (Inches^3)	I_o (Inches^4)	A*y^2 (Inches^4)	I_{total} (Inches^4)
Girder only:						
Top flange	35.00	58.00	2030.0	18.2	30009.7	30027.9
Web	27.00	29.75	803.3	6561.0	28.7	6589.7
Bottom flange	38.50	1.38	52.9	24.3	28784.7	28809.0
Total	100.50	28.72	2886.2	6603.5	58823.1	65426.6
Composite (deck concrete using 3n):						
Girder	100.50	28.72	2886.2	65426.6	10049.0	75475.6
Slab	38.60	64.75	2499.6	232.4	26161.1	26393.5
Total	139.10	38.72	5385.8	65659.0	36210.1	101869.2
Composite (deck concrete using n):						
Girder	100.50	28.72	2886.2	65426.6	37401.0	102827.7
Slab	115.81	64.75	7498.9	697.3	32455.9	33153.2
Total	216.31	48.01	10385.0	66123.9	69857.0	135980.9
Composite (deck reinforcement only):						
Girder	100.50	28.72	2886.2	65426.6	466.3	65892.9
Deck reinf.	6.39	64.75	413.8	0.0	7333.8	7333.8
Total	106.89	30.87	3299.9	65426.6	7800.1	73226.7
Section	y_{botgdr} (Inches)	y_{topgdr} (Inches)	y_{deck} (Inches)	S_{botgdr} (Inches^3)	S_{topgdr} (Inches^3)	S_{deck} (Inches^3)
Girder only	28.72	30.53	---	2278.2	2142.9	---
Composite (3n)	38.72	20.53	30.282	2631.1	4961.4	3364.0
Composite (n)	48.01	11.24	20.991	2832.4	12097.4	6478.2
Composite (rebar)	30.87	28.38	33.88	2371.9	2580.4	2161.5

Table E45-4.2-2
Negative Moment Region Section Properties

E45-4.3 Dead Load Analysis - Interior Girder

Dead Load Components		
Resisted by	Type of Load Factor	
	DC	DW
Noncomposite section	• Steel girder • Concrete deck • Concrete haunch • Stay-in-place deck forms • Misc. (including cross-frames, stiffeners, etc.)	
Composite section	• Concrete parapets	• Future wearing surface & utilities

Table E45-4.3-1
Dead Load Components

COMPONENTS AND ATTACHMENTS: DC1 (NON-COMPOSITE)

GIRDER:

For the steel girder, the dead load per unit length varies due to the change in plate sizes. The moments and shears due to the weight of the steel girder can be computed using readily available analysis software. Since the actual plate sizes are entered as input, the moments and shears are computed based on the actual, varying plate sizes.

DECK:

For the concrete deck, the dead load per unit length for an interior girder is computed as follows:

$w_c = 0.150$ kcf

$S = 9.75$ ft

$t_{deck} = 9.00$ in

$$DL_{deck} := w_c \cdot S \cdot \frac{t_{deck}}{12} \qquad \boxed{DL_{deck} = 1.097} \quad \text{kip/ft}$$

HAUNCH:

For the concrete haunch, the dead load per unit length varies due to the change in top flange plate sizes. The moments and shears due to the weight of the concrete haunch can be computed using readily available analysis software. Since the top flange plate sizes are entered as input, the moments and shears due to the concrete haunch are computed based on the actual, varying haunch thickness.

MISC:

For the miscellaneous dead load (including cross-frames, stiffeners, and other miscellaneous structural steel), the dead load per unit length is assumed to be as follows (17.2.4.1):

$DL_{misc} := 0.030$ kip/ft

COMPONENTS AND ATTACHMENTS: DC2 (COMPOSITE)

PARAPET:

For the concrete parapets, the dead load per unit length is computed as follows, assuming that the superimposed dead load of the two parapets is distributed uniformly among all of the girders **LRFD [4.6.2.2.1]**:

$w_{par} = 0.39$ kip/ft

$N_b = 5$

$DL_{par} := \frac{w_{par} \cdot 2}{N_b}$ $\boxed{DL_{par} = 0.155}$ kip/ft

WEARING SURFACE: DW (COMPOSITE)

FUTURE WEARING SURFACE:

For this example, future wearing surface is only applied for permit vehicle rating checks.

Since the plate girder and its section properties are not uniform over the entire length of the bridge, an analysis software was used to compute the dead load moments and shears.

The following two tables present the unfactored dead load moments and shears, as computed by an analysis computer program. Since the bridge is symmetrical, the moments and shears in Span 2 are symmetrical to those in Span 1.

Dead Load Moments (Kip-feet)											
Dead Load Component	**Location in Span 1**										
	0.0L	**0.1L**	**0.2L**	**0.3L**	**0.4L**	**0.5L**	**0.6L**	**0.7L**	**0.8L**	**0.9L**	**1.0L**
Steel girder	0.0	71.8	119.3	142.5	141.3	115.8	66.0	-8.2	-110.2	-244.4	-423.9
Concrete deck & haunches	0.0	480.5	796.7	948.6	936.1	759.4	418.4	-86.9	-756.0	-1588.1	-2581.3
Miscellaneous Steel Weight	0.0	12.6	21.0	25.0	24.6	20.0	11.0	-2.3	-19.9	-41.8	-68.0
Concrete parapets	0.0	67.7	113.1	136.1	136.9	115.3	71.4	5.1	-83.4	-194.3	-327.5
Future wearing surface	0.0	76.9	128.4	154.6	155.4	130.9	81.0	5.8	-94.7	-220.6	-371.9

Table 45E-4.3-2
Dead Load Moments

Dead Load Shears (Kips)											
Dead Load Component	Location in Span 1										
	0.0L	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	1.0L
Steel girder	7.0	5.0	2.9	0.9	-1.1	-3.1	-5.2	-7.2	-9.8	-12.9	-17.0
Concrete deck & haunches	46.9	33.2	19.5	5.8	-7.9	-21.6	-35.3	-49.0	-62.6	-76.1	-89.5
Miscellaneous Steel Weight	1.2	0.9	0.5	0.2	-0.2	-0.6	-0.9	-1.3	-1.7	-2.0	-2.4
Concrete parapets	6.6	4.7	2.9	1.0	-0.9	-2.7	-4.6	-6.5	-8.3	-10.2	-12.0
Future wearing surface	7.5	5.4	3.2	1.1	-1.0	-3.1	-5.2	-7.3	-9.4	-11.6	-13.7

Table 45E-4.3-3
Dead Load Shears

E45-4.4 Compute Live Load Distribution Factors for Interior Girder

The live load distribution factors for an interior girder are computed as follows **LRFD [4.6.2.2.2]**:

First, the longitudinal stiffness parameter, K_g, must be computed **LRFD [4.6.2.2.1]**:

$$K_g := n \cdot \left(I + A \cdot e_g^{\,2}\right)$$

Where:

I	= Moment of inertia of beam (in^4)
A	= Area of stringer, beam, or girder (in^2)
e_g	= Distance between the centers of gravity of the basic beam and deck (in)

Longitudinal Stiffness Parameter, K_g				
	Region A (Pos. Mom.)	**Region B (Intermediate)**	**Region C (At Pier)**	**Weighted Average ***
Length (Feet)	84	20	16	
n	8	8	8	
I (Inches^4)	23,605.3	34,639.8	65,426.6	
A (Inches^2)	49.750	63.750	100.500	
e_g (Inches)	35.978	35.777	36.032	
K_g (Inches^4)	704,020	929,915	1,567,250	856,767

Table E45-4.4-1
Longitudinal Stiffness Parameter

After the longitudinal stiffness parameter is computed, **LRFD [Table 4.6.2.2.1-1]** is used to find the letter corresponding with the superstructure cross section. The letter corresponding with the superstructure cross section in this design example is "a."

If the superstructure cross section does not correspond with any of the cross sections illustrated in **LRFD [Table 4.6.2.2.1-1]**, then the bridge should be analyzed as presented in **LRFD [4.6.3]**.

Based on cross section "a", **LRFD [Table 4.6.2.2.2b-1 & Table 4.6.2.2.2.3a-1]** are used to compute the distribution factors for moment and shear, respectively.

For the 0.4L point:

$K_g = 856766.65$ in^4

$L := 120$ ft

For one design lane loaded, the distribution of live load per lane for moment in interior beams is as follows **LRFD [4.6.2.2.2b-1]**:

$$g_{m1} := 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0L\cdot t_s^{\,3}}\right)^{0.1}$$

$\boxed{g_{m1} = 0.466}$ lanes

For two or more design lanes loaded, the distribution of live load per lane for moment in interior beams is as follows **LRFD [Table 4.6.2.2.2b-1]**:

$$g_{m2} := 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0\cdot L\cdot t_s^{\,3}}\right)^{0.1}$$

$\boxed{g_{m2} = 0.688}$ lanes

The live load distribution factors for shear for an interior girder are computed in a similar manner. The range of applicability is similar to that for moment **LRFD [Table 4.6.2.2.3a-1]**.

For one design lane loaded, the distribution of live load per lane for shear in interior beams is as follows:

$$g_{v1} := 0.36 + \frac{S}{25.0}$$

$\boxed{g_{v1} = 0.750}$ lanes

For two or more design lanes loaded, the distribution of live load per lane for shear in interior beams is as follows:

$$g_{v2} := 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0}$$

$\boxed{g_{v2} = 0.935}$ lanes

Since this bridge has no skew, the skew correction factor does not need to be considered for this design example **LRFD [4.6.2.2.2e & 4.6.2.2.3c]**.

HL-93 Live Load Effects (for Interior Beams)											
Live Load Effect	**Location in Span 1**										
	0.0L	**0.1L**	**0.2L**	**0.3L**	**0.4L**	**0.5L**	**0.6L**	**0.7L**	**0.8L**	**0.9L**	**1.0L**
Maximum positive moment (K-ft)	0.0	848.1	1435.4	1780.1	1916.6	1859.0	1626.9	1225.6	699.7	263.6	0.0
Maximum negative moment (K-ft)	0.0	-134.3	-268.7	-403.0	-537.4	-671.7	-806.0	-940.4	-1087.0	-1591.6	-2414.2
Maximum positive shear (kips)	111.1	92.9	76.0	60.4	46.4	34.0	23.3	14.5	7.6	3.0	0.0
Maximum negative shear (kips)	-15.2	-15.7	-21.9	-35.0	-49.2	-63.6	-78.1	-92.3	-106.1	-119.3	-132.0

Table 45E-4.4-2
Live Load Effects

The live load values for HL-93 loading, as presented in the previous table, are computed based on the product of the live load effect per lane and live load distribution factor. These values also include the effects of dynamic load allowance. However, it is important to note that the dynamic load allowance is applied only to the design truck or tandem. The dynamic load allowance is not applied to pedestrian loads or to the design lane load **LRFD [3.6.1, 3.6.2, 4.6.2.2]**.

Two sections will be checked for illustrative purposes. First, the ratings will be performed for the location of maximum positive moment, which is at 0.4L in Span 1. Second, the ratings will be performed for the location of maximum negative moment and maximum shear, which is at the pier.

The following are for the location of maximum positive moment, which is at 0.4L in Span 1, as shown in Figure E453.4-1.

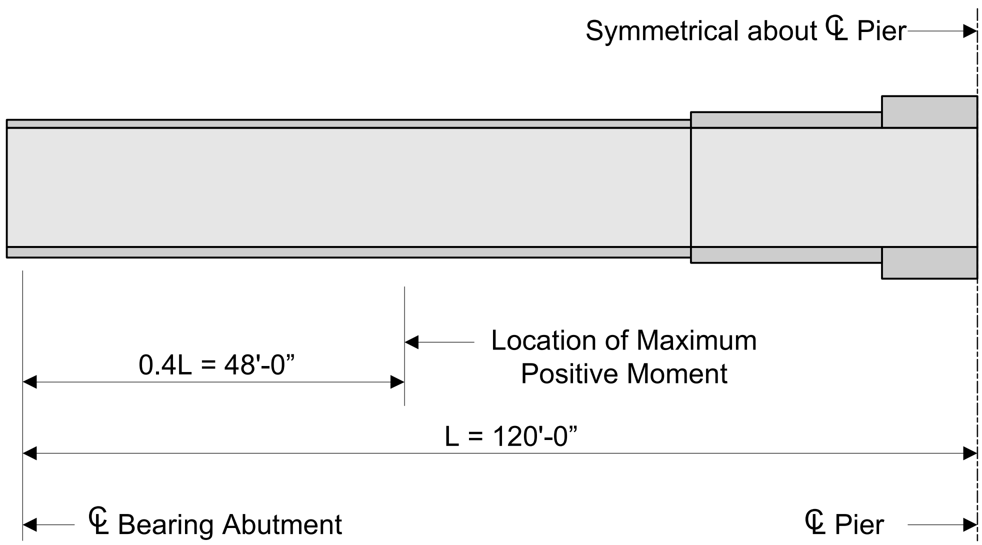


Figure E45-4.4-1
Location of Maximum Positive Moment

E45-4.5 Compute Plastic Moment Capacity - Positive Moment Region

For composite sections, the plastic moment, M_p, is calculated as the first moment of plastic forces about the plastic neutral axis **LRFD [Appendix D6.1]**.

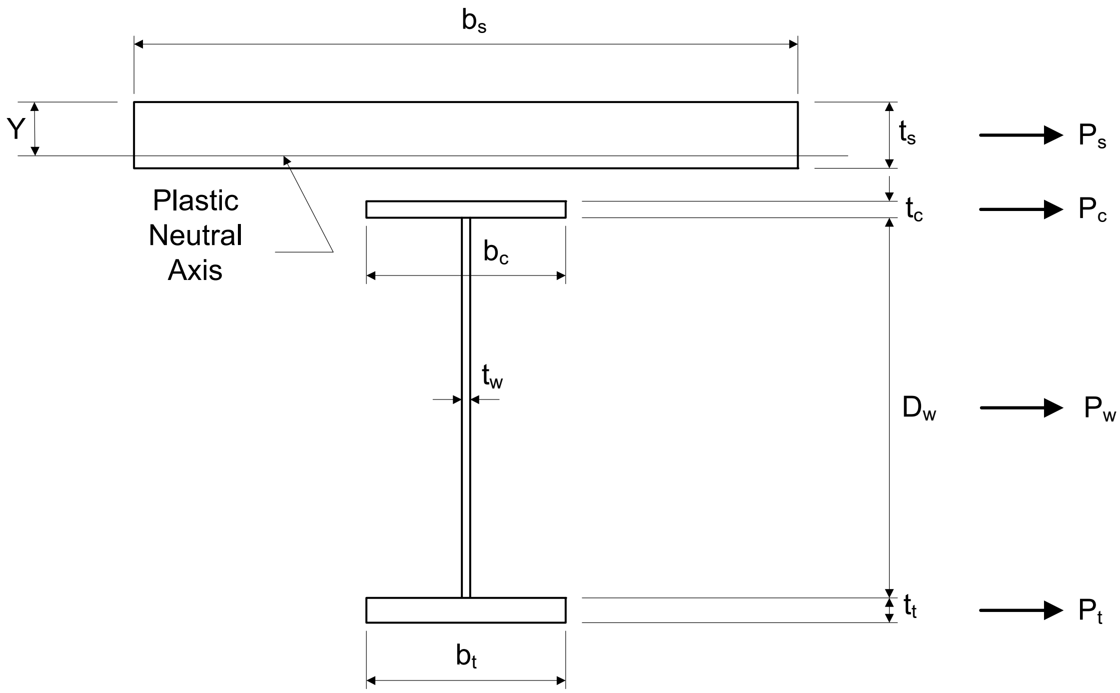


Figure E45-4.5-1
Computation of Plastic Moment Capacity for Positive Bending Sections

For the tension flange:

$$P_t = F_{yt} \cdot b_t \cdot t_t$$

Where:

F_{yt}	= Specified minimum yield strength of a tension flange (ksi)
b_t	= Full width of the tension flange (in)
t_t	= Thickness of tension flange (in)

$F_{yt} := 50$ ksi

$b_t := 14$ in

$t_t := 0.875$ in

$P_t := F_{yt} \cdot b_t \cdot t_t$ $\boxed{P_t = 613}$ kips

For the web:

$P_w := F_{yw} \cdot D \cdot t_w$

Where:

F_{yw} = Specified minimum yield strength of a web (ksi)

$F_{yw} := 50$ ksi

$D = 54.00$ in

$t_w = 0.50$ in

$P_w := F_{yw} \cdot D \cdot t_w$ $\boxed{P_w = 1350}$ kips

For the compression flange:

$P_c = F_{yc} \cdot b_c \cdot t_c$

Where:

F_{yc} = Specified minimum yield strength of a compression flange (ksi)

b_c = Full width of the compression flange (in)

t_c = Thickness of compression flange (in)

$F_{yc} := 50$ ksi

$b_c := 14$ in

$t_c := 0.75$ in

$P_c := F_{yc} \cdot b_c \cdot t_c$ $\boxed{P_c = 525}$ kips

For the slab:

$P_s = 0.85 \cdot f'_c \cdot b_s \cdot t_s$

Where:

b_s = Effective width of concrete deck (in)

t_s = Thickness of concrete deck (in)

$f'_c = 4.00$ ksi

$b_s := 109$ in

$t_s = 8.50$ in

$P_s := 0.85 \cdot f'_c \cdot b_s \cdot t_s$ $\boxed{P_s = 3150}$ kips

The forces in the longitudinal reinforcement may be conservatively neglected in regions of positive flexure.

Check the location of the plastic neutral axis, as follows:

$P_t + P_w = 1963$ kips $\boxed{P_c + P_s = 3675}$ kips

$P_t + P_w + P_c = 2488$ kips $\boxed{P_s = 3150}$ kips

Therefore, the plastic neutral axis is located within the slab **LRFD [Table D6.1-1]**.

$$Y := (t_s)\cdot\left(\frac{P_c + P_w + P_t}{P_s}\right) \qquad \boxed{Y = 6.71} \text{ in}$$

Check that the position of the plastic neutral axis, as computed above, results in an equilibrium condition in which there is no net axial force.

$$\text{Compression} := 0.85\cdot f'_c\cdot b_s\cdot Y \qquad \boxed{\text{Compression} = 2487} \text{ kips}$$

$$\text{Tension} := P_t + P_w + P_c \qquad \boxed{\text{Tension} = 2488} \text{ kips} \quad \text{OK}$$

The plastic moment, M_p, is computed as follows, where d is the distance from an element force (or element neutral axis) to the plastic neutral axis **LRFD [Table D6.1-1]**:

$$d_c := \frac{-t_c}{2} + 3.75 + t_s - Y \qquad \boxed{d_c = 5.16} \text{ in}$$

$$d_w := \frac{D}{2} + 3.75 + t_s - Y \qquad \boxed{d_w = 32.54} \text{ in}$$

$$d_t := \frac{t_t}{2} + D + 3.75 + t_s - Y \qquad \boxed{d_t = 59.98} \text{ in}$$

$$M_p := \frac{\frac{Y^2\cdot P_s}{2\cdot t_s} + (P_c\cdot d_c + P_w\cdot d_w + P_t\cdot d_t)}{12} \qquad \boxed{M_p = 7643} \text{ kip-ft}$$

E45-4.6 Determine if Section is Compact or Noncompact - Positive Moment Region

Since the section is in a straight bridge, the next step in the design process is to determine if the section is compact or noncompact. This, in turn, will determine which formulae should be used to compute the flexural capacity of the girder.

If the specified minimum yield strengths of the flanges do not exceed 70.0 ksi and the girder does not have longitudinal stiffeners, then the first step is to check the compact-section web slenderness provisions, as follows **LRFD [6.10.6.2.2]**:

$$\frac{2\cdot D_{cp}}{t_w} \le 3.76\cdot\sqrt{\frac{E}{F_{yc}}}$$

Where:

D_{cp} = Depth of web in compression at the plastic moment (in)

Since the plastic neutral axis is located within the slab,

$D_{cp} := 0$ in

Therefore the web is deemed compact. Since this is a composite section in positive flexure and there are no holes in the tension flange at this section, the flexural resistance is computed as defined by the composite compact-section positive flexural resistance provisions of **LRFD [6.10.7.1.2]**.

E45-4.7 Flexural Resistance of Composite Section - Positive Moment Region

Since the section was determined to be compact, and since it is a composite section in the positive moment region with no holes in the tension flange, the flexural resistance is computed in accordance with **LRFD [6.10.7.1.2]**.

$$M_{n\_0.4L} = 1.3 \cdot R_h \cdot M_y$$

Where:

R_h = Hybrid factor

M_y = Yield Moment (kip-in)

All design sections of this girder are homogenous. That is, the same structural steel is used for the top flange, the web, and the bottom flange. Therefore, the hybrid factor, R_h, is as follows **LRFD [6.10.1.10.1]**:

$R_h := 1.0$

The yield moment, M_y, is computed as follows **LRFD [Appendix D6.2.2]**:

$$F_y = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}}$$

Where:

M_{D1} = Bending moment caused by the factored permanent load applied before the concrete deck has hardened or is made composite (kip-in)

S_{NC} = Noncomposite elastic section modulus (in^3)

M_{D2} = Bending moment caused by the factored permanent load applied to the long-term composite section (kip-in)

S_{LT} = Long-term composite elastic section modulus (in^3)

M_{AD} = Additional bending moment that must be applied to the short-term composite section to cause nominal yielding in either steel flange (kip-in)

S_{ST} = Short-term composite elastic section modulus (in^3)

$M_y = M_{D1} + M_{D2} + M_{AD}$

$F_y := 50$ ksi

$M_{D1} := [1.25 \cdot (M_{girder} + M_{deck} + M_{misc})]$ $\boxed{M_{D1} = 1378}$ kip-ft

$M_{D2} := (1.25 \cdot M_{DC2})$ $\boxed{M_{D2} = 171}$ kip-ft

For the bottom flange:

$S_{NC\_pos} = 877.63$ in^3

$S_{LT\_pos} = 1219.60$ in^3

$S_{ST\_pos} = 1332.01$ in^3

$$M_{AD} := \left[\frac{S_{ST\_pos}}{12^3} \cdot \left(F_y \cdot 144 - \frac{M_{D1}}{\frac{S_{NC\_pos}}{12^3}} - \frac{M_{D2}}{\frac{S_{LT\_pos}}{12^3}}\right)\right]$$ $\boxed{M_{AD} = 3272}$ kip-ft

$M_{ybot} := M_{D1} + M_{D2} + M_{AD}$ $\boxed{M_{ybot} = 4821}$ kip-ft

For the top flange:

$S_{NC\_pos\_top} = 821.67$ in^3

$S_{LT\_pos\_top} = 3995.47$ in^3

$S_{ST\_pos\_top} = 19473.97$ in^3

$$M_{AD} := \frac{S_{ST\_pos\_top}}{12^3} \cdot \left(F_y \cdot 144 - \frac{M_{D1}}{\frac{S_{NC\_pos\_top}}{12^3}} - \frac{M_{D2}}{\frac{S_{LT\_pos\_top}}{12^3}}\right)$$

$\boxed{M_{AD} = 47658}$ kip-ft

$M_{ytop} := M_{D1} + M_{D2} + M_{AD}$ $\boxed{M_{ytop} = 49207}$ kip-ft

The yield moment, M_y, is the lesser value computed for both flanges. Therefore, M_y is determined as follows **LRFD [Appendix D6.2.2]**:

$M_y := \min(M_{ybot}, M_{ytop})$ $\boxed{M_y = 4821}$ kip-ft

Therefore, for the positive moment region of this design example, the nominal flexural resistance is computed as follows **LRFD [6.10.7.1.2]**:

$D_p \le 0.1 D_t$

$D_p := Y$ $\boxed{D_p = 6.71}$ in

$D_t := 0.875 + 54 + .75 + 8$ $\boxed{D_t = 63.63}$ in

$\boxed{0.1 \cdot D_t = 6.36}$ $< D_p$

Therefore

$$M_{n\_0.4L} := M_p \cdot \left(1.07 - 0.7 \cdot \frac{D_p}{D_t}\right)$$ $\boxed{M_{n\_0.4L} = 7614}$ kip-ft

Since this is neither a simple span nor a continuous span where the span and the sections in the negative-flexure region over the interior supports satisfy the special conditions outlined at the end of **LRFD[6.10.7.1.2]**, the nominal flexural resistance of the section must not exceed the following:

$M_{n\_0.4L} := 1.3 \cdot R_h \cdot M_y$ $\boxed{M_{n\_0.4L} = 6267}$ kip-ft

The ductility requirement is checked as follows **LRFD [6.10.7.3]**:

$D_p \leq 0.42D_t$

Where:

D_p = Distance from top of the concrete deck to the neutral axis of the composite section at the plastic moment (in)

D_t = Total depth of the composite section (in)

$\boxed{0.42 \cdot D_t = 26.72}$ in OK

The factored flexural resistance, M_r, is computed as follows (note that since there is no curvature, skew and wind load is not considered under the Strength I load combination, the flange lateral bending stress is taken as zero in this case **LRFD [6.10.7.1.1]**:

$$M_u + \frac{1}{3}(0) \leq \phi_f M_n$$

Where:

M_u = Moment due to the factored loads (kip-in)

M_n = Nominal flexural resistance of a section (kip-in)

$\phi_f := 1.00$

$M_r := \phi_f \cdot M_{n\_0.4L}$ $\boxed{M_r = 6267}$ kip-ft

E45-4.8 Design Load Rating @ 0.4L

$$RF = \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_0.4L} - \gamma_{DC}(DC)}{\gamma_L(LLIM)}$$

Where:

Load Factors per Table 45.3-1

$\gamma_{Linv} := 1.75$

$\gamma_{Lop} := 1.35$

$\gamma_{DC} := 1.25$

Resistance Factors

$\phi := 1.0$ **MBE [6A.7.3]**

$\phi_c := 1.0$ per 45.3.7.4

$\phi_s := 1.0$ per 45.3.7.5

$M_{DC1} := M_{girder} + M_{deck} + M_{misc}$ $M_{DC1} = 1102.07$ ft – kips

$M_{LLIM} := M_{LL}$ $M_{LLIM} = 1916.55$ ft – kips

A. Strength Limit State

Inventory

$$RF_{inv\_0.4L} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_0.4L} - \gamma_{DC} \cdot M_{DC1} - \gamma_{DC} \cdot M_{DC2}}{\gamma_{Linv} \cdot (M_{LLIM})}$$

$RF_{inv\_0.4L} = 1.41$

Operating

$$RF_{op\_0.4L} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_0.4L} - \gamma_{DC} \cdot M_{DC1} - \gamma_{DC} \cdot M_{DC2}}{\gamma_{Lop} \cdot (M_{LLIM})}$$

$RF_{op\_0.4L} = 1.82$

B. Service II Limit State

$$RF = \frac{f_R - \gamma_D \cdot (f_D)}{\gamma_L \cdot (f_{LLIM})}$$

Allowable Flange Stress per **LRFD 6.10.4.2.2**

$$f_R = 0.95 R_b \cdot R_h \cdot F_y$$

Checking only the tension flange as compression flanges typically do not control for composite sections.

$R_b := 1.0$ For tension flanges

$R_h := 1.0$ For non-hybrid sections

$f_R := 0.95 \cdot R_b \cdot R_h \cdot F_y$

$f_R = 47.50$ ksi

$f_D = f_{DC1} + f_{DC2}$

$$f_D := \left(\frac{M_{DC1} \cdot 12}{S_{NC\_pos}}\right) + \left(\frac{M_{DC2} \cdot 12}{S_{LT\_pos}}\right)$$

$f_D = 16.42$ ksi

$$f_{LLIM} := \frac{M_{LLIM} \cdot 12}{S_{ST\_pos}}$$

$f_{LLIM} = 17.27$ ksi

Load Factors Per Table 45.3-1

$\gamma_D := 1.0$

$\gamma_{Lin} := 1.3$ Inventory

$\gamma_{Lop} := 1.0$ Operating

Inventory

$$RF_{inv\_0.4L\_service} := \frac{f_R - \gamma_D \cdot f_D}{\gamma_{Lin} \cdot f_{LLIM}}$$

$RF_{inv\_0.4L\_service} = 1.38$

Operating

$$RF_{op\_0.4L\_service} := \frac{f_R - \gamma_D \cdot f_D}{\gamma_{Lop} \cdot f_{LLIM}}$$

$RF_{op\_0.4L\_service} = 1.80$

E45-4.9 Check Section Proportion Limits - Negative Moment Region

Now the specification checks are repeated for the location of maximum negative moment, whicl is at the pier, as shown in Figure 24E1.17-1. This is also the location of maximum shear in this case.

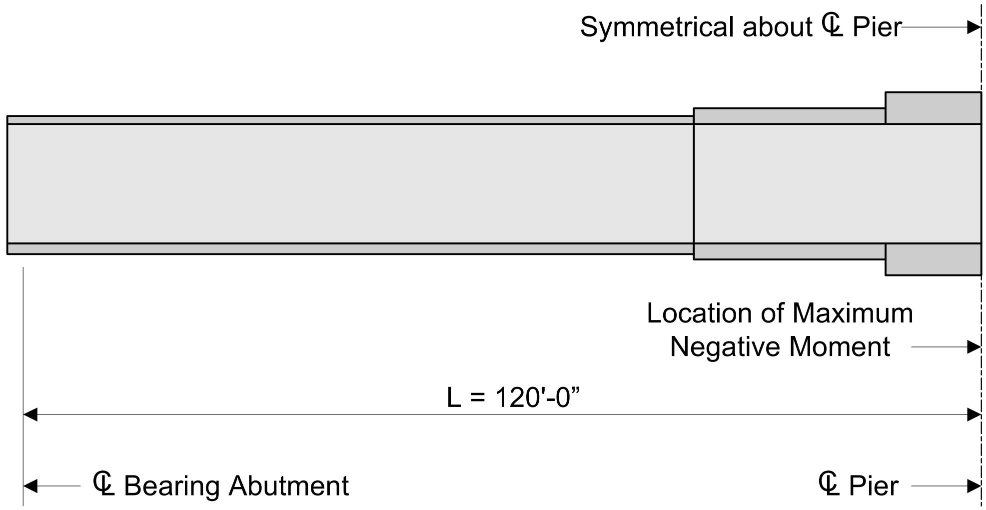


Figure E45-4.9-1
Location of Maximum Negative Moment

Several checks are required to ensure that the proportions of the girder section are within specified limits **LRFD [6.10.2]**.

The first section proportion check relates to the web slenderness **LRFD [6.10.2.1]**. For a section without longitudinal stiffeners, the web must be proportioned such that:

$\frac{D}{t_w} \le 150$ $\boxed{\frac{D}{t_w} = 108.00}$ OK

The second set of section proportion checks relate to the general proportions of the section **LRFD [6.10.2.2]**. The compression and tension flanges must be proportioned such that:

$\frac{b_f}{2 \cdot t_f} \le 12.0$

$b_f := 14$

$t_f := 2.50$ $\boxed{\frac{b_f}{2 \cdot t_f} = 2.80}$ OK

$b_f \ge \frac{D}{6}$ $\boxed{\frac{D}{6} = 9.00}$ in OK

$t_f \ge 1.1 \cdot t_w$ $\boxed{1.1t_w = 0.55}$ in OK

$$0.1 \le \frac{I_{yc}}{I_{yt}} \le 10$$

$I_{yc} := \dfrac{2.75 \cdot 14^3}{12}$ $\boxed{I_{yc} = 628.83}$ in^4

$I_{yt} := \dfrac{2.50 \cdot 14^3}{12}$ $\boxed{I_{yt} = 571.67}$ in^4

$\boxed{\dfrac{I_{yc}}{I_{yt}} = 1.100}$ OK

E45-4.10 Compute Plastic Moment Capacity - Negative Moment Region

For composite sections, the plastic moment, M_p, is calculated as the first moment of plastic forces about the plastic neutral axis **LRFD [Appendix D6.1]**. For composite sections in negative flexure, the concrete deck is ignored and the longitudinal deck reinforcement is included in the computation of M_p.

The plastic force in the tension flange, P_t, is calculated as follows:

$t_t := 2.50$ in

$P_t := F_{yt} \cdot b_t \cdot t_t$ $\boxed{P_t = 1750}$ kips

The plastic force in the web, P_w, is calculated as follows:

$P_w := F_{yw} \cdot D \cdot t_w$ $\boxed{P_w = 1350}$ kips

The plastic force in the compression flange, P_c, is calculated as follows:

$t_c := 2.75$ in

$P_c := F_{yc} \cdot b_c \cdot t_c$ $\boxed{P_c = 1925}$ kips

The plastic force in the top layer of longitudinal deck reinforcement, P_{rt}, used to compute the plastic moment is calculated as follows:

$$P_{rt} = F_{yrt} \cdot A_{rt}$$

Where:

F_{yrt} = Specified minimum yield strength of the top layer of longitudinal concrete deck reinforcement (ksi)

A_{rt} = Area of the top layer of longitudinal reinforcement within the effective concrete deck width (in^2)

$F_{yrt} := 60$ ksi

$A_{rt} := 0.44 \cdot \left(\frac{b_{effflange} \cdot 12}{7.5} \right)$ $\boxed{A_{rt} = 6.39}$ in^2

$P_{rt} := F_{yrt} \cdot A_{rt}$ $\boxed{P_{rt} = 384}$ kips

The plastic force in the bottom layer of longitudinal deck reinforcement, P_{rb}, used to compute the plastic moment is calculated as follows (WisDOT Policy is to ignore bottom mat steel)

$$P_{rb} = F_{yrb} \cdot A_{rb}$$

Where:

F_{yrb} = Specified minimum yield strength of the bottom layer of longitudinal concrete deck reinforcement (ksi)

A_{rb} = Area of the bottom layer of longitudinal reinforcement within the effective concrete deck width (in^2)

$F_{yrb} := 60$ ksi

$A_{rb} := 0 \cdot \left(\frac{b_{effflange} \cdot 12}{1} \right)$ $\boxed{A_{rb} = 0.00}$ in^2

$P_{rb} := A_{rb} \cdot F_{yrb}$ $\boxed{P_{rb} = 0}$ kips

NOTE: For continuous girder type bridges, the negative moment steel shall conservatively consist of only the top mat of steel over the piers per **45.6.3**

Check the location of the plastic neutral axis, as follows:

$\boxed{P_c + P_w = 3275}$ kips

$\boxed{P_t + P_{rb} + P_{rt} = 2134}$ kips

$\boxed{P_c + P_w + P_t = 5025}$ kips

$\boxed{P_{rb} + P_{rt} = 384}$ kips

Therefore the plastic neutral axis is located within the web **LRFD [Appendix Table D6.1-2]**.

$Y := \left(\frac{D}{2} \right) \cdot \left(\frac{P_c - P_t - P_{rt} - P_{rb}}{P_w} + 1 \right)$ $\boxed{Y = 22.83}$ in

Although it will be shown in the next design step that this section qualifies as a nonslender web section at the strength limit state, the optional provisions of Appendix A to **LRFD [6]** are not employed in this example. Thus, the plastic moment is not used to compute the flexural resistance and therefore does not need to be computed.

E45-4.11 Determine if Section is a Compact-Web, Noncompact-Web, or Slender-Web Section - Negative Moment Region

Since the section is in a straight bridge, the next step is to determine if the section is a compact-web, noncompact-web, or slender-web section. This, in turn, will determine which formulae should be used to compute the flexural capacity of the girder.

Where the specified minimum yield strengths of the flanges do not exceed 70.0 ksi and the girder does not have longitudinal stiffeners, then the first step is to check the noncompact-web slenderness limit, as follows **LRFD [6.10.6.2.3]**:

$$\frac{2 \cdot D_c}{t_w} \le 5.7 \cdot \sqrt{\frac{E}{F_{yc}}}$$

At sections in negative flexure, D_c of the composite section consisting of the steel section plus the longitudinal reinforcement is to be used at the strength limit state.

$D_c := 30.872 - 2.75$ (see Figure 24E1.2-1 and Table 24E1.3-2)

$\boxed{D_c = 28.12}$ in

$\boxed{\frac{2 \cdot D_c}{t_w} = 112.5}$

$\boxed{5.7 \cdot \sqrt{\frac{E_s}{F_{yc}}} = 137.3}$

The section is a nonslender web section (i.e. either a compact-web or noncompact-web section). Next, check:

$I_{yc} := \frac{2.75 \cdot 14^3}{12}$ $\boxed{I_{yc} = 628.83}$ in^4

$I_{yt} := \frac{2.5 \cdot 14^3}{12}$ $\boxed{I_{yt} = 571.67}$ in^4

$\boxed{\frac{I_{yc}}{I_{yt}} = 1.10}$ > 0.3 OK

Therefore, the web qualifies to use the optional provisions of **LRFD [Appendix A6]** to compute the flexural resistance. However, since the web slenderness is closer to the noncompact web slenderness limit than the compact web slenderness limit in this case, the simpler equations of **LRFD [6.10.8]**, which assume slender-web behavior and limit the resistance to Fyc or below, will conservatively be applied in this example to compute the flexural resistance at the strength limit state. The investigation proceeds by calculating the flexural resistance of the discretely braced compression flange.

E45-4.12 Rating for Flexure - Strength Limit State - Negative Moment Region

The nominal flexural resistance of the compression flange shall be taken as the smaller of the local buckling resistance and the lateral torsional buckling resistance **LRFD [6.10.8.2.2 & 6.10.8.2.3]**.

Local buckling resistance **LRFD [6.10.8.2.2]**:

$b_{fc} := 14$ (see Figure 24E1.2-1)

$t_{fc} := 2.75$ (see Figure 24E1.2-1)

$$\lambda_f := \frac{b_{fc}}{2 \cdot t_{fc}} \qquad \boxed{\lambda_f = 2.55}$$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_{yc}}} \qquad \boxed{\lambda_{pf} = 9.15}$$

Since $\lambda_f < \lambda_{pf}$, F_{nc} is calculated using the following equation:

$$F_{nc} := R_b \cdot R_h \cdot F_{yc}$$

Since $2D_c/t_w$ is less than λ_{rw} (calculated above), R_b is taken as 1.0 **LRFD [6.10.1.10.2]**.

$$\boxed{F_{nc} = 50.00} \quad \text{ksi}$$

Lateral torsional buckling resistance **LRFD [6.10.8.2.3]**:

$$r_t := \frac{b_{fc}}{\sqrt{12 \cdot \left(1 + \frac{1}{3} \cdot \frac{D_c \cdot t_w}{b_{fc} \cdot t_{fc}}\right)}} \qquad \boxed{r_t = 3.82} \quad \text{in}$$

$$L_p := 1.0 \cdot r_t \cdot \sqrt{\frac{E}{F_{yc}}} \qquad \boxed{L_p = 91.90} \quad \text{in}$$

$$F_{yr} := \max\left(\min\left(0.7 \cdot F_{yc}, F_{yw}\right), 0.5 \cdot F_{yc}\right) \qquad \boxed{F_{yr} = 35.00} \quad \text{ksi}$$

$$L_r := \pi \cdot r_t \cdot \sqrt{\frac{E}{F_{yr}}} \qquad \boxed{L_r = 345.07} \quad \text{in}$$

$$L_b = 240.00$$

The moment gradient correction factor, C_b, is computed as follows:

Where the variation in the moment along the entire length between brace points is concave in shape, which is the case here, $f_1 = f_0$. (calculated below based on the definition of f_0 given in **LRFD [6.10.8.2.3]**).

$M_{NCDC0.8L} := 110.2 + 756.0 + 19.9$ $\boxed{M_{NCDC0.8L} = 886.10}$ kip-ft

$S_{NCDC0.8L} := 2278.2$ in^3

$M_{par0.8L} := 83.4$ kip-ft

$M_{LL0.8L} := 1087.0$ kip-ft

$S_{rebar0.8L} := 2371.9$ in^3

$$f_1 := 1.25 \cdot \frac{M_{NCDC0.8L} \cdot 12}{S_{NCDC0.8L}} + 1.25 \cdot \frac{M_{par0.8L} \cdot 12}{S_{rebar0.8L}} + 1.75 \cdot \frac{M_{LL0.8L} \cdot 12}{S_{rebar0.8L}}$$

$\boxed{f_1 = 15.99}$ ksi

$f_2 := 46.50$ ksi (Table E24-1.6-2)

$\boxed{\frac{f_1}{f_2} = 0.34}$

$$C_b := 1.75 - 1.05 \cdot \left(\frac{f_1}{f_2}\right) + 0.3 \cdot \left(\frac{f_1}{f_2}\right)^2 \quad < 2.3$$ $\boxed{C_b = 1.42}$

Therefore:

$$F_{nc} := C_b \cdot \left[1 - \left(1 - \frac{F_{yr}}{R_h \cdot F_{yc}}\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \cdot R_b \cdot R_h \cdot F_{yc}$$

$\boxed{F_{nc} = 58.72}$ ksi

$F_{nc} \leq R_b \cdot R_h \cdot F_{yc}$ $\boxed{R_b \cdot R_h \cdot F_{yc} = 50.00}$ ksi

Use:

$F_{nc} := 50$ ksi

$\boxed{\phi_f \cdot F_{nc} = 50.00}$ ksi

$$M_{n\_1.0L} := F_{nc} \cdot S_{rebar} \cdot \left(\frac{1}{12}\right)$$ $M_{n\_1.0L} = 9883.01$ ft – kips

E45-4.13 Design Load Rating @ Pier

$$RF = \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_1.0L} - \gamma_{DC}(M_{DC\_neg})}{\gamma_L(M_{LLIM\_neg})}$$

Where:

Load Factors per Table 45.3-1

$\gamma_{Linv} := 1.75$

$\gamma_{Lop} := 1.35$

$\gamma_{DC} := 1.25$

Resistance Factors

$\phi := 1.0$ **MBE [6A.7.3]**

$\phi_c := 1.0$ per 45.3.7.4

$\phi_s := 1.0$ per 45.3.7.5

$M_{DC1\_neg} := M_{girder\_neg} + M_{deck\_neg} + M_{misc\_neg}$ $M_{DC1\_neg} = -3073.22$ ft – kips

$M_{LLIM\_neg} := M_{LL\_neg}$ $M_{LLIM\_neg} = -2414.17$ ft – kips

A. Strength Limit State

$$RF_{inv\_1.0L} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot (-F_{nc}) - \gamma_{DC} \cdot \frac{M_{DC1\_neg} \cdot 12}{S_{NC\_neg}} - \gamma_{DC} \cdot \frac{M_{DC2\_neg} \cdot 12}{S_{rebar}}}{\gamma_{Linv} \cdot \left(\frac{M_{LLIM\_neg} \cdot 12}{S_{rebar}}\right)}$$

$RF_{inv\_1.0L} = 1.30$

$$RF_{op\_1.0L} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot (-F_{nc}) - \gamma_{DC} \cdot \frac{M_{DC1\_neg} \cdot 12}{S_{NC\_neg}} - \gamma_{DC} \cdot \frac{M_{DC2\_neg} \cdot 12}{S_{rebar}}}{\gamma_{Lop} \cdot \left(\frac{M_{LLIM\_neg} \cdot 12}{S_{rebar}}\right)}$$

$RF_{op\_1.0L} = 1.68$

B. Service II Limit State

$$RF = \frac{f_R - \gamma_D \cdot (f_D)}{\gamma_L \cdot (f_{LLIM})}$$

Allowable Flange Stress per **LRFD [6.10.4.2.2]**

$$f_R := 0.95 \cdot R_h \cdot F_y$$

$R_h := 1.0$ For non-hybrid sections

$f_R := 0.95 \cdot R_b \cdot R_h \cdot F_y$

$f_R = 47.50$ ksi

$f_D = f_{DC1} + f_{DC2}$

$$f_D := -\left[\left(\frac{M_{DC1\_neg} \cdot 12}{S_{NC\_neg}}\right) + \left(\frac{M_{DC2\_neg} \cdot 12}{S_{LT\_neg}}\right)\right]$$

$f_D = 17.68$ ksi

$$f_{LLIM} := \frac{-M_{LL\_neg} \cdot 12}{S_{rebar}}$$

$f_{LLIM} = 12.21$ ksi

Load Factors Per Table 45.3-1

$\gamma_D := 1.0$

$\gamma_{Lin} := 1.3$ Inventory

$\gamma_{Lop} := 1.0$ Operating

Inventory

$$RF_{inv\_1.0L\_service} := \frac{f_R - \gamma_D \cdot f_D}{\gamma_{Lin} \cdot f_{LLIM}}$$

$RF_{inv\_1.0L\_service} = 1.88$

Operating

$$RF_{op\_1.0L\_service} := \frac{f_R - \gamma_D \cdot f_D}{\gamma_{Lop} \cdot f_{LLIM}}$$

$RF_{op\_1.0L\_service} = 2.44$

E45-4.14 Rate for Shear - Negative Moment Region

Shear must be checked at each section of the girder. For this Rating example, shear is maximum at the pier, and will only be checked there for illustrative purposes.

The transverse intermediate stiffener spacing is 120". The spacing of the transverse intermediate stiffeners does not exceed 3D, therefore the section can be considered stiffened and the provisions of **LRFD [6.10.9.3]** apply.

$d_o := 120$ in

$D = 54.00$ in

$$k := 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2} \qquad k = 6.01$$

$$\frac{D}{t_w} = 108.00 \qquad \frac{D}{t_w} \geq 1.40 \cdot \sqrt{\frac{E_s \cdot k}{F_{yw}}} \qquad 1.40 \cdot \sqrt{\frac{E_s \cdot k}{F_{yw}}} = 82.67$$

$$C := \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \cdot \left(\frac{E_s \cdot k}{F_{yw}}\right) \qquad \boxed{C = 0.469}$$

The plastic shear force, V_p, is then:

$$V_p := 0.58 \cdot F_{yw} \cdot D \cdot t_w \qquad \boxed{V_p = 783.00} \text{ kips}$$

$$V_n := V_p \cdot \left[C + \frac{0.87 \cdot (1 - C)}{\sqrt{1 + \left(\frac{d_o}{D}\right)^2}}\right] \qquad \boxed{V_n = 515.86} \text{ kips}$$

The factored shear resistance, V_r, is computed as follows **LRFD [6.10.9.1]**:

$\phi_v := 1.00$

$$V_r := \phi_v \cdot V_n \qquad \boxed{V_r = 515.86} \text{ kips}$$

HL-93 Maximum Shear @ Pier:

$$V_{DC1} := V_{girder} + V_{deck} + V_{misc} \qquad V_{DC1} = -108.84 \text{ kips}$$

$V_{DC2} = -12.03$ kips

$V_{LL} = -131.95$ kips

$M_{LLIM\_neg} = -2414.17$ ft – kips

E45-4.15 Design Load Rating @ Pier for Shear

$$RF = \frac{\phi \cdot \phi_c \cdot \phi_s \cdot V_n - \gamma_{DC}(V_{DC})}{\gamma_L(V_{LLIM})}$$

Where:

Load Factors per Table 45.3-1	Resistance Factors	
$\gamma_{Linv} := 1.75$	$\phi := 1.0$	**MBE [6A.7.3]**
$\gamma_{Lop} := 1.35$	$\phi_c := 1.0$	per 45.3.7.4
$\gamma_{DC} := 1.25$	$\phi_s := 1.0$	per 45.3.7.5

A. Strength Limit State

Inventory

$$RF_{inv\_shear} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot (-V_n) - \gamma_{DC} \cdot (V_{DC1} + V_{DC2})}{\gamma_{Linv} \cdot (V_{LL})}$$

$$\boxed{RF_{inv\_shear} = 1.58}$$

Operating

$$RF_{op\_shear} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot (-V_n) - \gamma_{DC} \cdot (V_{DC1} + V_{DC2})}{\gamma_{Lop} \cdot (V_{LL})}$$

$$\boxed{RF_{op\_shear} = 2.05}$$

Since RF>1.0 @ operating for all checks, Legal Load Ratings are not required for this example.

E45-4.16 - Permit Load Ratings

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed (per 45.12). Since the span lengths are less than 200', the lane loading requirements will not be considered for positive moments.

The bridge shall be analyzed for this vehicle considering both single-lane and multi-lane distribution. Also, the vehicle will be analyzed assuming it is mixing with other traffic on the bridge and that full dynamic load allowance is utilized. Future wearing surface shall not be included.

Since this example is rating a newly designed bridge, an additional check is required. The designer shall ensure that the results of the single-lane analysis are greater than 190 kips MVW. Future wearing surface shall be included in the check.

E45-4.16.1 - Wis-SPV Permit Rating with Single Lane Distribution w/ FWS

The values from this analysis are used for performing the Wis-SPV design check per 45.12

Load Distribution Factors

Single Lane Interior DF - Moment $g_{m1} = 0.47$

Single Lane Interior DF - Shear $g_{v1} = 0.75$

Load Factors per Tables 45.3-1 and 45.3-3

$\gamma_L := 1.2$

$\gamma_{DC} := 1.25$ $\gamma_{DW} := 1.50$

Wis-SPV Moments and Shears (w/o Dynamic Load allowance or Distribution Factors included)

$M_{pos} := 2842.10$ kip-ft

$M_{neg} := 2185.68$ kip-ft

$V_{max} := 154.32$ kips

$$M_{0.4L} := \frac{g_{m1}}{1.2} \cdot 1.33 \cdot M_{pos} \qquad \boxed{M_{0.4L} = 1468.47} \text{ kip-ft}$$

$$M_{1.0L} := \left(\frac{g_{m1}}{1.2}\right) \cdot ((1.33 \cdot M_{neg})) \qquad \boxed{M_{1.0L} = 1129.31} \text{ kip-ft}$$

$$V_{1.0L} := \left(\frac{g_{v1}}{1.2}\right) \cdot ((1.33 \cdot V_{max}))$$ $\boxed{V_{1.0L} = 128.28}$ kips

$$RF_{pos} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_0.4L} - \gamma_{DC} \cdot (M_{DC1} + M_{DC2}) - \gamma_{DW} \cdot M_{DW}}{\gamma_L \cdot (M_{0.4L})}$$

$\boxed{RF_{pos} = 2.55}$ $\boxed{RF_{pos} \cdot 190 = 483.65}$ kips

$$RF_{neg} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_1.0L} - \gamma_{DC} \cdot (-M_{DC1\_neg} - M_{DC2\_neg}) - \gamma_{DW} \cdot (-M_{DW\_neg})}{\gamma_L \cdot (M_{1.0L})}$$

$\boxed{RF_{neg} = 3.74}$ $\boxed{RF_{neg} \cdot 190 = 711.43}$ kips

$$RF_{shear} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot V_n - \gamma_{DC} \cdot [-(V_{DC1} + V_{DC2})] - \gamma_{DW} \cdot (-V_{DW})}{\gamma_L \cdot (V_{1.0L})}$$

$\boxed{RF_{shear} = 2.24}$ $\boxed{RF_{shear} \cdot 190 = 424.87}$ kips

424.87k > 190k minimum : CHECK OK

E45-4.16.2 - Wis-SPV Permit Rating with Single Lane Distribution w/o FWS

For use with plans and rating sheet only.

Load Distribution Factors

Single Lane Interior DF - Moment $g_{m1} = 0.47$

Single Lane Interior DF - Shear $g_{v1} = 0.75$

Load Factors per Tables 45.3-1 and 45.3-3

$\gamma_L := 1.2$

$\gamma_{DC} := 1.25$ $\gamma_{DW} := 1.50$

Wis-SPV Moments and Shears (w/o Dynamic Load allowance
or Distribution Factors included)

$M_{pos} := 2842.10$ kip-ft

$M_{neg} := 2185.68$ kip-ft

$V_{max} := 154.32$ kips

$M_{0.4L} := \frac{g_{m1}}{1.2} \cdot 1.33 \cdot M_{pos}$ $\boxed{M_{0.4L} = 1468.47}$ kip-ft

$M_{1.0L} := \left(\frac{g_{m1}}{1.2}\right) \cdot ((1.33 \cdot M_{neg}))$ $\boxed{M_{1.0L} = 1129.31}$ kip-ft

$V_{1.0L} := \left(\frac{g_{v1}}{1.2}\right) \cdot ((1.33 \cdot V_{max}))$ $V_{1.0L} = 128.28$ kips

$$RF_{pos1} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_0.4L} - \gamma_{DC} \cdot (M_{DC1} + M_{DC2})}{\gamma_L \cdot (M_{0.4L})}$$

$\boxed{RF_{pos1} = 2.68}$ $\boxed{RF_{pos1} \cdot 190 = 508.78}$ kips

$$RF_{neg1} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_1.0L} - \gamma_{DC} \cdot (-M_{DC1\_neg} - M_{DC2\_neg})}{\gamma_L \cdot (M_{1.0L})}$$

$\boxed{RF_{neg1} = 4.16}$ $\boxed{RF_{neg1} \cdot 190 = 789.64}$ kips

$$RF_{shear1} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot V_n - \gamma_{DC} \cdot [-(V_{DC1} + V_{DC2})]}{\gamma_L \cdot (V_{1.0L})}$$

$\boxed{RF_{shear1} = 2.37}$ $\boxed{RF_{shear1} \cdot 190 = 450.24}$ kips

E45-4.16.3 - Permit Rating with Multi-Lane Distribution w/o FWS

For use with plans and rating sheet only.

Load Distribution Factors

Multi Lane Interior DF - Moment $g_{m2} = 0.69$

Multi Lane Interior DF - Shear $g_{v2} = 0.93$

Load Factors per Tables 45.3-1 and 45.3-3

$\gamma_L := 1.3$

$\gamma_{DC} := 1.25$

Wis-SPV Moments and Shears (w/o Dynamic Load allowance or Distribution Factors included)

$M_{pos} := 2842.10$ kip-ft

$M_{neg} := 2185.68$ kip-ft

$V_{max} := 154.32$ kips

Multi Lane Ratings

$M_{0.4L} := g_{m2} \cdot 1.33 \cdot M_{pos}$ $\boxed{M_{0.4L} = 2600.09}$ kip-ft

$M_{1.0L} := g_{m2} \cdot (1.33 \cdot M_{neg})$ $\boxed{M_{1.0L} = 1999.56}$ kip-ft

$V_{1.0L} := g_{v2} \cdot (1.33 \cdot V_{max})$ $\boxed{V_{1.0L} = 191.88}$ kips

$$RF_{pos\_ml} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_0.4L} - \gamma_{DC} \cdot (M_{DC1} + M_{DC2})}{\gamma_L \cdot (M_{0.4L})}$$

$\boxed{RF_{pos\_ml} = 1.40}$ $\boxed{RF_{pos\_ml} \cdot 190 = 265.24}$ kips

$$RF_{neg\_ml} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_1.0L} - \gamma_{DC} \cdot \left(-M_{DC1\_neg} - M_{DC2\_neg}\right)}{\gamma_L \cdot \left(M_{1.0L}\right)}$$

$\boxed{RF_{neg\_ml} = 2.17}$ $\boxed{RF_{neg\_ml} \cdot 190 = 411.67}$ kips

$$RF_{shear\_ml} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot V_n - \gamma_{DC} \cdot \left[-\left(V_{DC1} + V_{DC2}\right)\right]}{\gamma_L \cdot \left(V_{1.0L}\right)}$$

$\boxed{RF_{shear\_ml} = 1.46}$ $\boxed{RF_{shear\_ml} \cdot 190 = 277.84}$ kips

E45-4.17 Summary of Rating

Steel Interior Girder								
Limit State		Design Load Rating		Legal Load Rating	Wis-SPV Ratings (kips)		
		Inventory	Operating		Single Lane w/ FWS	Single Lane w/o FWS	Multi Lane w/o FWS
Strength I @ 0.4L	Flexure	1.41	1.82	N/A	484	509	265
	Shear	N/A	N/A	N/A	N/A	N/A	N/A
Strength I @ 1.0L	Flexure	1.30	1.68	N/A	711	790	412
	Shear	1.58	2.05	N/A	425	450	278
Service II	0.4L	1.38	1.80	N/A	Optional		Optional
	1.0L	1.88	2.44	N/A	Optional		Optional

Table of Contents

E45-5 Reinforced Concrete Slab Rating Example - LFR

Reference E45-1 for bridge data. For LFR, the Bureau of Structures rates concrete slab structures for the Design Load (HS20) and for Permit Vehicle Loads on an interior strip equal to one foot width.

This example calculates ratings of the controlling locations at the 0.4 tenths point of span 1 for positive moment and at the pier for negative moment.

E45-5.1 Design Criteria

Geometry:

$L_1 := 38.0$ ft — Span 1 Length

$L_2 := 51.0$ ft — Span 2 Length

$L_3 := 38.0$ ft — Span 3 Length

$slab_{width} := 42.5$ ft — out to out width of slab

$cover_{top} := 2.5$ in — concrete cover on top bars (includes 1/2in wearing surface)

$cover_{bot} := 1.5$ in — concrete cover on bottom bars

$d_{slab} := 17$ in — slab depth (not including 1/2in wearing surface)

$b := 12$ in — Interior strip width for analysis

$D_{haunch} := 28$ in — haunch depth (not including 1/2in wearing surface)

$A_{st\_0.4L} := 1.71$ $\frac{in^2}{ft}$ — Area of longitudinal bottom steel at 0.4L (# 9's at 7in centers)

$A_{st\_pier} := 1.88$ $\frac{in^2}{ft}$ — Area of longitudinal top steel at Pier (# 8's at 5in centers)

Material Properties:

$f'_c := 4$ ksi — concrete compressive strength

$f_y := 60$ ksi — yield strength of reinforcement

Weights:

$w_c := 150$ pcf — concrete unit weight

$w_{LF} := 387$ plf — weight of Type LF parapet (each)

E45-5.2 Analysis of an Interior Strip - one foot width

Use Strength Limit States to rate the concrete slab bridge. **MBE [6B.5.3.2]**

E45-5.2.1 Dead Loads

The slab dead load, D_{slab}, and the section properties of the slab, do not include the 1/2 inch wearing surface. But the 1/2 inch wearing surface load, D_{WS}, of 6 psf must be including in the analysis of the slab. <u>For a one foot slab width</u>:

$D_{ws} := 6$ 1/2 inch wearing surface load, plf

The parapet dead load is uniformly distributed over the full width of the slab when analyzing an Interior Strip. <u>For a one foot slab width</u>:

$D_{para} := 2 \cdot \frac{w_{LF}}{slab_{width}}$ $\boxed{D_{para} = 18}$ plf

The unfactored dead load moments, M_D, due to slab dead load (D_{slab}), parapet dead load (D_{para}), and the 1/2 inch wearing surface (D_{ws}) are shown in Chapter 18 Example E18-1 (Table E18.4). For LFR, the total deal load moment (M_D) is the sum of the values M_{DC} and M_{DW} tabulated separately for LRFD calculations.

The structure was designed for a possible future wearing surface, D_{FWS}, of 20 psf.

$D_{FWS} := 20$ Possible wearing surface, plf

E45-5.2.2 Live Load Distribution

Live loads are distributed over an equivalent width, E, as calculated below.
The live loads to be placed on these widths are <u>wheel loads</u> (i.e., one line of wheels) or <u>half of the lane load</u>. The equivalent distribution width applies for both live load moment and shear.

Multi - Lane Loading: $E = 48.0 + 0.06 \cdot S$ $\leq$ 84 in **Std [3.24.3.2]**

Single - Lane Loading: $E = 144$ in **[45.6.2.1]**

where:

S = effective span length, in inches

For multi-lane loading:

(Span 1, 3) $E_{m13} := \min[84, 48.0 + 0.06 \cdot (38 \cdot 12)]$

$\boxed{E_{m13} = 75.36}$ in

(Span 2) $E_{m2} := \min[84, 48.0 + 0.06 \cdot (51 \cdot 12)]$

$\boxed{E_{m2} = 84.00}$ in

For single-lane loading:

(Span 1, 3) $\boxed{E_{s13} := 144.0}$ in

(Span 2) $\boxed{E_{s2} := 144.0}$ in

E45-5.2.3 Nominal Flexural Resistance: (M_n)

The depth of the compressive stress block, (a) is:

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b}$$ **Std (8-17)**

For rectangular sections, the nominal moment resistance, M_n, (tension reinforcement only) equals:

$$M_n = A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$$ **Std (8-16)**

where:

d_s = slab depth (excl. 1/2" wearing surface) - bar clearance - 1/2 bar diameter

Maximum Reinforcement Check

The area of reinforcement to be used in calculating nominal resistance (M_n) shall not exceed 75 percent of the reinforcement required for the balanced conditions **MBE [6B.5.3.2]**.

$$\rho_b := 0.85^2 \cdot \left(\frac{f'_c}{f_y}\right) \cdot \frac{87}{87 + f_y} = 0.029$$

$$A_{smax} := \rho_b \cdot b \cdot d_s$$

E45-5.2.4 General Load - Rating Equation (for flexure)

$$RF = \frac{C - A_1 \cdot M_D}{A_2 \cdot M_L \cdot (1 + I)}$$ **MBE [6B.4.1]**

where:

$C := \phi \cdot M_n$

$\phi := 0.9$ **Std [8.16.1.2.2]**

A_1 = 1.3 for Dead Loads

A_2 = Live Load factor: 2.17 for Inventory, 1.3 for Operating

M_D = Unfactored Dead Load moments

M_L = Unfactored Live Load moments

I = Live Load Impact Factor (maximum 30%)

E45-5.2.5 Design Load (HS20) Rating

Equivalent Strip Width (E) and Distribution Factor (DF):

Use the multi-lane wheel distribution width for (HS20) live load.

The distribution factor, DF, is computed for a slab width equal to one foot.

$$DF = \frac{1}{E}$$ (where E is in feet)

Spans 1 & 3:

$$DF_{13} := \frac{12}{E_{m13}} = 0.159$$ wheels / ft-slab

Span 2:

$$DF_2 := \frac{12}{E_{m2}} = 0.143$$ wheels / ft-slab

Live Load Impact Factor (I)

$$I := \frac{50}{L + 125}$$ (maximum 0.3) **Std [3.8.2.1]**

Spans 1 & 3:

$$I_{13} := \min\left(0.3, \frac{50}{L_1 + 125}\right)$$ $\boxed{I_{13} = 0.3}$

Span 2:

$$I_2 := \min\left(0.3, \frac{50}{L_2 + 125}\right) \qquad \boxed{I_2 = 0.284}$$

Live Loads (LL)

The live loads shall be determined from live load analysis software using the higher of the HS20 Truck or Lane loads.

Rating for Flexure

$$RF = \frac{\phi \cdot M_n - 1.3 \cdot M_D}{A_2 \cdot M_L \cdot (1 + I)}$$

The Design Load Rating was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing limit state and location for the HS20 load is positive moment is in span 1 at the 0.4 pt.

Span 1 (0.4 pt.)

Flexural capacity:

$$A_{st\_0.4L} = 1.71 \quad \frac{in^2}{ft}$$

$$d_s := 17.0 - cover_{bot} - \frac{1.128}{2} \qquad \boxed{d_s = 14.94} \quad in$$

$$a := \frac{A_{st\_0.4L} \cdot f_y}{0.85 \cdot f'_c \cdot b} \qquad \boxed{a = 2.51} \quad in$$

$$A_{smax} := \rho_b \cdot b \cdot d_s = 5.109 \qquad A_{smax} > A_{st\_0.4L} \quad OK$$

$$M_n := A_{st\_0.4L} \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) \qquad \boxed{M_n = 1403.4} \quad kip\text{-}in$$

$$\boxed{M_n = 117.0} \quad kip\text{-}ft$$

The dead load consists of the slab self-weight and parapet weight divided evenly along the slab width:

$M_D := 18.1 \;\; kip - ft$ (from Chapter 18 Example, Table E18.4)

The positive live load moment shall be the largest caused by the following (from live load analysis software):

Design Lane:	17.48 kip-ft
Design Truck:	24.01 kip-ft

Therefore:

$$M_L := 24.01 \qquad \text{kip} - \text{ft}$$

Inventory:

$$RF_i := \frac{\phi \cdot M_n - 1.3 \cdot M_D}{2.17 \cdot M_L \cdot (1 + I_{13})} = 1.21$$

$\boxed{\text{Inventory Rating = HS21}}$

Operating:

$$RF_o := \frac{\phi \cdot M_n - 1.3 \cdot M_D}{1.3 \cdot M_L \cdot (1 + I_{13})} = 2.01$$

$\boxed{\text{Operating Rating = HS36}}$

Rating for Shear:

Shear rating for concrete slab bridges may be ignored. Bending moment is assumed to control per **Std [3.24.4]**.

The Rating Factors, RF, for Inventory and Operating Rating are shown on the plans and also on the load rating summary sheet.

E45-5.2.6 Permit Vehicle Load Ratings

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed per **[45.12]**.

The bridge shall be analyzed for this vehicle considering both single-lane and multi-lane distribution, and full dynamic load allowance is utilized. Future wearing surface will not be considered.

For a newly designed bridge, an additional check is required. The designer shall ensure that the results of a single-lane analysis utilizing the future searing surface are greater than 190 kips MVW.

E45-5.2.6.1 Wis-SPV Permit Rating with Multi Lane Distribution

The Maximum Permit Vehicle Load was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location is the C/L of the Pier.

The distribution width and impact factors are the same as calculated for the HS20 load.

At C/L of Pier

Flexural capacity:

$A_{st\_pier} = 1.88 \ \frac{in^2}{ft}$

$d_{s\_pier} := 28.0 - cover_{top} - \frac{8}{16}$ $\boxed{d_{s\_pier} = 25}$ in

$a\_pier := \frac{A_{st\_pier} \cdot f_y}{0.85 \cdot f'_c \cdot b}$ $\boxed{a\_pier = 2.76}$ in

$A_{smax\_pier} := \rho_b \cdot b \cdot d_{s\_pier} = 8.552$ in^2 $A_{smax} > A_{st\_pier}$ OK

$M_{n\_pier} := A_{st\_pier} \cdot f_y \cdot \left(d_{s\_pier} - \frac{a\_pier}{2}\right)$ $\boxed{M_{n\_pier} = 2664.1}$ kip-in

$\boxed{M_{n\_pier} = 222}$ kip-ft

The dead load consists of the slab self-weight and parapet weight divided evenly along the slab width:

$M_{D\_pier} := 59.2$ kip-ft (from Chapter 18 Example, Table E18.4)

From live load analysis software, the live load moment at the C/L of Pier due to the Wisconsin Standard Permit Vehicle (Wis-SPV) having a gross vehicle load of 190 kips and utilizing the maximum multi-lane distribution (at Spans 1 and 3) is:

$M_{LSPVm\_pier} := 66.06$ kip-ft

Annual Permit:

$$RF_{mpermit} := \frac{\phi \cdot M_{n\_pier} - 1.3 M_{D\_pier}}{1.3 \cdot M_{LSPVm\_pier} \cdot (1 + I_{13})} = 1.10$$

The maximum Wisconsin Standard Permit Vehicle (Wis_SPV) load is:

$RF_{mpermit} \cdot (190) = 209$ kips

E45-5.2.6.2 Wis-SPV Permit Rating with Single Lane Distribution w/o FWS

The live load moment at the C/L of Pier due to the Wis-SPV with single-lane loading may be determined by scaling the live load moment from multi-lane loading:

$$M_{LSPVs\_pier} := M_{LSPVm\_pier} \cdot \frac{E_{m13}}{E_{s13}} = 34.57 \quad \text{kip-ft}$$

Single-Trip Permit w/o FWS:

$$RF_{spermit} := \frac{\phi \cdot M_{n\_pier} - 1.3M_{D\_pier}}{1.3 \cdot M_{LSPVs\_pier} \cdot (1 + I_{13})} = 2.10$$

The Wisconsin Standard Permit Vehicle (Wis-SPV) load that can be carried by the bridge is:

$$RF_{spermit} \cdot (190) = 399 \quad \text{kips}$$

The Single-Lane MVW for the Wis-SPV is shown on the plans, up to a maximum of 250 kips. This same procedure used for the (Wis-SPV) can also be used when evaluating the bridge for an actual "Single-Trip Permit" vehicle.

E45-5.2.6.3 Wis-SPV Permit Rating with Single Lane Distribution w/ FWS

From Chapter 18 Example, Table E18.4, the applied moment at the pier from the future wearing surface is:

$$M_{DW\_pier} := 4.9 \quad \text{kip-ft}$$

Single-Trip Permit w/ FWS:

$$RF_{spermit\_fws} := \frac{\phi \cdot M_{n\_pier} - 1.3 \cdot (M_{D\_pier} + M_{DW\_pier})}{1.3 \cdot M_{LSPVs\_pier} \cdot (1 + I_{13})} = 1.99$$

The Wisconsin Standard Permit Vehicle (Wis-SPV) load that can be carried by the bridge is:

$$RF_{spermit\_fws} \cdot (190) = 379 \quad \text{kips}$$ > 190k OK

E45-5.3 Summary of Rating

Slab - Interior Strip						
Limit State	Design Load Rating		Permit Load Rating (kips)		
	Inventory	Operating	Multi DF w/o FWS	Single DF w/o FWS	Single DF w/ FWS
Flexure	HS21	HS36	209	399	379

This page intentionally left blank.