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## 2.1 Organizational Charts

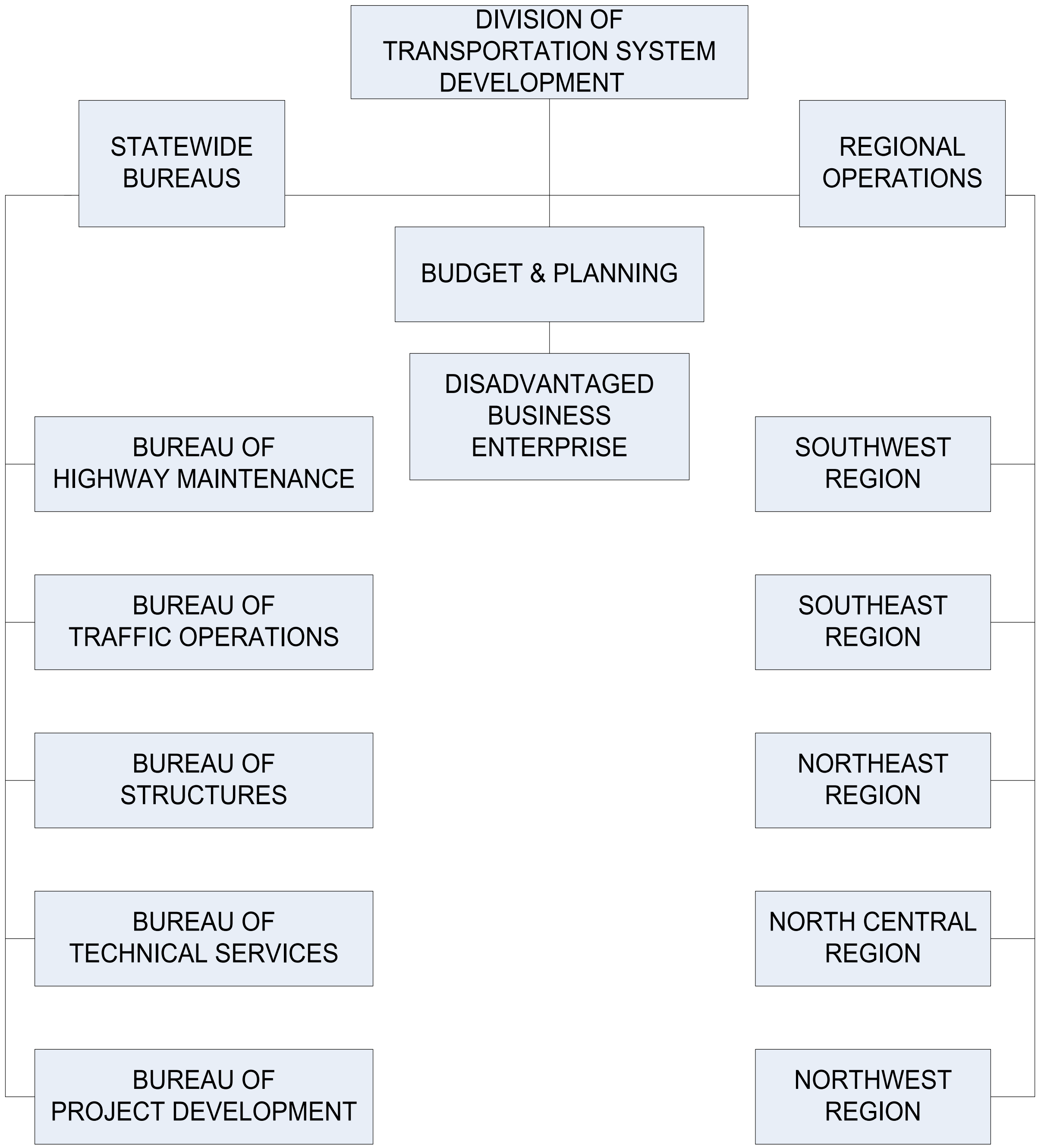



**Figure 2.1-1**
Division of Transportation System Development

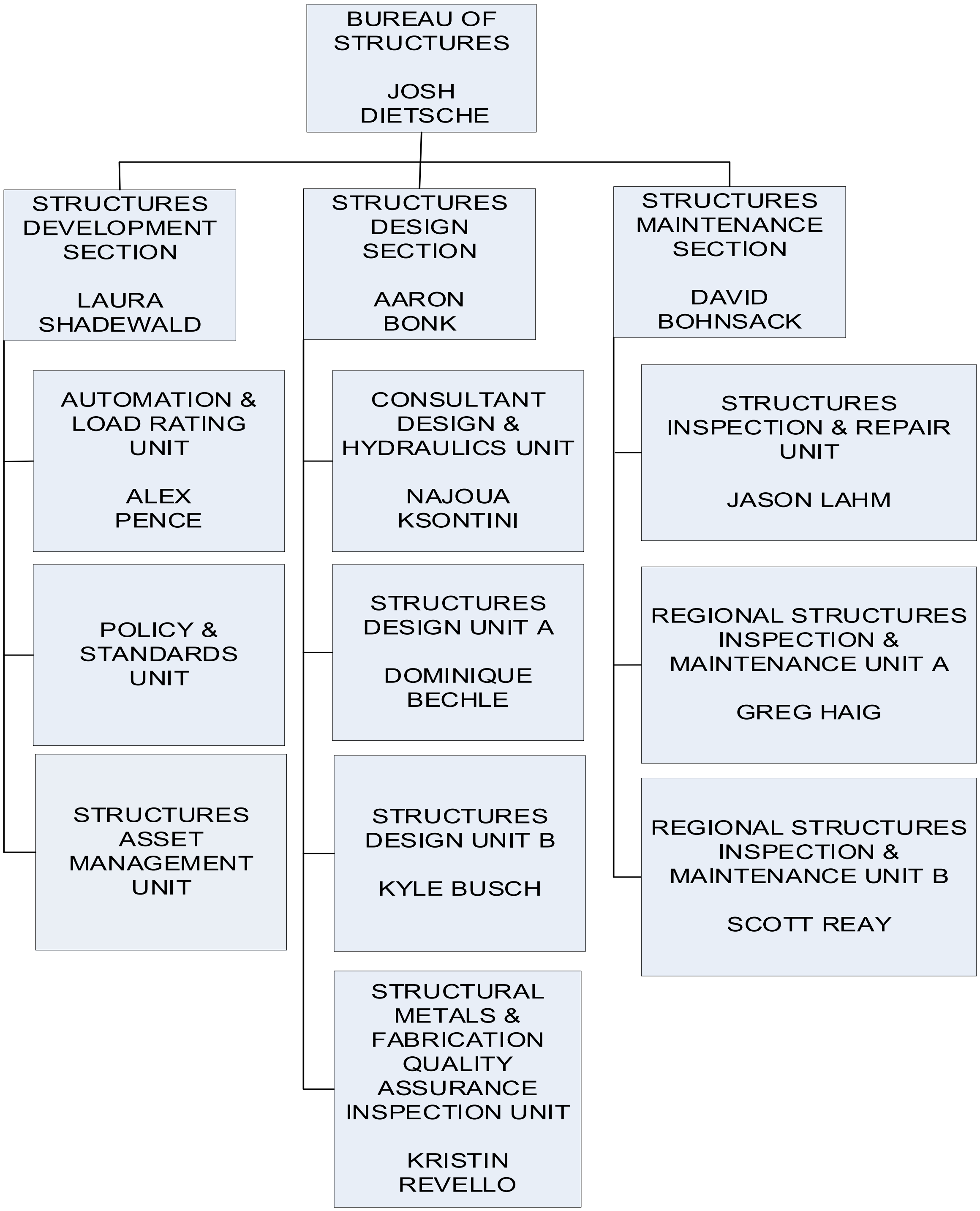
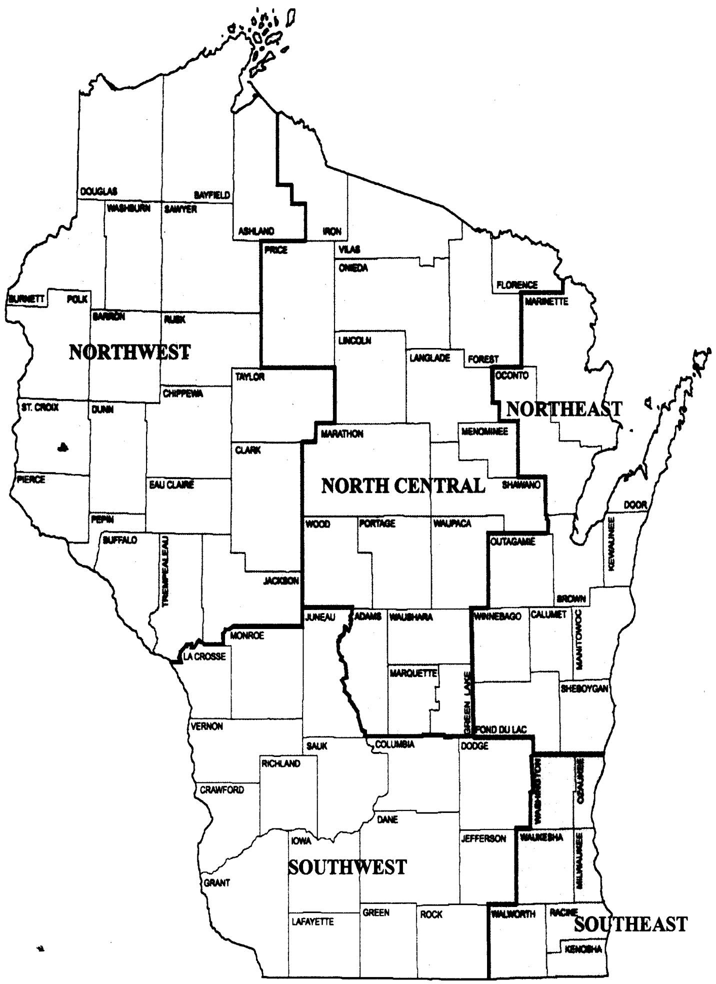



**Figure 2.1-2**
Bureau of Structures




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

**Figure 2.1-3**
Region Map

## 2.5 Structure Numbers

An official number, referred to as a structure number, is assigned to bridge structures and ancillary structures in the WisDOT right-of-way. As shown in Figure 2.5-1, structure numbers begin with a letter based on the structure type. The structure type designation is then followed by a two-digit county number, a unique four-digit structure number, and in some cases a unit number. Note: leading zeroes may be omitted from the structure number (i.e. B-5-70).

Structure numbers should be assigned to structures prior to submitting information to the Bureau of Structures for the structural design process or the plan review process. For assigning structure numbers and structure unit numbers, contact the Regional Structures Program Manager for B-Structures and the Regional Ancillary Program Manager for ancillary structures. As of 2024, the practice of assigning unit numbers to bridge structures has been discontinued. Existing bridge structures assigned unit numbers will remain in place, unless directed otherwise. Refer to the WisDOT Structures Maintenance and Inspection website for additional information.

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

The following criteria should be used when assigning structure numbers to bridge (B) and ancillary structures (C, P, S, L, R, N, or M):

- B is assigned to bridge structures (B-Structures) over 20 ft. in structure length, measured along the roadway centerline between the inside faces of abutments or exterior walls. The following should be considered when assigning structure numbers to bridges:
    - A set of nested pipes may be assigned as a bridge structure if the distance between the inside diameters of the end pipes exceeds 20 ft. and the clear distance between pipe openings is less than half the diameter of the smallest pipe.
    - Refer to the Structure Inspection Manual for measurements used to define a bridge structure.
    - Bridges on state boundary lines also have a number designated by the adjacent state. Pedestrian only bridge structures are assigned a B-Structure if they are over 20 ft in structure length <u>and</u> are state maintained, DNR bridges reviewed by WisDOT, or cross a roadway. Pedestrian boardwalks may be assigned a B-Structure when a clear span exceeds 20 ft. Other cases may be considered on a project-to-project basis.
    - A bridge number should not change except in very rare or unusual circumstances.
    - When any portion of the existing bridge is retained for rehabilitation or partially replaced, it will retain the existing bridge number.
    - A new number is assigned for a completely new bridge (i.e. do not retain the existing bridge number).

  - Assign one bridge number to any bridge with a closed median, where the area between the two roadways on the bridge is bridged over and can support traffic sharing a common substructure unit or units. Closed medians may have either mountable or non-mountable curbs or barriers. Refer to Figure 2.5-2.
  - Assign two bridge numbers to separate superstructures with an open median (not meeting the closed median criteria above) sharing a common substructure unit or units. Refer to Figure 2.5-2.
  - Separate bridge numbers be reported for each mainline bridge and the ramp that connects to the mainline bridge, when the ramp has at least one distinct abutment and is greater than 20 feet in length. Separate bridge numbers are to be assigned for a bridge that divides into two or more separate bridges, or two or more bridges that merge into one single bridge. In both cases, the separating point between bridges should be the closest deck joint, or substructure unit to the separating point, or other logical and reasonable location as determined by BOS.

- In general, C is assigned to small bridge structures (C-Structures) 20 ft. or less in structure length that have a unique structural design and/or a heightened inspection interest. This includes bridge-like structures (deck girders, flat slabs, etc.), concrete box culverts with a cross-sectional opening greater than, or equal to 20 square feet, rigid frames (three-sided concrete structures), and structural plate structures (pipes, pipe arches, box culverts, etc.). Structures not meeting the bridge structure or small bridge structure criteria are then typically considered a roadway culvert as described in Facilities Development Manual (FDM) 13-1. Buried structures listed in FDM 13-1 are typically not assigned a structure number, except for closely nested pipes and structural plate structures. Refer to the Structure Inspection Manual for additional information on small bridge structures.

- P designates structures for which there are no structural plans on file.

**WisDOT Policy Item:**

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

- S is assigned to overhead sign structures and signal monotubes. Unit numbers should be assigned to signal monotubes at an intersection with multiple structures. In this case, the base structure number should be the same for all signal monotubes and the unit numbers use to designate individual structures (i.e. S-13-1421-0001, S-13-1421-0002, etc.).

- L is assigned to high mast lighting structures. High mast light structures grouped at a location, such as an interchange or rest area, should be assigned unit numbers.

- R is assigned to permanent retaining walls. For a continuous wall consisting of various wall types, such as a secant pile wall followed by a soldier pile wall, unit numbers should be assigned to each wall type segment. Wall facing discontinuities (e.g.

stairwells, staged construction, tiers, or changes to external loads) do not require unique wall numbers if the leveling pad or footing is continuous between the completed wall segments. For soldier pile walls with anchored and non-anchored segments, unique wall numbers are not required for each segment.

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

Retaining walls whose height exceeds the below criteria require R numbers:

- Proprietary retaining walls (e.g., modular block MSE walls)
  - MSE walls having a maximum height of less than 5.5 ft. measured from the bottom of wall or top of leveling pad to top of wall are deemed to be “minor retaining walls” and do not require an R number. Refer to FDM 11-55-5.2 for more information.
  - Modular block gravity walls having a maximum height of less than 4.0 ft. measured from the bottom of wall or top of leveling pad to top of wall are deemed to be “minor retaining walls” and do not require an R number. Refer to FDM 11-55-5.2 for more information.
- Non-proprietary walls (e.g., sheet pile walls, cast-in-place walls):

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

- N is assigned to noise barriers. Unit numbers may be assigned to long barriers or complex interchanges where it is desirable to have only one structure number for the site. Unit numbers should also be used if a continuous noise barrier is supported by different structure type (e.g. ground mounted or structure mounted).

  M is assigned to miscellaneous structures where it is desirable to have a structure plan record while not meeting the above-mentioned structure assigned criteria.

Numeric code for the County where the structure is located.
Leading zeros are not shown.

R-5-70 R-40-702-2

Unique Structure Number assigned for that County by the Regional Office.

Unit Number (if used)

**Figure 2.5-1**
Structure Number Detail

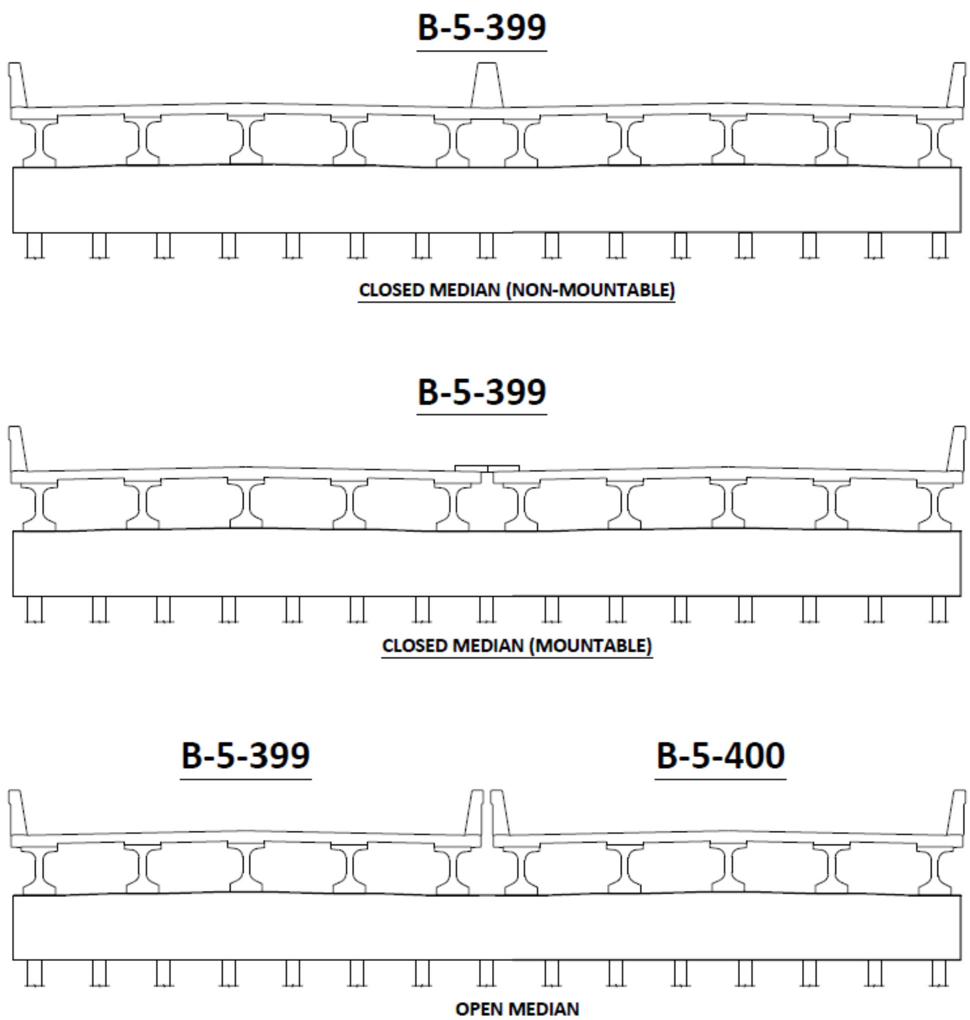



**Figure 2.5-2**
Structure Numbers for Closed and Open Medians

## 2.6 Bridge Files

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

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

| | | | |
|---|---|---|---|
| | City-Village-Town (CVT) Maps showing location of bridges. | | |
| Payment estimates to Contractors | | ----------- | BPD |
| ASTM Specifications | | Structures Development | BOS |
| Plans | | ----------- | BOS |
| As built. All plans are digitized. | | Structures Development | BOS |
| | Bridge Plans: Plans of structures designed but not yet advertised are in files. | ----------- | BOS |
| Shop Plans of Active Steel Projects | | Metals Fabrication and Inspection Unit | BOS |
| Records (Accounting) | | | |
| | Bridge Standards: Documentation for Standards and Bridge Manual | Structures Development | BOS |
| | Rainfall and Runoff Data | Structures Design | BOS |
| | Bids on Individual Items | ______________________ | BPD |
| Reports | | | |
| | Bridge Maintenance Reports | Structures Maintenance | BOS |
| | Federal Highway Experimental Project Reports | Structures Development | BOS |
| | Foundation Reports | Geotechnical Section | BTS |
| | Preliminary Reports: Contains Information necessary for Design of Structures. | ----------- | Region |
| | Research Reports | Structures Development | BOS |
| Special Provisions of Active Projects | | ----------- | BOS |
| Specifications | | Structures Development | BOS |
| | AASHTO, ACI, AWS, AREMA, AISC, CRSI, PTI, SSPC, etc. | | |
| Survey Notes | | ----------- | Region |
| Text Books on Foundations, Structures and Bridge Design | | Structures Development | BOS |

Bureau Legend:

BOS - Bureau of Structures

BPD - Bureau of Project Development

BTS - Bureau of Technical Services

## 2.7 Contracts

Contracts are administered by construction personnel in the Regional Office where the project is located. The Bureau of Project Development coordinates the activities of the Regional Offices.

The contract contains the plans, specifications, supplemental specifications where applicable and special provisions where applicable. These parts of the contract are intended to be cooperative. In the event of a discrepancy, the Standard Specifications gives the priority part to be used.

## 2.8 Special Provisions

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

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

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

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

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

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

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

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

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

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

## 2.9 Terminology

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

| | |
|---|---|
| Bridge | A structure having a span of more than 20 ft. from face to face of abutments, measured along the roadway centerline. |
| Bridge Approach | Includes the embankment materials and surface pavements that provide the transition between bridges and roadways. |
| Bushings | A lining used to reduce friction and/or insulate mating surfaces usually on steel hanger plate bearings. |
| Butt Splice | A splice where the ends of two adjoining pieces of metal in the same plane are fastened together by welding. |
| CADDS | Computer Aided Design and Drafting System. |
| Caisson | A watertight box of wood or steel sheeting; or a cylinder of steel and concrete, used for the purpose of making an excavation. Caissons may be either open (open to free air) or pneumatic (under compressed air). |
| Camber | A slight vertical curvature built into a structural member to allow for deflection and/or vertical grade. |
| Cathodic Protection | A method of protecting steel in concrete by impressing direct current via anodes thus making the bar steel cathodically protected. |
| Causeway | A raised road across wet or marshy ground or across water. |
| Change Order | A written order to the Contractor, signed by the Engineer, ordering a change in the work from that originally shown by the Plans and Specifications that has been found necessary. If the work is of a nature involving an adjustment or unit price, a Supplemental Agreement shall be executed. Change orders duly signed and executed by the Contractor constitute authorized modifications of the Contract. |
| City and Village Streets | City and Village streets are the public thoroughfares within the boundaries of incorporated municipalities. They are improved and maintained under the jurisdiction of the respective city and village authorities that constitute the local governing bodies. A few city and village streets are eligible for federal aid. |
| Cofferdam | A barrier built in the water so as to form an enclosure from which the water is pumped to permit free access to the area within. |
| Composite Section | Two sections made of the same or different materials together to act as one integral section; such as a concrete slab on a steel or prestressed girder. |
| Compression Seals | A preformed, compartmented, elastomeric (neoprene) device, which is capable of constantly maintaining a compressive force against the joint interfaces in which it is inserted. |
| Concrete Overlay | 1 1/2” to 2” of concrete placed on top of the deck, used to extend the life of the deck and provide a good riding surface. |
| Construction Limits | The Stations at which construction begins and ends. |
| Contract Time | The number of calendar days shown in the proposal which is allowed for completion of the work. |

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|---|---|
| Contraction Joint | A joint in concrete that does not provide for expansion but allows for contraction or shrinkage by the opening up of a crack or joint. |
| Coordinates | Linear or angular dimensions designating the position of a point in relation to a given reference frame. In Wisconsin it refers to the State Plane Coordinate System. |
| County Trunk Highway System | The County Trunk Highway System, established in 1925, which forms the secondary system of highways within the State, constitutes the interconnecting highways of the State Trunk System, and is made up mainly of highways secondary in traffic importance. It consists generally of highways of local service and is improved and maintained by the 72 county boards, which constitute the local governing authorities. Many county trunks are eligible for federal aid. |
| Creep | Time dependent inelastic deformation under elastic loading of concrete or steel resulting solely from the presence of stress. |
| Cross Bracing | Bracing used between stringers and girders to hold them in place and stiffen the structure. |
| Culvert | A structure not classified as a bridge having a span of 20 ft. or less spanning a watercourse or other opening on a public highway. |
| Curb | A vertical or sloping member along the edge of a pavement or shoulder forming part of a gutter, strengthening or protecting the edge, and clearly defining the edge of vehicle operators. The surface of the curb facing the general direction of the pavement is called the "face". |
| Cut-Off-Wall | A wall built at the end of a culvert apron to prevent the undermining of the apron. |
| Dead Load | The weight of the materials used to build the structure including parapets, utilities and future wearing surface on deck. |
| Deadman | A concrete mass, buried in the earth behind a structure, that is used as an anchor for a rod or cable to resist horizontal forces that act on the structure. |
| Deck Structure | A structure that has its floor resting on top of all the main stress carrying members. |
| Deflection Joint | A joint placed in the parapets of bridges to prevent cracking of the parapet due to deflection of the superstructures. |
| Design Volume | A volume determined for use in design representing traffic expected to use the highway. Unless otherwise stated, it is an hourly volume. |
| DHV | Design hourly volume. |
| Diaphragm | A structural member used to tie adjoining girders together and stiffen them in a lateral direction as well as distribute loads. |
| Dolphins | A group of piles or sheet piling driven adjacent to a pier. Their purpose is to prevent extensive damage or possible collapse of a pier from a collision with a ship or barge. |

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|---|---|
| Draped Strands | Strand pattern for prestressing strands, where strands are draped at the ends of the girder to decrease the prestressing stress where the applied moments are small. |
| Drift Pin | A metal pin, tapered at both ends, used to draw members of a steel structure together by being driven through the corresponding bolt holes. |
| Drip Groove | A groove formed into the underside of a projecting sill or coping to prevent water from following around the projection and reaching the face of the wall. |
| Dummy Joint | A groove in the surface of a concrete structure that resembles a joint but does not go all the way through. It provides a plane of weakness, and is used to ensure that any cracks that occur will be in a straight line. |
| Epoxy Coated Rebar | Bar steel reinforcement coated with a powdered epoxy resin to prevent corrosion of the bar steel. |
| Expansion Bearings | Bearings that allow longitudinal movement of the superstructure relative to the substructure and rotation of the superstructure relative to the substructure. |
| Expansion Device | A device placed at expansion points in bridge superstructures to carry the vertical bridge loads without preventing longitudinal movement. |
| Expansion Joint | An expansion device in concrete that allows expansion due to temperature changes, thereby preventing damage to the slabs. |
| Filler Plate | A steel plate or shim used to filling in space between compression members. |
| Fixed Bearings | Bearings that do not provide for any longitudinal movement of the superstructure relative to the substructure, but allows for rotation of the superstructure relative to the substructure. |
| Flat Slab | A reinforced concrete superstructure that has a uniform depth throughout. |
| Floor Beam | A transverse structural member that extends from truss to truss or from girder to girder across the bridge. |
| Fracture Critical Members | Steel tension members or steel tension components of members whose failure would probably cause a portion of or the entire bridge to collapse. |
| Fracture Mechanics | Study of crack growth in materials. |
| GVW | Gross vehicle weight which is the total weight of basic truck, body and related payload. |
| Geotextiles | Sheets of woven or nonwoven synthetic polymers or nylon used for drainage and soil stabilization. |
| Girder | Main longitudinal load carrying member in a structure. |
| Grade Separation | A crossing of two highways, or a highway and a railroad, at different levels. |
| Grid Floors | Prefabricated steel grids set on girders and/or stringers provide the roadway surface, generally on moveable highway structures. |

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|---|---|
| Hammerhead Pier | A pier which has only one column with a cantilever cap and is somewhat similar to the shape of a hammer. |
| Hanger Plate | A steel plate which connects the pins at hinge points thus transmitting the load through the hinge. |
| Haunch | An increase in depth of a structural member usually at points of intermediate support. |
| Haunched Slab | A reinforced concrete superstructure that is haunched (has an increased depth) at the intermediate supports. |
| Hinge | A device used to hold the ends of two adjoining girders together, but allowing for longitudinal movement of the superstructure. |
| Hinged Bearing | At hinge location along a girder, where forces from supported member are transferred to supporting member by a bearing (See Std. 24.8). |
| Hold down Device | A device used on bridge abutments to prevent girders from lifting off their bearings as a result of the passage of live load over the bridge. |
| Hybrid Girder | A steel plate girder with the web steel having a lower yield strength than the steel in one or both flanges. |
| Inlet Control | The case where the discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater and the entrance geometry, including barrel shape, cross sectional area, and inlet edge. |
| Intermediate Stiffener | A vertical transverse steel member used to stiffen the webs of plate girders between points of supports. |
| Jetting | Forcing water into holes in an embankment to settle or compact the earth. |
| Laminated Elastomeric Bearing | A bearing device constructed from elastomer layers restraining at their interfaces by integrally bonded steel or fabric reinforcement. Its purpose is to transmit loads and accommodate movements between a bridge and its supporting structure. |
| Lateral Bracing | Bracing placed in a horizontal plane between steel girders near the bottom and/or top flanges. |
| Leads | The vertical members of a pile driver that steady the hammer and pile during the driving. |
| Liquid Penetrant Inspection | Nondestructive testing method that reveals surface discontinuities by the bleedout of a penetrating medium against a contrasting colored background. |
| Live Load | For highway structures AASHTO truck or lane loadings. The weight of moving loads. |
| LRFD | Load Resistance Factor Design. |
| Longitudinal Stiffener | A longitudinal steel plate (parallel to girder flanges) used to stiffen the webs of welded plate girders. |
| Low Relaxation Strands | Prestressing tendons which are manufactured by subjecting the strands to heat treatment and tensioning causing a permanent elongation. This increases the strand yield strength and reduces strand relaxation under constant tensile stress. |

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|---|---|
| Low Slump Concrete | Grade "E" concrete, used for concrete masonry overlays and repairs on decks. |
| Mag Particle Inspection | Nondestructive testing method that is used primarily to discover surface discontinuities in ferro magnetic materials by applying dry magnetic particles to a weld area or surface area that has been suitably magnetized. |
| Modular Exp. Joints | Multiple, watertight units placed on structures requiring expansion movements greater than 4”. |
| Mud Sill | A timber platform laid on earth as a support for vertical members or bridge falsework. |
| NCHRP | National Cooperative Highway Research Program. |
| Negative Moment | The moment causing tension in the top fibers and compression in the bottom fibers of a structural member. |
| Negative Reinforcement | Reinforcement placed in concrete to resist negative bending moments. |
| Non-Redundant Structure | Type of structure with single load path, where a single fracture in a member can lead to the collapse of the structure. |
| Oil Well Pipe Pile | High quality pipe used in oil industry drilling operations that may be used as an alternate to HP piling. |
| Outlet Control | The case where the discharge capacity of a culvert is controlled by the elevation of the tailwater in the outlet channel and the slope, roughness, and length of the culvert barrel, in addition to the cross sectional area and inlet geometrics. |
| P S & E | Literally plans, specifications, and estimates. Usually it refers to the time when the plans, specifications, and estimates on a project have been completed and referred to FHWA for approval. When the P S & E have been approved, the project goes from the preliminary engineering phase to the construction phase. |
| Parapet | A masonry barrier designed and placed to protect traffic from falling over the edge of a bridge, or in some cases, from crossing lanes of traffic traveling in opposite directions. |
| Pier | Intermediate substructure unit of a bridge. |
| Pile | A long, slender piece of wood, concrete, or metal to be driven or jetted into the earth or river bed to serve as a support or protection. |
| Pile Bent | A pier where the piles are extended to the pier cap to support the structure. |
| Pile Cap | A slab, usually of reinforced concrete, covering the tops of a group of piles for the purpose of tying them together and transmitting to them as a group the load of the structure which they are to carry. |
| Pile Foot | The lower extremity of a pile. |
| Pile Head | The top of a pile. |
| Pile Points | Metal tip fastened to the lower end of pile to protect it when the driving is hard. |
| Pin Plate | A steel plate attached to the web plate of girders at hinge points to strengthen the web plate of girders at the hinge locations. |

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| Positive Moment | The moment causing compression in the top fibers and tension in the bottom fibers in a structural member. |
| Post-Tensioned | Method of prestressing in which the tendon is tensioned after the concrete has cured. |
| Prestress Camber | The deflection in prestressed girders (usually upward) due to the application of the prestressing force. |
| Prestressed Concrete | Concrete in which there have been introduced internal stresses of such magnitude and distribution that the stresses resulting from given external loadings are counteracted to a desired degree. In reinforced-concrete members the prestress is commonly introduced by tensioning the steel reinforcement. |
| Pretensioned | Any method of prestressing in which the strands are tensioned before the concrete is placed. |
| Radiographic Inspection | Nondestructive testing method where gamma rays or X rays pass thru the object and cast an image of the internal structure onto a sheet of film as the result of density changes. |
| Redundant Structure | Type of structure with multi-load paths where a single fracture in a member cannot lead to the collapse of the structure. |
| Reflection Crack | A crack appearing in a resurface or overlay caused by movement at joints or cracks in underlying base or surface. |
| Residual Camber | Camber due to the prestressing force minus the deadload deflection of the girder. |
| RIPRAP | A facing of stone used to prevent erosion. It is usually dumped into place, but is occasionally placed by hand. |
| Rolled Girder Structure | A structure which has a rolled steel beam as the main stress carrying member. |
| Roughometer | A wheeled instrument used for testing riding qualities or road surfaces. |
| S.S.P.C. | Steel Structures Painting Council. |
| Semi-Retaining Abutment | An abutment used for retaining part of the back-fill of the roadway as well as supporting the end of the bridge. |
| Semi-Through Structure | A structure that has no overhead bracing, but the main stress carrying members project above the floor level. |
| Shear Connector | A connector used to join cast-in-place concrete to a steel section and to resist the shear at the connection. |
| Sheet Pile | A pile made of flat or arched cross section to be driven into the ground and meshed or interlocked with like members to form a wall, or bulkhead. |
| Shoulders | The portions of the roadway between the traveled way and the inside edges of slopes of ditches or fills, exclusive of auxiliary lanes, curbs, and gutters. |
| Shrinkage | Contraction of concrete due to drying and chemical changes, dependent on time. |
| Sill Abutment | A shallow concrete masonry abutment generally about 5 feet deep. |
| Simple Spans | Spans with the main stress carrying members non-continuous, or broken, at the intermediate supports. |

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| Skew or Skew Angle | The acute angle formed by the intersection of a line normal to the centerline of the roadway with a line parallel to the face of the abutments or piers, or in the case of culverts with the centerline of the culverts. Left hand forward skew indicates that, look up station, the left side of the structure is further up station than the right hand side. Right hand skew indicates that the right side of structure is further up station than the left side. |
| Slope | The degree of inclination to the horizontal. Usually expressed as a ratio, such as 1:25 or 1 of 25, indicating 1 unit rise in 25 units of horizontal distance. |
| Slope Paving | Paving placed on the slope in front of abutment under a bridge to prevent soil erosion and sliding. |
| Spandrel | The area between the roadway and the arch in the side view of an arch bridge. |
| Special Provisions | Special directions and requirements that are prepared for the project under consideration and made a part of the contract. |
| Specifications | The body of directions, provisions, and requirements contained herein, together with written agreements and all documents of any description, made or to be made, pertaining to the method or manner of performing the work, the quantities, and the quality of materials to be furnished under the contract. |
| Spread Footing | A footing that is supported directly by soil or rock. |
| Spur Dike | A wall or mound built or extended out from the upstream side of an abutment used to train the stream flow to prevent erosion of stream bank. May also be used where there is no bridge, but the stream flows along the side of highway embankment. |
| Stainless Steel Teflon Bearings | Incorporates stainless steel and Teflon with steel to provide the necessary expansion movement. |
| State Plane Coordinates | The plane-rectangular coordinate system established by the United States Coast and Geodetic Survey. The plane coordinate system in Wisconsin is based on the Lambert conformal conic projection. Plane coordinates are used to locate geographic position. |
| State Trunk Highway Network | The system of highways heretofore selected and laid out by the Legislature and special legislative committees and by the Commission, and as revised, altered and changed by the Commission, including temporary routes designated by the Commission, the portions of the Interstate Highway System within the state, and routes adopted by the American Association of State Highway Officials as part of the U.S. Numbered Route System. |
| Stirrup | Vertical U-shaped or rectangular shaped bars placed in concrete beams to resist the shearing stresses in the beam. |
| Strip Seal Joint | Molded neoprene glands inserted and mechanically locked between armored interfaces of extruded steel sections. |
| Substructure | All of that part of the structure below the bridge seats or below the skewbacks of arches, or below the tops of the caps of piling or |

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| | framed trestles, except that the wingwalls and parapets of abutments shall be considered as part of the substructure. |
| Superstructure | That part of the structure above the bridge seats, or above the skewbacks of arches, or above the tops of the caps of piling or framed trestles, including the flooring, but excluding wing walls and parapets of abutments (See substructures). |
| Supplemental Specifications | Specifications adopted subsequent to the publication of these specifications. They generally involve new construction items or substantial changes in the approved specifications. Supplemental specifications prevail over those published whenever in conflict therewith. |
| Surcharge | Any load that causes thrust on a retaining wall, other than backfill to the level of the top of the wall. |
| TRB | Transportation Research Board. |
| Temporary Hold down Device | A device used at the ends of steel bridges where the slab pour terminates to prevent the girders from lifting off the abutment bridge seats during the pouring of the concrete deck. |
| Tendon | A name for prestressed reinforcing element whether wires, bars, or strands. |
| Through Structure | A structure that has its floor connected to the lower portion of the main stress-carrying members, so that the bracing goes over the traffic. A structure whose main supporting members project above the deck or surface. |
| Tining | Used on finished concrete deck or slab surfaces to provide friction and reduce hydroplaning. |
| Town Road System | The town road system, or tertiary system of highways within the state, has been improved or maintained under the jurisdiction of the town boards, which are the local governing bodies. Some of the town roads are eligible for federal aid. |
| Transfer Stresses | In pretensioned prestressed concrete members the stresses that take place at the release of prestress from the bulkheads. |
| Ultrasonic Inspection | A non-destructive inspection process where by an ultra-high frequency sound wave induced into a material is picked up in reflection from any interface or boundary. |
| Unbonded Strands | Strands so coated as to prevent their forming a bond with surrounding concrete. Used to reduce stress at the ends of a member. |
| Underpinning | The adding of new permanent support to existing foundations, to provide either additional capacity or additional depth. |
| Uplift | A force tending to raise a structure or part of a structure and usually caused by wind and/or eccentric loads, or the passage of live-load over the structure. |
| Waterproofing Members | Impervious asphaltic sheets overlaid with bituminous concrete to protect decks from the infiltration of chlorides and resulting deterioration. |

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| Wearing Surface | The top layer of a pavement designed to provide a surface resistant to traffic abrasion. |
| Weep Hole | A drain hole through a wall to prevent the building up of hydraulic pressure behind the wall. |
| Weir | A dam across a stream for diverting or measuring the flow. |
| Weld Inspection | Covers the process, written procedure, and welding in process. Post weld heat maintenance if required, post weld visual inspection and non-destructive testing as specified in contract and Standard Specifications. |
| Welded Wire Fabric | A two-way reinforcement system, fabricated from cold-drawn steel wire, having parallel longitudinal wires welded at regular intervals to parallel transverse wires and conforming to "Specifications for Welded Steel Wire Fabric for Concrete Reinforcement", AASHTO. |
| Well-Graded | An aggregate possessing proportionate distribution of successive particle sizes. |
| Wingwall | A wall attached to the abutments of bridges or box culverts retaining the backfill of the roadway. The sloping retaining walls on each side of the center part of a bridge abutment. |

**Table 2.9-1**
Terminology

## 2.10 WisDOT Bridge History

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

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

2.10.1 Unique Structures

| Structure<br><br>Type | Bridge<br><br>Number | Year<br><br>Constructed | (feet)<br><br>Span Configuration |
|---|---|---|---|
| Steel Rigid Frames | B-40-48 - Milwaukee | 1959 | 45.3, 168.5, 46.3 |
| Steel Rigid Frames | B-56-47/48* - Mirror Lake | 1961 | 50.6, 22.0, 49.4 |
| | | | |
| Overhead Timber Truss | B-22-50* - Cassville | 1962 | 48.0 |
| | | | |
| Arch Truss | B-16-5 - Superior | 1961 | 270.0, 600.1, 270.0 |
| | | | |
| Tied Arches | B-9-87* - Cornell | 1971 | 485.0 |
| Tied Arches | B-12-27* - Prairie du Chien | 1974 | 462.0 |
| Tied Arches | B-40-400 - Milwaukee | 1974 | 270.0, 600.0, 270.0 |
| Tied Arches | B-5-158* - Green Bay | 1980 | 450.1 |
| Tied Arches | B-22-60 - Dubuque, IA | 1982 | 670.0 |
| Tied Arches | B-16-38* - Superior | 1984 | 500.0 |
| | | | |
| Prestressed “I” Girders with Cantilever | B-40-524* - Milwaukee | 1985 | 112.0, 69.0, 107.8, 383.5<br>Spans with 25’ Cantilevers |
| | | | |
| Prestressed Strutted Arches | B-40-603 - Milwaukee | 1992 | 8-158.0 Strutted Arch Spans |
| | | | |
| Tied Arches | B-32-202* - La Crosse | 2004 | 475.0 |

**Table 2.10-1**
Unique Structures

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

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## 5.1 Factors Governing Bridge Costs

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

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

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

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

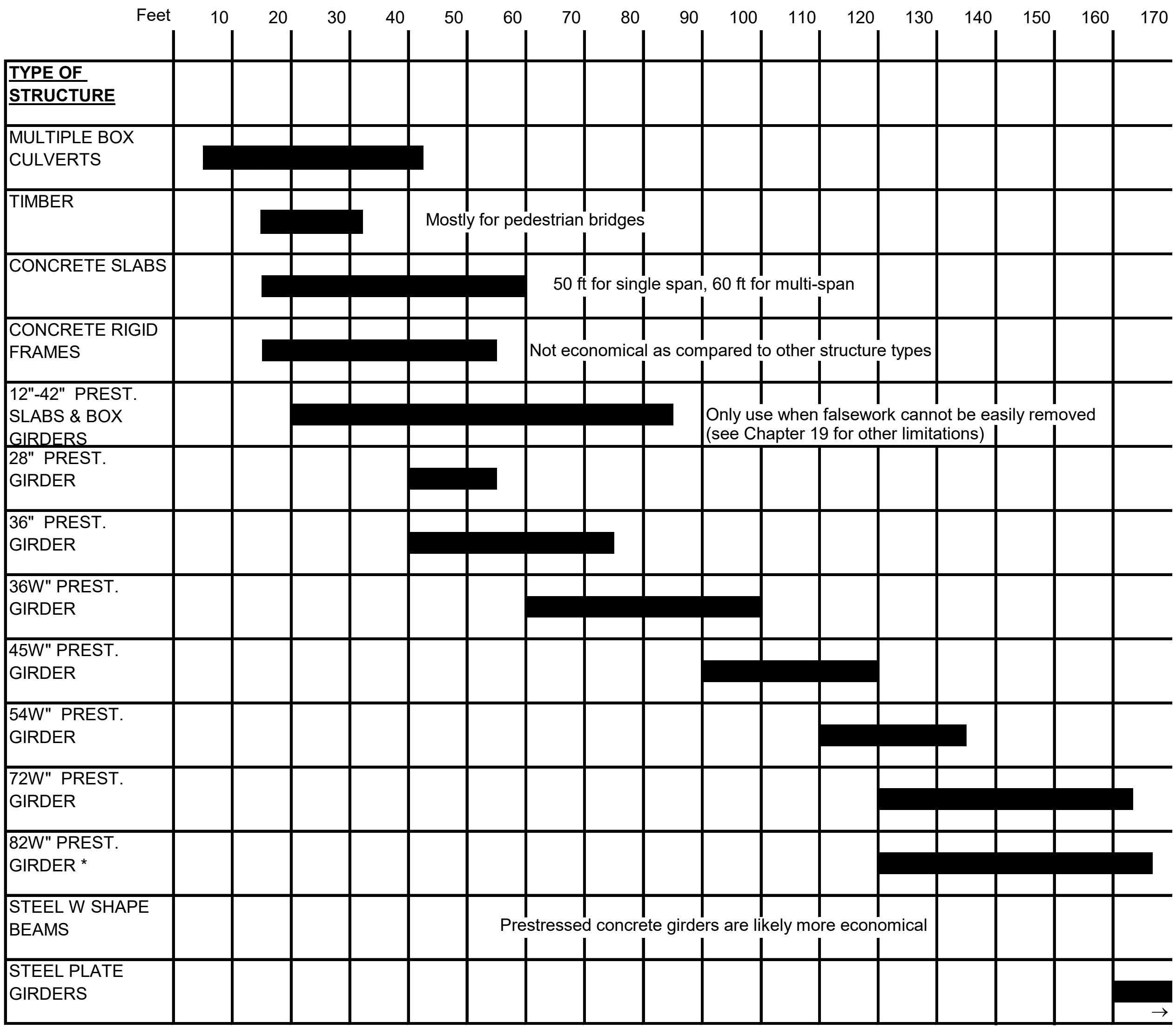
In these reports prestressed girder costs are grouped together because there is a small cost difference between girder sizes. Refer to unit costs. Concrete slab costs are also grouped together for this reason.

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

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

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

## 5.2 Economic Span Lengths




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

**Figure 5.2-1**
Economic Span Lengths

## 5.3 Contract Unit Bid Prices

Refer to FDM 19-5-5 when preparing construction estimates and use the following estimating tools:

- Bid Express
- AASHTOWare Project Estimator
- Estimating Tools website

## 5.4 Bid Letting Cost Data

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

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

### 5.4.1 2020 Year End Structure Costs

| Structure Type | No. of Bridges | Total Area (Sq. Ft.) | Total Costs | Super. Only Cost Per Square Foot | Cost per Square Foot |
|---|---|---|---|---|---|
| Prestressed Concrete Girders | 28 | 236,564 | 35,597,272 | 70.46 | 150.48 |
| Reinf. Conc. Slabs (Flat) | 35 | 57,402 | 10,783,692 | 72.40 | 187.86 |
| Reinf. Conc. Slabs (Haunched) | 7 | 53,236 | 6,866,154 | 65.48 | 128.98 |
| Prestressed Box Girder | 2 | 9,050 | 2,694,672 | 157.15 | 297.75 |
| Steel Plate Girders | 1 | 19,076 | 5,258,732 | 120.51 | 275.67 |

**Table 5.4-1**
Stream Crossing Structures

| Structure Type | No. of Bridges | Total Area (Sq. Ft.) | Total Costs | Super. Only Cost Per Square Foot | Cost per Square Foot |
|---|---|---|---|---|---|
| Prestressed Concrete Girders | 22 | 211,991 | 34,051,252 | 71.64 | 160.63 |
| Reinf. Conc. Slabs (Flat) | 1 | 2,179 | 379,028 | 62.35 | 173.95 |
| Reinf. Conc. Slabs (Haunched) | 1 | 5,563 | 870,732 | 43.94 | 156.52 |

**Table 5.4-2**
Grade Separation Structures

| Box Culvert Type | No. of Culverts | Cost per Lin. Ft. |
|---|---|---|
| Single Cell | 17 | 1,708 |
| Twin Cell | 1 | 2,073 |
| Three Cell | 0 | -- |

**Table 5.4-3**
Box Culverts

5.4.2 2021 Year End Structure Costs

| Structure Type | No. of Bridges | Total Area (Sq. Ft.) | Total Costs | Super. Only Cost Per Square Foot | Cost per Square Foot |
|---|---|---|---|---|---|
| Prestressed Concrete Girders | 29 | 220,753 | 35,044,116 | 71.47 | 158.75 |
| Reinf. Conc. Slabs (Flat) | 51 | 76,036 | 15,497,984 | 76.94 | 203.82 |
| Reinf. Conc. Slabs (Haunched) | 10 | 46,682 | 7,340,768 | 70.37 | 157.25 |
| Prestressed Box Girder | 0 | -- | -- | -- | -- |
| Buried Slabs | 2 | 5,419 | 1,256,806 | 72.16 | 231.93 |
| Steel Plate Girders | 0 | -- | | -- | -- |

**Table 5.4-4**
Stream Crossing Structures

| Structure Type | No. of Bridges | Total Area (Sq. Ft.) | Total Costs | Super. Only Cost Per Square Foot | Cost per Square Foot |
|---|---|---|---|---|---|
| Prestressed Concrete Girders | 29 | 244,240 | 37,780,465 | 73.38 | 154.69 |
| Reinf. Conc. Slabs (Flat) | 0 | -- | -- | -- | -- |
| Reinf. Conc. Slabs (Haunched) | 0 | -- | -- | -- | -- |

**Table 5.4-5**
Grade Separation Structures

5.4.3 2022 Year End Structure Costs

| Structure Type | No. of Bridges | Total Area (Sq. Ft.) | Total Costs | Super. Only Cost Per Square Foot | Cost per Square Foot |
|---|---|---|---|---|---|
| Prestressed Concrete Girders | 29 | 134,583 | 25,559,025 | 88.73 | 189.91 |
| Reinf. Conc. Slabs (Flat) | 53 | 79,248 | 17,397,862 | 85.21 | 219.54 |
| Reinf. Conc. Slabs (Haunched) | 6 | 49,138 | 9,413,541 | 88.63 | 191.57 |
| Prestressed Box Girder | 0 | -- | -- | -- | -- |
| Buried Slabs | 0 | -- | -- | -- | -- |
| Steel Plate Girders | 0 | -- | -- | -- | -- |

**Table 5.4-6**
Stream Crossing Structures

| Structure Type | No. of Bridges | Total Area (Sq. Ft.) | Total Costs | Super. Only Cost Per Square Foot | Cost per Square Foot |
|---|---|---|---|---|---|
| Prestressed Concrete Girders | 8 | 81,829 | 13,443,218 | 78.36 | 164.28 |
| Reinf. Conc. Slabs (Flat) | 0 | -- | -- | -- | -- |
| Reinf. Conc. Slabs (Haunched) | 0 | -- | -- | -- | -- |

**Table 5.4-7**
Grade Separation Structures

5.4.4 2023 Year End Structure Costs

| Structure Type | No. of Bridges | Total Area (Sq. Ft.) | Total Costs | Super. Only Cost Per Square Foot | Cost per Square Foot |
|---|---|---|---|---|---|
| Prestressed Concrete Girders | 12 | 70,546 | 13,054,625 | 93.75 | 185.05 |
| Reinf. Conc. Slabs (Flat) | 36 | 67,796 | 15,075,049 | 86.82 | 222.36 |
| Reinf. Conc. Slabs (Haunched) | 4 | 13,032 | 3,208,985 | 79.85 | 246.24 |
| Prestressed Box Girder | 1 | 1,374 | 482,870 | 210.74 | 351.43 |
| Buried Slabs | 1 | 1,446 | 199,089 | 50.84 | 137.68 |
| Steel Plate Girders | 0 | -- | -- | -- | -- |

**Table 5.4-8**
Stream Crossing Structures

| Structure Type | No. of Bridges | Total Area (Sq. Ft.) | Total Costs | Super. Only Cost Per Square Foot | Cost per Square Foot |
|---|---|---|---|---|---|
| Prestressed Concrete Girders | 6 | 78,611 | 12,600,448 | 72.67 | 160.29 |
| Reinf. Conc. Slabs (Flat) | 0 | -- | -- | -- | -- |
| Reinf. Conc. Slabs (Haunched) | 4 | 27,603 | 7,188,282 | 73.19 | 260.42 |

**Table 5.4-9**
Grade Separation Structures

## 5.4.5 2024 Year End Structure Costs

| Structure Type | No. of Bridges | Total Area (Sq. Ft.) | Total Costs | Super. Only Cost Per Square Foot | Cost per Square Foot |
|---|---|---|---|---|---|
| Prestressed Concrete Girders | 12 | 43,125 | 7,753,194 | 84.68 | 179.78 |
| Reinf. Conc. Slabs (Flat) | 73 | 116,017 | 24,143,428 | 86.69 | 227.97 |
| Reinf. Conc. Slabs (Haunched) | 5 | 39,031 | 5,700,285 | 81.54 | 146.05 |
| Prestressed Box Girder | 1 | 1,401 | 526,521 | 118.21 | 375.82 |
| Buried Slabs | 1 | 2,897 | 743,006 | 105.07 | 256.47 |
| Steel Plate Girders | 0 | -- | -- | -- | -- |

**Table 5.4-10**
Stream Crossing Structures

| Structure Type | No. of Bridges | Total Area (Sq. Ft.) | Total Costs | Super. Only Cost Per Square Foot | Cost per Square Foot |
|---|---|---|---|---|---|
| Prestressed Concrete Girders | 15 | 174,056 | 13,572,036 | 77.98 | 178.23 |
| Reinf. Conc. Slabs (Flat) | 0 | -- | -- | -- | -- |
| Reinf. Conc. Slabs (Haunched) | 0 | -- | -- | -- | -- |

**Table 5.4-11**
Grade Separation Structures

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damaged girder replacements should use the sheet border for a new structure. See Chapter 40 - Bridge Rehabilitation for criteria as to when superstructure replacements are allowed.

1. Plan View

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

   a. Sufficient dimensions to layout structure in the field.

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

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

   On Structure Replacements

   Show existing structure in dashed-lines on Plan View.

2. Elevation View

   Same requirements as specified for preliminary plan except:

   a. Show elevation at bottom of all substructure units.

   b. Give estimated pile lengths where used.

3. Cross-Section View

   Same requirements as specified for preliminary plan except:

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

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

4. Grade Line

   Same requirements as specified for preliminary plan.

5. Design and Traffic Data

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

6. Hydraulic Information, if Applicable

7. Foundations

   Give soil/rock bearing capacity or pile capacity.

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

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

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

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

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

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

8. Estimated Quantities

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

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

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

9. General Notes

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

10. List of Drawings

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

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 all S.T.H., U.S.H., and I.H. bridges. Provide a paving notch at each end of all structures for all C.T.H. with bridges with concrete approaches. Paving notches optional elsewhere. See standards 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. For continuous girders, show and dimension tension zones on top and bottom flanges. Show the tension zones on replacement and

rehabilitation projects as applicable, including on deck replacements projects, widenings, and overlay projects with substantial full-depth deck repair areas. See Chapter 24 – Steel Girder Structures for additional information on tension zones. 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.

Waterway Remove Debris (structure)"; "Removing Structure Over Waterway Minimal Debris (structure)"; and "Removing Structure Over Waterway Debris Capture (structure)". If these four Standard Specification bid items do not encapsulate site specific constraints for specialized cases, which should be a rare occurrence, the designer can utilize special provisions to augment the standard spec removal items.

The designer should review all of these Standard Specifications and coordinate with the Wisconsin Department of Natural Resources (DNR) to determine which bid items to use when removing a particular structure. **If the designer disagrees with the recommendation from the DNR's Initial Review Letter (IRL), the designer should work with WisDOT Regional Environmental Coordinator (REC), WisDOT Regional Stormwater & Erosion Control Engineer (SWECE) and DNR Transportation Liaison (TL) to come to a consensus on the appropriate bid item, considering constructability and cost impacts of the items.** For unique or difficult removals, designers should consult with the contracting community to assess costs and the feasibility of a particular removal technique. One of the following Removing Structure bid items should be selected for removals over waterways:

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

- Removing Structure Over Waterway <u>Minimal Debris (structure)</u> is used where it is possible to remove the structure with only minimal debris dropping into a waterway or wetland, and that waterway or wetland is not highly environmentally sensitive. This bid item is typically appropriate for removing all structure types except for the following bridges which are typically covered under Removing Structure Over Waterway Remove Debris (structure): slab spans; voided slabs; cast-in-place concrete girder bridges; earth-filled bridges; large trestle bridges. This bid item will likely be used for most stream crossing removals. The designer may need to expand the standard spec with special provision language to address additional DNR concerns and/or issues. CMM 645.6 contains example removal and clean-up methods corresponding to this bid item.

- Removing Structure Over Waterway <u>Debris Capture (structure)</u> is typically used when resources are present such that additional protection is required due to the waterway or wetland being highly environmentally sensitive. Before including this bid item in the contract, consult with the DNR and the department's regional environmental coordinator, as well as BOS, to determine if this bid item is appropriate. The designer may need to expand the standard spec with special provision language to address pier or abutment removal, and other project specific details.

Debris Containment bid items are used where structure removal, reconstruction, or other construction operations may generate falling debris that might pose a safety hazard or environmental/contamination concern to facilities located under the structure. Two standard spec bid items for debris containment are available for use depending on the project location. For grade separation structures, the "Debris Containment (structure)" item is utilized. This item is most typically used where the removal area is located over a railroad, but may also be used over roadways, bike paths, pedestrian ways, or other facilities that will not be closed during removal operations. See 6.3.3.8.3 for additional information related to False Decking and project specific criteria for when language is to be included in contracts.

The “Debris Containment Over Waterway (structure)” item is not used when one of the three Removing Structure Over Waterway standard spec bid items is used. This item may be used for structure repair projects occur over waterways where full removals are not involved. One example of this is a standalone joint replacement project at a stream crossing structure.

#### 6.3.3.8.1 Structure Repairs

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

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

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

- For work **over waterways**, a method of protecting the waterway is needed in some cases. Use Debris Containment over Waterway (structure), **only as needed** based on the extent and location of removal, and environmental sensitivity of the waterway. Debris is expected to be minimal.

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

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

### 6.3.3.8.2 Complete or Substantial Removals

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

- For work **over waterways or wetlands**, a method of protecting the waterway is needed if the removal area is located over the waterway. If the removal area is located over the waterway, use one of the three Removing Structure Over Waterway (structure) bid items noted in 6.3.3.8. If the removal area is not located over the waterway, use Removing Structure (structure). The Debris Containment Over Waterway (structure) item is not used for this work.

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

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

### 6.3.3.8.3 False Decking

In some cases, false decking systems are required and language in the form of stp-502-015 should be included in the contract. This language is included where the removal operations carry a risk of falling construction debris onto live traffic or pedestrian facilities below, specifically in situations that carry significant risk. The following conditions are the baseline for when this

STSP language is to be included (note that these criteria are minimum application locations and that Regions may determine other suitable locations for project-specific reasons):

- All bridges over interstate highways with live traffic below.
- All bridges over roadways with a minimum ADT of 10,000 with live traffic below.
- All bridges over pedestrian facilities that will remain open during construction.

## 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 Computations 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)

Bid as Each and as a single unit item for the entire structure. 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 (e.g. Pier 1) to the nearest 0.1 cubic yard. Show the total quantity to the nearest cubic yard. The unit quantities do not need to be adjusted so the sum of the unit quantities equals 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 (Size)(Shell Thickness), Piling Steel HP (Size)

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

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

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

### 6.4.11 Preboring CIP Concrete Piling or Steel Piling

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

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

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

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

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

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

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

Record this quantity to the nearest 1 cubic yard.

### 6.4.15 Pile Points

When recommended in soils report. Bid as each.

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

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

### 6.4.17 Cofferdams (Structure)

Bid as Each and as a single unit item for the entire structure.

### 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 the outermost extent of the expansion device 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 Structure and Debris Containment

For work over roadways and railroads, "Removing Structure (structure)" is most typically used for complete or substantial removals. For work over waterways, one of the following Standard Specification bid items should be used for complete or substantial removals: Removing Structure Over Waterway Remove Debris (structure); Removing Structure Over Waterway Minimal Debris (structure); or Removing Structure Over Waterway Debris Capture (structure).

For work other than complete or substantial removals, a Removing Structure (structure) bid item may not be required.

Use Debris Containment (structure) bid items, **only as needed** based on the significance, extent, or location of the removal.

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

Bid as each.

## 6.4.29 Anchor Assemblies for Steel Plate Beam Guard

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

### 6.4.30 Steel Diaphragms (Structure)

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

### 6.4.31 Welded Stud Shear Connectors X -Inch

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

### 6.4.32 Concrete Masonry Seal

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

### 6.4.33 Geotextile Fabric Type

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

### 6.4.34 Concrete Adhesive Anchors

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

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

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

### 6.4.36 Piling Steel Sheet Temporary

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

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

### 6.4.37 Temporary Shoring

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

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

### 6.4.38 Concrete Masonry Deck Repair

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

## 6.4.39 Sawing Pavement Deck Preparation Areas

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

## 6.4.40 Removing Bearings

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

## 6.4.41 Ice Hot Weather Concreting

Used to provide a mechanism for payment of ice during hot weather concreting operations. See FDM 19-5-3.2 for bid item usage guidance and quantity calculation guidance. Bid as LB and round to the nearest 5 lbs.

## 6.4.42 Asphaltic Overlays

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

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

Coordinate asphaltic quantity assumptions with the Region and roadway designers.

## 6.4.43 Longitudinal Grooving

This quantity is typically used for High Performance Concrete (HPC) structures with a design speed of 40 mph or greater. See 17.8.2 for additional guidance. Record this quantity to the nearest square foot.

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

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.

The QA/QC plan shall also include the following items:

- Identification of a lead QA/QC Structures Program contact
- Identification of the QA/QC plan and procedures implementation date
- A statement indicating that the independent design check will be performed by an individual other than the designer, and the independent plan check will be performed by an individual other than the drafter.

Provisions for periodic reviews and update of the QA/QC plan with a frequency no less than 5 years; or as needed due to changes in the firm's personnel or firm's processes or procedures; or as requested by BOS.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.

Consultants' QA/QC plans and verification summary sheets may be subject to periodic reviews by BOS. These reviews are intended to assess compliance with BOS requirements listed above.

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

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

## 6.5.1 Approvals, Distribution, and Work Flow

| | |
|---|---|
| Consultant | Meet with Regional Office and/or local units of government to determine need. |
| | Prepare Structure Survey Report including recommendation of structure type. |
| Geotechnical Consultant | Make site investigation and prepare Site 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 responses to all preliminary plan comments. Responses to comments shall either agree or disagree with an explanation why the comment will not be incorporated in the final design and plans. |
| | Prepare and complete final design and plans for the specified structure. |
| | Write special provisions. |

| | |
|---|---|
| | At least **two months** in advance of the PS&E date, submit the required final design documents via e-submit per 6.5.3. |
| Structures Design Section | Determine which final plans will be reviewed and perform quality assurance review as applicable. |
| | For final plans that are reviewed, return comments to Consultant and send copy to Regional Office, including FHWA as appropriate. |
| Consultant | Modify final plans and specifications as required. |
| | Submit modified final plans via e-submit as required. |
| Structures Design Section | Review modified final plans as applicable. |
| | Sign final plans and send performance evaluation form to Region and Consultant. |
| Geotechnical Consultant | At time of PSE, transmit gINT boring logs, soils laboratory testing summary and data sheets, and Soil Reports to the emails provided in the Soils and Subsurface Investigations section of Two/Three Party Design Contract Special Provisions. |
| Bureau of Project Development | Prepare final accepted structure plans for pre-development contract administration. |
| Consultant | If a plan change is needed after being advertised but before being let, an addendum is required per FDM 19-22-1 and 19-22 Attachment 1.2. |
| Structures Design Section | Review structure addendum as applicable. |
| | Sign structure addendum. |
| Bureau of Project Development | Distributes structure addendum to bidders. |
| Consultant | If a plan change is required after being let, a post-let revision is required per 6.5.5. |
| Structures Design Section | Review post-let revision as applicable. |
| | Stamp post-let revision plan as accepted. |
| | Delivers revised plan to DOT construction team for distribution. |

**Table 6.5-1**
Approvals, Distribution and Work Flow

## 6.5.2 Preliminary Plan Requirements

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

1. Hydrology Report

2. Structure Survey Report

3. Preliminary plan, including log borings shown on the subsurface exploration sheet

4. Evaluation of Site 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. Final Site Investigation Report

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

5. QA/QC Verification Sheet

6. Inventory Data Sheet

7. Bridge Load Rating Summary Form

8. Responses to all BOS Preliminary Plan comments. Include responses (agree/disagree and why) on the marked up Preliminary Plan provided by BOS. Attach any additional clarifying notes or correspondence regarding Preliminary Plan comments to the end of the Preliminary Plan. E-submit as OTHER document.

9. 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.

### 6.5.7 Locally-Funded Projects

Local highway bridges designed under authority of the local municipality or county highway department without utilizing state or federal oversight must provide to BOS at minimum the following documents: (1) Bridge Inventory Data Sheet, (2) Scour assessment/evaluation documentation, (3) Bridge load rating summary sheet, and (4) Construction documentation such as an as-built plan and shop drawings. The scour assessment/evaluation documentation should be prepared in accordance with 8.3.2.7. All bridges shall be evaluated to determine the vulnerability to scour. See Chapter 8 – Hydraulics for additional guidance on hydrologic and hydraulic analysis.

## 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. Site Investigation 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 Summary sheet

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

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

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

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

10. As-Let Plans: After bid letting, a digital image of the As-Let plans are placed in a computer folder in the Bureau of Project Development (BPD). BOS office support staff extract a digital copy of the As-Let structure plans and place it in the structure folder for viewing on HSI.

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

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

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

**Table 6.6-1**
Various Inspection Reports

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

## 6.6.2 Resources

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

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

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

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

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

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

http://bridges.transportation.org

https://www.arema.org/

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## 9.1 General

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

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

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

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

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

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

## 9.7 Miscellaneous Materials

Several types of materials are being used as part of a bridge deck protective system. Epoxy coated reinforcing steel, mentioned earlier, is part of this system. Some of these materials or products, are experimental and are placed on specific structures and then monitored and evaluated. A list of materials or products that are part of a bridge deck protective system and are currently used or under evaluation are:

- Galvanized or stainless steel reinforcing bars
- Waterproofing membrane with bituminous concrete overlay
- Microsilica modified concrete or polymer impregnated concrete
- Low slump concrete overlays
- Low-viscosity crack sealer
- Cathodic protection systems with surface overlays

Other materials or products used on highway structures are:

- Downspouts for Type GC and H drains may be fabricated from fiberglass conforming to ASTM D2996, Grade 1, Class A.
- Elastomeric bearing pads (non-laminated) are primarily used with prestressed “I” girders at fixed abutments and piers and at semi-expansion abutments. They are also used with prestressed “slab and box” sections at all supports. The requirements for these pads are described in Section 506.2.6 of the *Standard Specifications*.
- Elastomeric bearing pads (laminated) are primarily used with prestressed “I” girders at expansion supports. The requirements for these pads are described in Section 506.2.6 of the *Standard Specifications*.
- Preformed fillers are placed vertically in the joint between wing and diaphragm in A1 and A5 abutments, in the joint between wing and barrel in box culverts and at expansion joints in concrete cast-in-place retaining walls. Preformed fillers are placed along the front top surface of A1 and A5 abutments, along the outside top surfaces of fixed piers and under flanges between elastomeric bearing pad (non-laminated) and top edge of support. Cork filler is placed vertically on semi-expansion abutments. The requirements for fillers are described in Section 502.2.7 of the *Standard Specifications*.
- Polyethelene sheets are placed on the top surface of semi-expansion abutments to allow movement of the superstructure across the bearing surface. They are also placed between the structural approach slab and the subgrade, and the approach slab and its footing.

- Rubberized waterproofing membranes are used to seal horizontal and vertical joints at the backface of abutments, culverts and concrete cast-in-place retaining walls. See Section 516.2.3 of the *Standard Specifications*.

- Non-staining gray non-bituminous joint sealer is used to seal exposed surfaces of preformed fillers placed in joints as described above. It is also used to place a seal around exposed surfaces of plates used at deflection joints and around railing base plates. The requirement for this joint sealer is referenced in Section 502.2.9 of the *Standard Specifications*.

- Plastic plates may be used at deflection joints in sidewalks and parapets.

- Preformed Fabric, Class A, has been used as a bearing pad under steel bearings. The requirement for this material is given in Section 506.2.8.4 of the *Standard Specifications*.

- Neoprene strip seals are used in single cell and multi-cell (modular) expansion devices.

- Teflon sheets are bonded to steel plates in Type A-T expansion bearings. The requirements for these sheets are found in Section 506.2.8.3 of the *Standard Specifications*.

- Asphalt panels are used on railroad structures to protect the rubber membrane on top of the steel ballast plate from being damaged by the ballast. The requirements for these panels are in the “*Special Provisions*”.

- Geotextile fabric is used for drainage filtration, and under riprap and box culverts. This fabric consists of sheets of woven or non-woven synthetic polymers or nylon. Type DF is used for drainage filtration in the pipe underdrain detail placed behind abutments and walls. The fabric allows moisture to drain to the pipe while keeping the backfill from migrating into the coarse material and then into the pipe. Type DF is also used behind abutments or walls that retain soil with backing planks between or behind piling and also for some of the walls detailed in Chapter 14 – Retaining Walls. This fabric will allow moisture to pass through the fabric and the joints in the walls without migration of the soil behind the wall. Type R or HR is placed below riprap and will keep the soil beneath it from being washed away. Type C is placed under breaker run when it is used under box culverts. The requirements for these fabrics are found in Section 645.2 of the *Standard Specifications*.

- Permanent pavement marking types include paint, epoxy, grooved with wet reflective epoxy, and grooved with tape. Grooved pavement markings (0.09 to 0.175 inches deep) are acceptable for concrete bridge decks, concrete overlays, and polyester polymer concrete overlays (typically 3/4” to 1” thick) but not recommended on thin polymer overlays (bid items “Polymer Overlay” and “High Friction Surface Treatment Polymer Overlay”). To avoid damaging the overlay deck protection, pavement markings on thin polymer overlays should be limited to epoxy markings. Refer to the Traffic Engineering, Operations and Safety Manual (TEOpS) and Section 650 of the Construction and Materials Manual (CMM) for additional information.

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## 14.6 Mechanically Stabilized Earth Retaining Walls

### 14.6.1 General Considerations

Mechanically Stabilized Earth (MSE) is the term used to describe the practice of reinforcing a mass of soil with either metallic or geosynthetic soil reinforcement which allows the mass of soil to function as a gravity retaining wall structure. The soil reinforcement is placed horizontally across potential planes of shear failure and develops tension stresses to keep the soil mass intact. The soil reinforcement is attached to a wall facing located at the front face of the wall.

The design of MSE walls shall meet the *AASHTO LRFD* requirements in accordance with 14.4.2. The service life requirement for both permanent and temporary MSE wall systems is presented in 14.4.3.

The MSE walls shall be designed for external stability of the wall system and internal stability of the reinforced soil mass. The global stability shall also be considered as part of design evaluation. MSE walls are proprietary wall systems and the design responsibilities with respect to global, external, and internal stability as well as settlement are shared between the designer (WisDOT or Consultant) and contractor. The designer is responsible for the overall stability, preliminary external stability and settlement whereas the contractor is responsible for the internal stability, compound stability and structural design of the wall. For settlement, the designer shall select the appropriate wall facing type (e.g. small 5'x5' precast panels) and locate slip joints locations, as required. The contractor should accommodate wall settlement shown on contract documents and based on the wall supplier recommendations. The responsibilities of the designer and contractor are outlined in 14.6.3.2. The design and drawings of MSE walls provided by the contractor must also be in compliance with the WisDOT special provisions as stated in 14.15.2 and 14.16.

The design engineer should detail the MSE wall and any supporting structures (e.g. a bridge abutment) to ensure settlements are properly accommodated. This may include limiting the MSE wall to small precast concrete panels (<30 sf ft), detailing coping extensions on adjacent structures, or locating ship joints accordingly.

The guidelines provided herein for MSE walls do not apply to geometrically complex MSE wall systems such as tiered walls (walls stacked on top of one another), back-to-back walls, or walls which have trapezoidal sections. Design guidelines for these cases are provided in publications FHWA-NHI-10-024 and FHWA-NHI-10-025.

Horizontal alignment and grades at the bottom and top of the wall are determined by the design engineer. The design must be in compliance with the WisDOT special provisions for the project and the policy and procedures as stated in the *Bridge Manual* and FDM.

#### 14.6.1.1 Usage Restrictions for MSE Walls

Construction of MSE walls with either block or panel facings should not be used when any of the following conditions exist:

1. If the available construction limit behind the wall does not meet the soil reinforcement length requirements.

2. Sites where extensive excavation is required or sites that lack granular soils and the cost of importing suitable fill material may render the system uneconomical.

3. At locations where erosion or scour may undermine or erode the reinforced fill zone or any supporting leveling pad.

4. Soil is contaminated by corrosive material such as acid mine, drainage, other industrial pollutants, or any other condition which increases corrosion rate, such as the presence of stray electrical currents.

5. There is potential for placing buried utilities within (or below) the reinforced zone unless access is provided to utilities without disrupting reinforcement and breakage or rupture of utility lines will not have a detrimental effect on the stability of the wall. Contact Bureau of 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.

The non-metallic or extensible reinforcement includes the following:

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

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

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

### 14.6.2.3 Facing Elements

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

Major facing types are:

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

**Segmental Precast Concrete Panels**

Segmental precast concrete panels include small panels (<30 sq ft) to larger ($\geq$30 sq ft and < 75 sq ft) with a minimum thickness of 5-½ inches and square or rectangular in geometry. Less common geometries such as cruciform, diamond, and hexagonal are currently not being used. The geometric pattern of the joints and the smooth uniform surface finish of the factory provided precast panels give an aesthetically pleasing appearance. Segmental precast concrete panels are proprietary wall components.

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

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

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

**Concrete Modular Blocks Facings**

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

Figure 14.6-3 shows a typical modular block MSE wall system along with other wall components. Most modular block MSE walls are reinforced with geogrids.

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

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

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

Alignments that are not straight (i.e. kinked or curved) shall use 90 degree corners or curves. The minimum radius should be limited to 8 feet measured to the front face of the top course for small blocks and 15-ft for large blocks For a concave wall the limiting radius is measured to the front face of the bottom course. For convex walls the limiting radius is measured to the front face of the top course. In no case shall the radius be less than 6 feet for small blocks. Use of the minimum radii should be avoided and may require additional investigation. It is WisDOT policy to design modular block MSE walls for a maximum height of 22 ft (measured from the top of the leveling pad to the top of the wall).

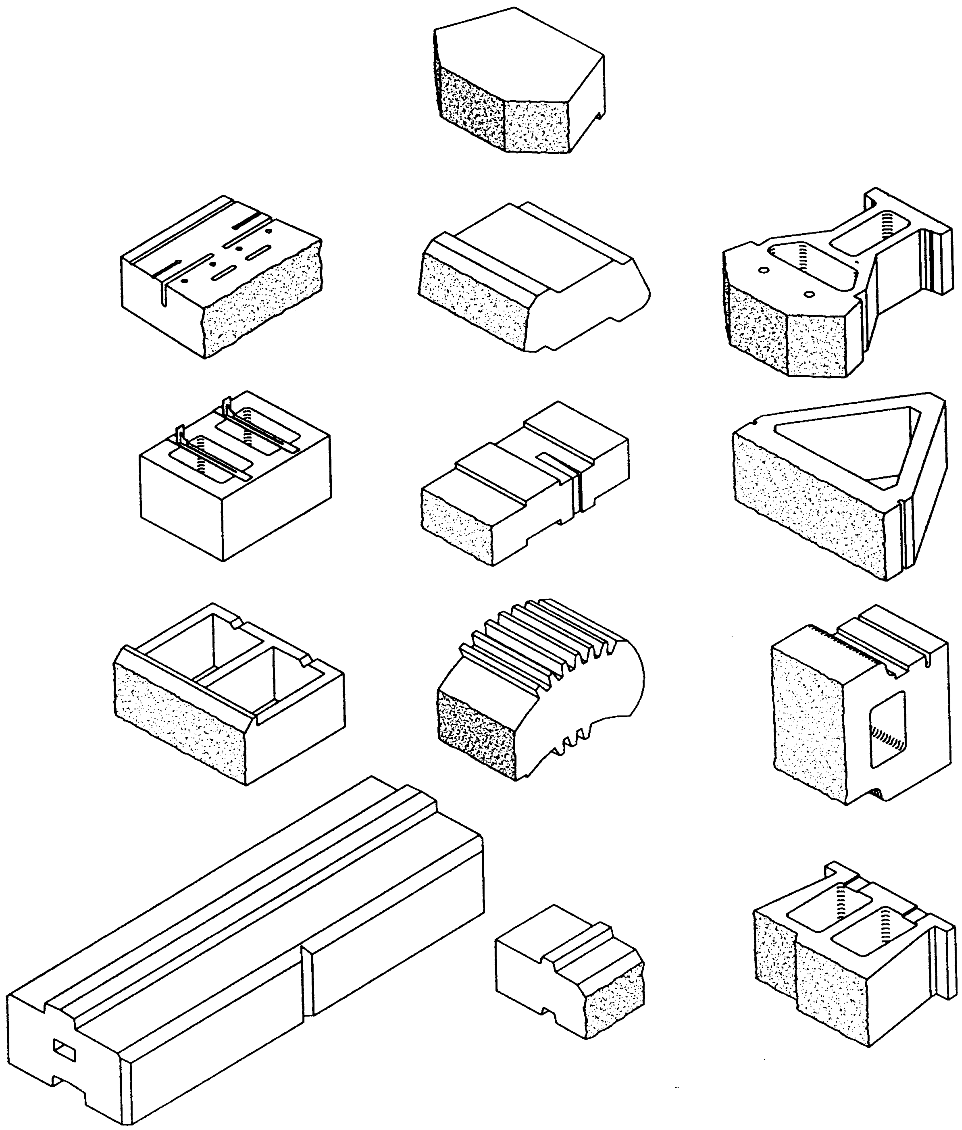
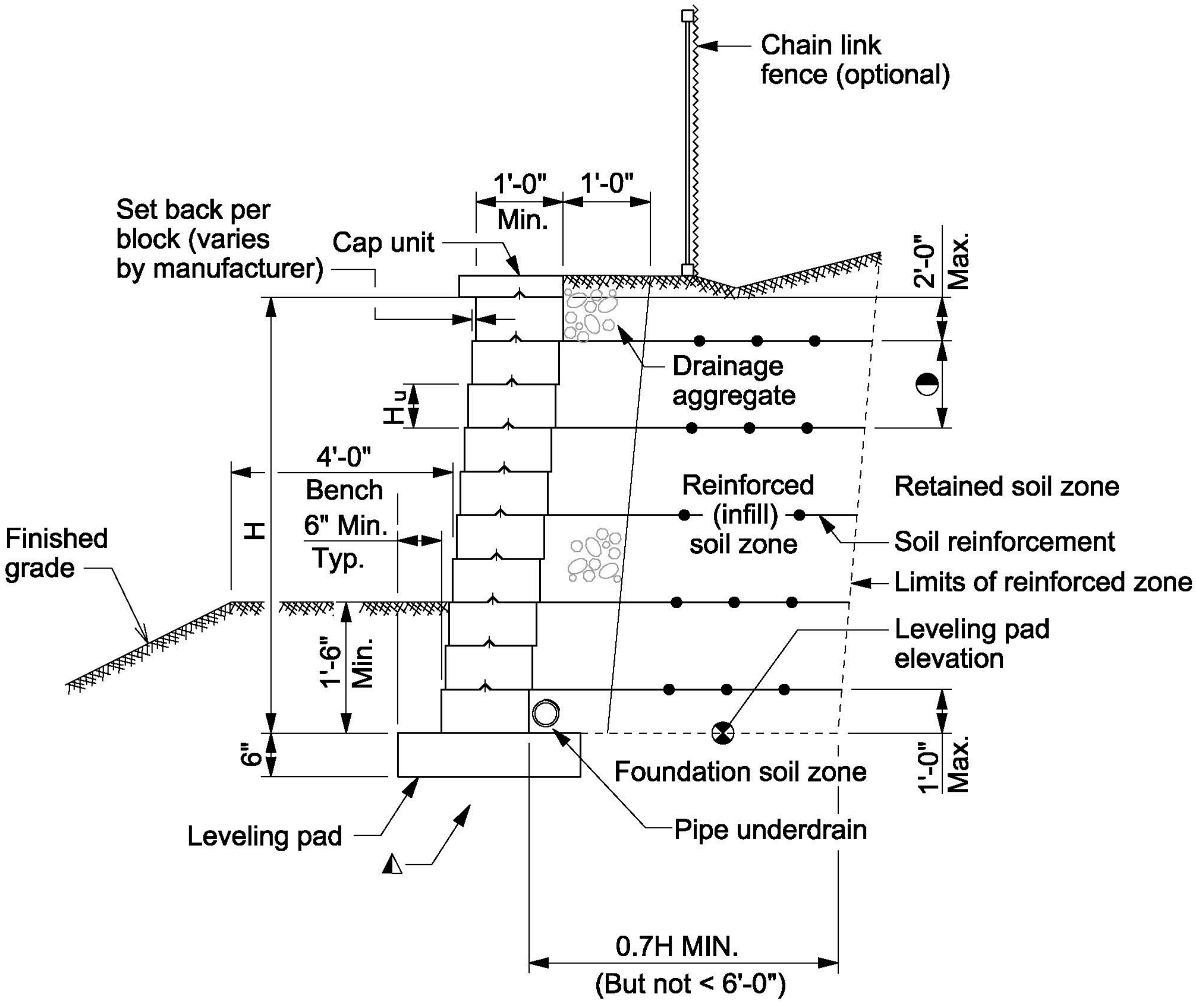


**Figure 14.6-2**
Modular Blocks
(Source FHWA-NHI-10-025)




**Modular Block MSE Wall**

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

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

**Figure 14.6-3**
Typical Modular Block MSE Walls

## 14.7 Modular Block Gravity Walls

The proprietary modular blocks used in combination with soil reinforcement "Mechanically Stabilized Earth Retaining Walls with Modular Block Facings" can also be used as pure gravity walls (no soil reinforcement). These walls consist of a single row of dry stacked blocks (without mortar) to resist external pressures. A drawback is that these walls are settlement sensitive. This wall type should only be considered when adequate provisions are taken to keep the surface water runoff and the ground water seepage away from the wall face.

The material specifications for the blocks used for gravity walls are identical to those for the blocks used for block MSE walls as discussed in 14.6.2.3. The modular block gravity walls are proprietary. The wall supplier is responsible for the design of these walls. Design drawings and calculations must be submitted to WisDOT for approval.

The height to which they can be constructed, is a function of the depth of the blocks, the setback of the blocks, the front slope and backslope angle, the surcharge on the retained soil and the angles of internal friction of the retained soil behind the wall. Walls of this type are limited to a height from top of leveling pad to top of wall of 8 feet or less, and are limited to a maximum differential settlement of 1/200.

Footings for modular block gravity walls are either base aggregate dense 1-¼ inch (Section 305 of the *Standard Specifications)* or Grade A concrete. Minimum footing thickness is 12 inches for aggregate and 6 inches for concrete. The width of the footing equals the width of the bottom block plus 12 inches for aggregate footings and plus 6 inches for concrete footings. The bottom modular block is central on the leveling pad. The standard special provisions for Modular Block Gravity Walls require a concrete footing if any portion of a wall is over 5 feet measured from the top of the footing to the bottom of the wall cap.

The coarse aggregate No. 1 (501.2.5.4 of the *Standard Specifications*), is placed within 1 foot behind the back face of the wall, extending down to the bottom of the footing.

### 14.7.1 Design Procedure for Modular Block Gravity Walls

All modular block gravity walls shall be designed to resist external pressure caused by the supported earth, surcharge loads, and water in accordance with the design criteria discussed in **LRFD [11.11.4]** and 14.4. The design requires an external stability evaluation including sliding, eccentricity check, and bearing resistance check at the Strength I limit state and the evaluation of wall settlement and overall stability at the Service I limit state.

The design of modular block gravity walls provided by the contractor must be in compliance with the WisDOT special provisions for the project and the policy and procedures as stated in 14.15.2 and 14.16. The design must include an analysis of external stability including sliding, eccentricity, and bearing stress check. Horizontal shear capacity between blocks must also be verified by the contractor.

Settlement and overall stability calculations are the responsibility of the designer. The soil design parameters and allowable bearing capacity for the design are provided by the Geotechnical Engineer, including the minimum required block depth.

### 14.7.1.1 Initial Sizing and Wall Embedment

The minimum embedment to the top of the footing for modular block gravity walls is the same as stated in **LRFD [11.10.2.2]** for mechanically stabilized earth walls. Wall backfill slope shall not be steeper than 2:1. Where practical, a minimum 4.0 ft wide horizontal bench shall be provided in front of the walls.

Wall embedment for prefabricated modular walls shall meet the requirements discussed in section14.4.7.5. The minimum embedment shall be 1.5 ft. or the requirement of scouring or erosion due to flooding defined in 14.6.3.4.1.

### 14.7.1.2 External Stability

The external stability analyses shall develop the unfactored and factored loads and include evaluations for sliding, eccentricity check, and bearing resistance in accordance with **LRFD [11.11.4]. LRFD [11.11.4.1]** requires that wall stability be performed at every block level.

#### 14.7.1.2.1 Unfactored and Factored Loads

Unfactored loads and moments shall be computed after establishing the initial wall geometry and using procedures defined in 14.4.5.4.5. A load diagram as shown in Figure 14.4-5 shall be developed. Factored loads and moments shall be computed as discussed in 14.4.6 by multiplying applicable load factors given in Table 14.4-1. A summary of load factors and load combinations as applicable for a typical modular block wall is presented in Table 14.7-1. Computed factored load and moments are used for performing stability checks.

#### 14.7.1.2.2 Sliding Stability

Sliding should be considered for the full height wall and at each block level in the wall. The stability should be computed in accordance with **LRFD [10.6.3.4]**, using the following equation:

$$R_R = \phi\, R_n = \phi_\tau\, R_\tau$$

Where:

| | | |
|---|---|---|
| $R_R$ | = | Factored resistance against failure by sliding |
| $R_n$ | = | Nominal sliding resistance against failure by sliding |
| $\phi_\tau$ | = | Resistance factor for shear between soil and foundation per **LRFD [Table 10.5.5.2.2-1]** |
| $\phi_\tau$ | = | 0.9 for concrete on sand and 1.0 for soil on soil |
| $R_\tau$ | = | Nominal sliding resistance between soil and foundation |

No passive soil pressure is allowed to resist sliding. The component of the passive resistance shall be ignored due to the possibility that permanent or temporary excavations in front of the

## 14.12 Temporary Shoring

This information is provided for guidance. Refer to the *Facilities Development Manual* for further details.

Temporary shoring is used to support a temporary excavation or protect existing transportation facilities, utilities, buildings, or other critical features when safe slopes cannot be made for structural excavations. Shoring may be required within the limits of structures or on the approach roadway due to grade changes or staged construction. Temporary shoring generally includes non-anchored temporary sheet piles, temporary soldier pile walls, temporary soil nails, cofferdam, or temporary mechanically stabilized earth (MSE) walls.

Temporary shoring is designed by the contractor. Shoring should not be required nor paid for when used primarily for the convenience of the contractor.

### 14.12.1 When Slopes Won't Work

Typically shoring will be required when safe slopes cannot be made due to geometric constraints of existing and proposed features within the available right-of-way. Occupation and Healthy Safety Administration (OSHA) requirements for temporary excavation slopes vary from a 1H:1V to a 2H:1V. The contractor is responsible for determining and constructing a safe slope based on actual site conditions.

In most cases, the designer can assume that an OSHA safe temporary slope can be cut on a 1.5H:1V slope; however other factors such as soil types, soil moisture, surface drainage, and duration of excavation should also be factored into the actual slope constructed. As an added safety factor, a 3-foot berm should be provided next to critical points or features prior to beginning a 1.5H:1V slope to the plan elevation of the proposed structure. Sufficient room should be provided adjacent to the structure for forming purposes (typically 2-3 feet).

### 14.12.2 Plan Requirements

Contract plans should schematically show in the plan and profile details all locations where the designer has determined that temporary shoring will be required. The plans should note the estimated length of the shoring as well as the minimum and maximum required height of exposed shoring. These dimensions will be used to calculate the horizontal projected surface area projected on a vertical plane of the exposed shoring face.

### 14.12.3 Shoring Design/Construction

The Contractor is responsible for design, construction, maintenance, and removal of the temporary shoring system in a safe and controlled manner. The adequacy of the design should be determined by a Wisconsin Professional Engineer knowledgeable of specific site conditions and requirements. The temporary shoring should be designed in accordance with the requirements described in 14.4.2 and 14.4.3. A signed and sealed copy of proposed designs must be submitted to the WisDOT Project Engineer for information.

## 14.13 Noise Barrier Walls

### 14.13.1 Wall Contract Process

WisDOT has classified all noise walls (both proprietary and non-proprietary) into three wall systems. All proprietary systems must be pre-approved prior to being considered for use on WisDOT projects. The three noise wall systems that are considered for WisDOT projects include the following:

1. Double-sided sound absorptive noise barriers
2. Single-sided sound absorptive noise barriers
3. Reflective noise barriers

If a wall is required, the designer must determine which wall system or systems are suitable for a given wall location. In some locations all wall systems may be suitable, whereas in other locations some wall systems may not be suitable. Information on aesthetic qualities and special finishes and colors of proprietary systems is available from the manufacturers. Information on approved concrete paints, stains and coatings is also available from the Structures Design Section. Designers are encouraged to contact the Bureau of Structures Design Section if they have any questions about the material presented in the *Bridge Manual*.

The step by step process required to select a suitable wall system or systems for a given wall location is as follows:

Step 1: Investigate alternatives

Investigate alternatives to walls such as berms, plantings, etc.

Step 2: Geotechnical analysis

If a wall is required, geotechnical personnel shall conduct a soil investigation at the wall location and determine soil design parameters for the foundation soil. Geotechnical personnel are also responsible for recommending remedial methods of improving soil bearing capacity if required.

Step 3: Evaluate basic wall restrictions

The designer shall examine the list of suitable wall systems using the Geotechnical Report and remove any system that does not meet usage restrictions for the site.

Step 4: Determine suitable wall systems

The designer shall further examine the list of suitable wall systems for conformance to other considerations. Refer to Chapter 2 – General and Chapter 6 – Plan Preparation for a discussion on aesthetic considerations.

Step 5: Determine contract letting

## 14.14 Contract Plan Requirements

The following minimum information shall be required on the plans.

1. Plan view showing wall and roadway stationing. A reference north arrow shall be included.

2. Elevation view showing the front face of the wall. It is acceptable to show the plan view and elevation view with the wall stationing progressing downstation from left to right on the sheet (i.e. end of wall shown on the left side of page and start of wall on the right) to allow for a front face elevation view of the wall.

3. Wall grades at the front face of wall at 25-foot intervals or less. Beginning and end stations of wall and offsets from reference line to front face of walls. If reference line is a horizontal curve give offsets from a tangent to the curve.

4. Cross-Section view.

5. Location of right-of-way boundaries, and construction easements relative to the front face of the walls.

6. Location of utilities if any and indicate whether to remain in place or be relocated or abandoned.

7. Special requirements on top of wall such as copings, railings, or traffic barriers.

8. Footing or leveling pad elevations if different than standard.

9. General notes on standard insert sheets.

10. Soil design parameters for retained soil, backfill soil and foundation soil including angle of internal friction, cohesion, coefficient of sliding friction, groundwater information and ultimate and/or allowable bearing capacity for foundation soil. If piles are required, give skin friction values and end bearing values for displacement piles and/or the allowable steel stress and anticipated driving elevation for end bearing piles.

11. Soil borings.

12. Details of special architectural treatment required for each wall system.

13. Wall systems, system or sub-systems allowed on projects.

14. Abutment details if wall is component of an abutment.

15. Connection and/or joint details where wall joins another structure.

16. Groundwater elevations.

17. Drainage provisions at heel of wall foundations.

18. Drainage at top of wall to divert run-off water.

19. Location of name plate.

## 14.15 Construction Documents

### 14.15.1 Bid Items and Method of Measurement

Proprietary retaining walls shall include all required bid items necessary to build the wall system provided by the contractor. The unit of measurement shall be square feet and shall include the exposed wall area between the footing and the top of the wall measured to the top of any copings. For setback walls the area shall be based on the walls projection on a vertical plane. The bid item includes designing the walls preparing plans, furnishing and placing all materials, including all excavations, temporary bracing, piling, (including delivered and driven), poured in place or precast concrete or blocks, leveling pads, soil reinforcement systems, structural steel, reinforcing steel, backfills and infills, drainage systems and aggregate, geotextiles, architectural treatment including painting and/or staining, drilled shafts, wall toppings unless excluded by contract, wall plantings, joint fillers, and all labor, tools, equipment and incidentals necessary to complete the work.

The contractor will be paid for the plan quantity as shown on the plans. (The intent is a lump sum bid item but is bid as square feet of wall). The top of wall coping is any type of cap placed on the wall. It does not include any barriers. Measurement is to the bottom of the barrier when computing exposed wall area.

Non-proprietary retaining walls are bid based on the quantity of materials used to construct the wall such as concrete, reinforcing steel, piling, etc. These walls are:

- Cast-in-Place Concrete Cantilever Walls
- Soldier Pile Walls
- Steel Sheet Piling Walls

### 14.15.2 Special Provisions

The Bureau of Structures has Special Provisions for:

- Wall Modular Block Gravity Landscape, Item SPV.0165.
- Wall Modular Block Gravity, Item SPV.0165.
- Wall Modular Block Mechanically Stabilized Earth, Item SPV.0165
- Wall Concrete Panel Mechanically Stabilized Earth, Item SPV.0165
- Wall Wire Faced Mechanically Stabilized Earth, Item SPV.0165. and Presstressed Precast Concrete Panel, Item SPV.0165
- Geosynthetic Reinforced Soil Abutment, Item SPV.0165
- Temporary Wall Wire Faced Mechanically Stabilized Earth, Item SPV.0165

- *Wall Gabion, Item* SPV.0165*

** Contact BOS Design before using and to obtain the most recent version.*

Note that the use of QMP Special Provisions began with the December 2014 letting or prior to December 2014 letting at the Region's request. Special provisions are available on the Wisconsin Bridge Manual website.

The designer determines what wall systems(s) are applicable for the project. The approved names of suppliers are inserted for each eligible wall system. The list of approved proprietary wall suppliers is maintained by the Bureau of Structures which is responsible for the Approval Process for earth retaining walls, 14.16.

## 14.16 Submittal Requirements for Pre-Approval Process

### 14.16.1 General

The following four wall systems require the supplier or manufacturer to submit to the Structural Design Section a package that addresses the items specified in 14.16.3.

1. Modular Block Gravity Walls

2. MSE Walls with Modular Block Facings

3. MSE Walls with Precast Concrete Panel Facings

4. Modular Concrete Bin or Crib Walls

### 14.16.2 General Requirements

Approval of retaining wall systems allows for use of these systems on Wisconsin Department of Transportation (WisDOT) projects upon the manufacturer's certification that the system as furnished to the contractor (or purchasing agency) complies with the design procedures specified in the *Bridge Manual*. WisDOT projects include: State, County and Municipal Federal Aid and authorized County and Municipal State Aid projects in addition to materials purchased directly by the state.

The manufacturer shall perform all specification tests with qualified personnel and maintain an acceptable quality control program. The manufacturer shall maintain records of all its control testing performed in the production of retaining wall systems. These test records shall be available at all times for examination by the Construction Materials Engineer for Highways or designee. Approval of materials will be contingent upon satisfactory compliance with procedures and material conformance to requirements as verified by source and field samples. Sampling will be performed by personnel during the manufacture of project specific materials.

### 14.16.3 Qualifying Data Required For Approval

Applicants requesting Approval for a specific system shall provide three copies of the documentation showing that they comply with *AASHTO LRFD* and *WisDOT Standard Specifications* and the design criteria specified in the *Bridge Manual*.

1. An overview of the system, including system theory.

2. Laboratory and field data supporting the theory.

3. Detailed design procedures, including sample calculations for installations with no surcharge, level surcharge and sloping surcharge.

4. Details of wall elements, analysis of structural elements, capacity demand ratio, load and resistance factors, estimated life, corrosion design procedure for soil reinforcement elements, procedures for field and laboratory evaluation including instrumentation and special requirements, if any.

5. Sample material and construction control specifications - showing material type, quality, certifications, field testing and placement procedures.

6. A well documented field construction manual describing in detail and with illustrations where necessary, the step by step construction sequence.

7. Details for mounting a concrete traffic barrier on the wall adjoining both concrete and flexible pavements (if applicable).

8. Pullout data for facing block/geogrid connection and soil pullout data (if applicable).

9. Submission of practical application with photos for all materials, surface textures and colors representative of products being certified.

10. Submission, if requested, to an on-site production process control review, and record keeping review.

11. List of installations including owner name and wall location.

12. Limitations of the wall system.

The above materials may be submitted at any time (recommend a minimum of 15 weeks) but, to be considered for a particular WisDOT project, must be approved prior to the bid opening date. The material should be clearly detailed and presented according to the prescribed outline.

After final review and approval of comments with the Bureau of Structures, the manufacturer will be approved to begin presenting the system on qualified projects.

## 14.16.4 Maintenance of Approval Status as a Manufacturer

The supplier or manufacturer must request to be reapproved bi-annually. The request shall be in writing and certify that the plant production process control and materials testing and design procedures haven't changed since the last review. The request shall be received within two years of the previous approval or the approval status will be terminated. Upon request for re-approval an on-site review of plant process control and materials testing may be conducted by WisDOT personnel. Travel expenses for trips outside the State of Wisconsin involved with this review will be borne by the manufacturer.

For periodic on-site reviews, access to the plant operations and materials records shall be provided to a representative of the Construction Materials Engineer during normal working hours upon request.

If the supplier or manufacturer introduces a new material, or cross-section, or a new design procedure, into its product line, the new feature must be submitted for approval. If the new feature/features are significantly different from the original product, the new product may be subjected to a complete review for approval.

### 14.16.5 Loss of Approved Status

Approval to deliver the approved system may be withdrawn under the following conditions:

Design Conformance

1. Construction does not follow design procedures.
2. Incorrect design procedures are used on projects.

Materials

3. Inability to consistently supply material meeting specification.
4. Inability to meet test method precision limits for quality control testing.
5. Lack of maintenance of required records.
6. Improper documentation of shipments.
7. Not maintaining an acceptable quality control program.

The decision to remove approval from a manufacturer on a specific system rests with the Construction Materials Engineer for Highways or the State Bridge Engineer.

## 14.17 References


1. State of Wisconsin, Department of Transportation, *Facilities Development Manual*

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

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

4. AASHTO LRFD Bridge Design Specification 4th Edition, 2007, AASHTO, 444 North Capitol Street, N.W., Suite 249, Washington, D.C. 20001.

5. Bowles, Joseph E*. Foundation Analysis and Design 4th Edition.* McGraw Hill *1989*

6. Cudoto, Donald P. *Foundation Design Principles and Practices (* 2nd Edition*),* Prentice Halls

7. National Concrete Masonry Association, "Design Manual for Segmental Retaining Walls", 2302 Horse Pen Road, Herndon, Virginia 22071-3406.

8. Publication No. FHWA-NHI-14-007, "Geotechnical Engineering Circular No. 7 Soil Nail Walls – Reference Manual"

9. Publication No. FHWA-SA-96-069R, "Manual for Design and Construction of Soil Nail walls"

10. Publication No. FHWA-HI-98-032, "Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures"

11. Publication No. FHWA-NHI-07-071, "Earth Retaining Structures"

12. Publication No. FHWA-NHI-09-083, "Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures"

13. Publication No. FHWA-NHI-09-087, "Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced slopes"

14. Publication No. FHWA-NHI-10-024, "Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes-Volume I"

15. Publication No. FHWA-NHI-10-025, "Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes-Volume II"

16. Publication No. FHWA-NHI-24-002, "Design and Construction of Mechanically Stabilized Earth (MSE) Walls"

## 14.18 Design Examples

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The live load is taken as live load combinations LL#5: Design Truck + IM or LL#6: 25%(Design Truck + IM)+ Lane from 17.2.4.2.6.

## 17.3 Selection of Structure Type

The selection of the proposed structure type is determined from evaluation of the Structure Survey Report with accompanying supplemental data, current construction costs and preference based on past experience. In selecting the most economical structure, ease of fabrication and erection, general features of terrain, roadway geometrics, subsurface exploration and geographic location in the State of Wisconsin are considered. The proposed structure must blend into existing site conditions in a manner that does not detract from its surrounding environment. Every attempt should be made to select an aesthetically attractive structure consistent with structural requirements, economy and geographic surroundings. For information about bridge aesthetics, see Chapter 4 – Aesthetics.

The economical span ranges of various types of structures are given in Chapter 5 – Economics and Costs. Superstructure span lengths are related to the cost of the substructure units. If the substructure units are relatively expensive, it is generally more economical to use longer span lengths available for a given structure type. Practicality dictates using the average structure length for twin structures if the preliminary structure lengths are within approximately 3 feet. In addition, a multiple-span structure should be made symmetrical if its end spans are within approximately 3 feet in length of each other.

For most structures, use 1-foot increments for span lengths. Specify the skew angle in 1-degree increments for grade separations and 5-degree increments for stream crossings. Use more precise angles or span lengths when necessary.

For geometric considerations in structure selection, reference is made to Chapter 3 – Design Criteria. The requirements for structure expansion and fixed pier locations are presented in Chapter 12 – Abutments, and bearing types are described in Chapter 27 – Bearings. Expansion joint types and requirements are specified in Chapter 28 – Expansion Devices. Since the skew angle for most snow plow blades is 35°, it is desirable to avoid this skew angle for bridge joints. This reduces the chances of joint damage resulting from the plow blades dropping into the expansion joints.

Use of non-redundant structures, including single-box and two-box steel box girder bridges, should be avoided unless absolutely necessary. Certain situations, including extreme span length over a navigational channel or tight curvature, may necessitate such bridges.

### 17.3.1 Alternate Structure Types

When developing bridge plans, consider the following procedures:

- Base preliminary plan development on an engineering and economic evaluation of alternate designs.
- Evaluate alternative designs on the basis of competitive materials appropriate to a specific structure type.
- Do not propose specific construction methods or erection procedures in the plans unless constraints are necessary to meet specific project requirements.

| Longitudinal Reinforcing Steel for Deck Slabs on Girders for Deck Replacements, HL-93 Loading, Slab Thickness “T” < 8” | | | |
|---|---|---|---|
| Slab Thickness “T” (Inches) | Girder Spacing “S” | Bar Size and Spacing (Inches) | |
| | | Prestressed Girder Bridges | Steel Girder Bridges |
| | | #4’s Top and Bottom, Continuity Reinforcement To Be Designed (Top) | #4’s Top and Bottom, Continuity Reinforcement** #6’s (Top) |
| 6.5 | 4’-0” | 7.0 | 7.0 |
| 6.5 | 4’-3” | 7.0 | 7.0 |
| 6.5 | 4’-6” | 7.0 | 7.0 |
| 6.5 | 4’-9” | 7.0 | 7.0 |
| 6.5 | 5’-0” | 7.0 | 7.0 |
| 6.5 | 5’-3” | 7.0 | 7.0 |
| 6.5 | 5’-6” | 7.0 | 7.0 |
| 6.5 | 5’-9” | 6.5 | 6.5 |
| 6.5 | 6’-0” | 6.5 | 6.5 |
| 6.5 | 6’-3” | 6.5 | 6.5 |
| 6.5 | 6’-6” | 6.5 | 6.5 |
| 6.5 | 6’-9” | 6.0 | 6.0 |
| 6.5 | 7’-0” | 6.0 | 6.0 |
| 7 | 4’-0” | 8.0 | 8.0 |
| 7 | 4’-3” | 8.0 | 8.0 |
| 7 | 4’-6” | 8.0 | 8.0 |
| 7 | 4’-9” | 8.0 | 8.0 |
| 7 | 5’-0” | 8.0 | 8.0 |
| 7 | 5’-3” | 8.0 | 8.0 |
| 7 | 5’-6” | 8.0 | 8.0 |
| 7 | 5’-9” | 7.5 | 7.5 |
| 7 | 6’-0” | 7.5 | 7.5 |
| 7 | 6’-3” | 7.5 | 7.5 |
| 7 | 6’-6” | 7.0 | 7.0 |
| 7 | 6’-9” | 7.0 | 7.0 |
| 7 | 7’-0” | 7.0 | 7.0 |
| 7 | 7’-3” | 6.5 | 6.5 |
| 7 | 7’-6” | 6.5 | 6.5 |
| 7 | 7’-9” | 6.5 | 6.5 |
| 7 | 8’-0” | 6.0 | 6.0 |
| 7.5 | 4’-0” | 9.0 | 9.0 |
| 7.5 | 4’-3” | 9.0 | 9.0 |
| 7.5 | 4’-6” | 9.0 | 9.0 |
| 7.5 | 4’-9” | 9.0 | 9.0 |

| | | | |
|---|---|---|---|
| 7.5 | 5'-0" | 9.0 | 9.0 |
| 7.5 | 5'-3" | 9.0 | 9.0 |
| 7.5 | 5'-6" | 9.0 | 9.0 |
| 7.5 | 5'-9" | 8.5 | 8.5 |
| 7.5 | 6'-0" | 8.5 | 8.5 |
| 7.5 | 6'-3" | 8.5 | 8.5 |
| 7.5 | 6'-6" | 8.0 | 8.0 |
| 7.5 | 6'-9" | 8.0 | 8.0 |
| 7.5 | 7'-0" | 7.5 | 7.5 |
| 7.5 | 7'-3" | 7.5 | 7.5 |
| 7.5 | 7'-6" | 7.5 | 7.5 |
| 7.5 | 7'-9" | 7.0 | 7.0 |
| 7.5 | 8'-0" | 7.0 | 7.0 |
| 7.5 | 8'-3" | 6.5 | 6.5 |
| 7.5 | 8'-6" | 6.5 | 6.5 |
| 7.5 | 8'-9" | 6.5 | 6.5 |
| 7.5 | 9'-0" | 6.0 | 6.0 |
| 7.5 | 9'-3" | 6.0 | 6.0 |
| 7.5 | 9'-6" | 5.5 | 5.5 |

Legend:

** Use for deck slabs on steel girders in negative moment regions when not designed for negative moment composite action.

**Table 17.5-4**
Longitudinal Reinforcing Steel for Deck Slabs
on Girders for Deck Replacements, HL-93 Loading, Slab Thickness "T" < 8"
(Only use Table 17.5-4 if Bridge Rating is unacceptable with "T" ≥ 8")

The longitudinal reinforcing steel presented in Table 17.5-3 and Table 17.5-4 is designed in accordance with *AASHTO LRFD*. The tables are developed based on deck concrete with a 28-day compressive strength of $f'_c$ = 4 ksi and reinforcing steel with a yield strength of $f_y$ = 60 ksi. The dead load includes 20 psf for future wearing surface.

The reinforcing bars presented in the "Bar Size and Spacing" column (the third column) in Table 17.5-3 and Table 17.5-4 are for one layer only. Identical steel should be placed in both the top and bottom layers, except for continuity steel.

## 17.5.3.3 Empirical Design of Slab on Girders

**WisDOT policy item:**

Approval from the Bureau of Structures Design Section Chief is required for use of the empirical design method.

| Effective Overhang (Feet) | Deck Thickness Between Girders, “t” (Inches) | | | | | | |
|---|---|---|---|---|---|---|---|
| | 8 | 8.5 | 9 | 9.5 | 10 | 10.5 | 11 |
| 1.75 | 0.277 | 0.277 | 0.277 | 0.277 | 0.251 | 0.202 | 0.159 |
| 2.00 | 0.287 | 0.287 | 0.287 | 0.287 | 0.264 | 0.220 | 0.180 |
| 2.25 | 0.295 | 0.295 | 0.295 | 0.295 | 0.274 | 0.234 | 0.198 |
| 2.50 | 0.302 | 0.302 | 0.302 | 0.302 | 0.282 | 0.246 | 0.212 |
| 2.75 | 0.307 | 0.307 | 0.307 | 0.307 | 0.290 | 0.255 | 0.224 |
| 3.00 | 0.312 | 0.312 | 0.312 | 0.312 | 0.295 | 0.278 | 0.263 |
| 3.25 | 0.394 | 0.394 | 0.394 | 0.394 | 0.392 | 0.389 | 0.340 |
| 3.50 | 0.465 | 0.465 | 0.465 | 0.465 | 0.464 | 0.436 | 0.412 |
| 3.75 | 0.497 | 0.497 | 0.497 | 0.497 | 0.477 | 0.489 | 0.480 |
| 4.00 | 0.567 | 0.567 | 0.567 | 0.567 | 0.542 | 0.501 | 0.504 |

**Table 17.6-4**
Reinforcing Steel (in$^2$/ft) for Cantilever Deck Slabs with Tubular Railing Type “NY”/”M”
Girder Type 1

| Effective Overhang (Feet) | Deck Thickness Between Girders, “t” (Inches) | | | | | | |
|---|---|---|---|---|---|---|---|
| | 8 | 8.5 | 9 | 9.5 | 10 | 10.5 | 11 |
| 1.25 | 0.542 | 0.435 | 0.345 | 0.272 | 0.213 | 0.161 | 0.117 |
| 1.5 | 0.542 | 0.435 | 0.345 | 0.272 | 0.213 | 0.161 | 0.117 |
| 1.75 | 0.525 | 0.435 | 0.345 | 0.272 | 0.213 | 0.161 | 0.117 |
| 2 | 0.423 | 0.423 | 0.345 | 0.269 | 0.203 | 0.147 | 0.096 |
| 2.25 | 0.290 | 0.280 | 0.228 | 0.185 | 0.146 | 0.114 | 0.128 |
| 2.5 | 0.237 | 0.237 | 0.217 | 0.176 | 0.151 | 0.146 | 0.160 |
| 2.75 | 0.275 | 0.275 | 0.275 | 0.263 | 0.247 | 0.234 | 0.222 |
| 3 | 0.269 | 0.269 | 0.269 | 0.269 | 0.269 | 0.256 | 0.244 |
| 3.25 | 0.334 | 0.334 | 0.334 | 0.334 | 0.334 | 0.330 | 0.314 |

**Table 17.6-5**
Reinforcing Steel (in$^2$/ft) for Cantilever Deck Slabs with Tubular Railing Type “NY”/“M”
Girder Type 2

Notes:

1. Tables show the total area of transverse deck reinforcement required per foot.

2. The values in Table 17.6-2, Table 17.6-3, Table 17.6-4 and Table 17.6-5 are based on the following design criteria:

   - f'c = 4 ksi
   - fy = 60 ksi
   - Top steel clearance = 2 1/2"
   - Effective Overhang as illustrated in Figure 17.6-1

3. For Tubular Railing Type "NY"/"M", the No. 6 "U" bars located at the rail post locations should not be included when calculating the total available area of reinforcement.

4. The values in the shaded region are satisfied by the standard transverse reinforcement for all girder spacings and standard transverse deck reinforcement. No additional checks or reinforcement are required.

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

6. For bridge decks with raised sidewalks according to Standard Detail 17.01, the additional reinforcement shown in Table 17.6-2, Table 17.6-3, Table 17.6-4, and Table 17.6-5, need not be used. See Standard Detail 17.01 – Median and Raised Sidewalk Details – for information pertaining to the additional reinforcement to be used at raised sidewalks. For bridge decks with raised sidewalks according to Standard Detail 30.41, the additional reinforcement shown in Table 17.6-2 and Table 17.6-3, is required.

Example Use of Tables:

Given Information:

54W" PSG, 15" from CL girder to Design Section -- (Girder Type 2)

Girder Spacing = 7'-0"

Overhang = 3'-0", Effective Overhang = 1'-9"

Type "NY" rail

From Table 17.5-1:

Deck thickness = 8"

Design Section at 15", use #5's @ 8.5", As provided = 0.43 in$^2$/ft

From Table 17.6-5:

## 17.8 Bridge Deck Protective Systems

### 17.8.1 General

FHWA encourages states that require the use of de-icers to employ bridge deck protective systems. The major problem resulting in bridge deck deterioration is delamination of the concrete near the top mat of the reinforcing steel followed by subsequent spalling of the surface concrete. Research shows that the most prevalent cause of extensive deck deterioration is corrosion of the reinforcing steel due to the intrusion of chlorides into the concrete from repeated de-icer applications during snow and/or ice removal.

Several types of bridge deck protective systems are currently available. Some have been approved by FHWA based on their initial performance. Some of the more common types of protective systems are epoxy coated reinforcing steel, galvanized or stainless steel reinforcing steel, microsilica modified concrete or polymer impregnated concrete, cathodic protection and deck sealers. Epoxy coated reinforcing steel and deck sealers are preferred by WisDOT.

Structures other than box culverts that are designed to carry an earth fill are required to have waterproofing membrane systems on the deck to protect the slab. This includes bridges designed for future grade changes.

### 17.8.2 Design Guidance

All deck reinforcement bars shall be epoxy coated and the top reinforcing bars shall have a minimum of 2 ½ inches of cover.

All decks shall receive an initial protective deck seal. This includes all deck, sidewalk, median, paving notch, and concrete overlay surfaces. For decks with open rails, the deck seal shall wrap around the edge of deck and include 1'-0" underneath the deck. A pigmented seal shall be used on the top and inside faces of parapets. After the initial deck seal, decks shall be resealed at regular intervals or receive a thin polymer overlay as described in Chapter 40 – Bridge Rehabilitation. Refer to the Standard drawing in Chapter 17 – Superstructure-General for additional information.

Additional protective systems may be desired to minimize future rehabilitations. One or a combination of systems may be used on large projects such as Mega Projects. Contact the WisDOT Bureau of Structures Design Section for approval and project specific guidance. The following systems are currently being used and should be considered on new structures and deck rehabilitations:

- High Performance Concrete (HPC) – This is typically used within the bridge superstructure (deck, diaphragms, parapets, structural approach slabs, etc.) on urban interchange projects. HPC structures with a design speed of 40 mph or greater shall use bid item "Longitudinal Grooving", unless directed otherwise. Longitudinal grooving improves the curing process, reduces tire noise, and restores friction. Groove surfaces prior to opening the bridge to traffic. If a polymer overlay will be placed on an HPC structure prior to opening to traffic, then longitudinal grooving can be eliminated.

- Polymer overlays - This system extends the decks service life before rehabilitation is required. Refer to Chapter 40 for additional information.

- Stainless steel deck reinforcement – Use of stainless steel in lieu of epoxy bars may be justified for urban interchange projects and complex structures. Savings from reducing the number of rehabilitation projects and user costs can be substantial. Currently, only the enhanced corrosion protection benefits shall be utilized and reinforcement shall be selected per the epoxy coated deck design tables. The use of stainless reinforcing steel shall be approved by Bureau of Structures Development Section Chief or Design Section Chief and may require a life cycle analysis.

- Alternative reinforcements – Use of alternative materials are being evaluated for WisDOT applications. This includes chromium reinforcing bars, textured epoxy coated reinforcing bars, glass fiber reinforced polymer (GFRP) reinforcing bars, and galvanized reinforcing bars. Contact the Bureau of Structures Development Section Chief if considering use of alternative reinforcements.

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# 18.1 Introduction

## 18.1.1 General

This chapter considers the following types of concrete structures:

- Flat Slab
- Haunched Slab

A longitudinal slab is one of the least complex types of bridge superstructures. It is composed of a single element superstructure in comparison to the two elements of the transverse slab on girders or the three elements of a longitudinal slab on floor beams supported by girders. Due to simplicity of design and construction, the concrete slab structure is relatively economical. Its limitation lies in the practical range of span lengths and maximum skews for its application. For longer span applications, the dead load becomes too high for continued economy. Application of the haunched slab has increased the practical range of span lengths for concrete slab structures.

## 18.1.2 Limitations

Concrete slab structure types are not recommended over streams where the normal water freeboard is less than 4 feet; formwork removal requires this clearance. When spans exceed 35 feet, freeboard shall be increased to 5 feet above normal water.

All concrete slab structures are limited to a maximum skew of 30 degrees. Slab structures with skews in excess of 30 degrees, require analysis of complex boundary conditions that exceed the capabilities of the present design approach used in the Bureau of Structures.

Continuous span slabs are to be designed using the following pier types:

- Piers with pier caps (on columns or shafts)
- Wall type piers

These types will allow for ease of future superstructure replacement. Piers that have columns without pier caps, have had the columns damaged during superstructure removal. This type of pier will not be allowed without the approval of the Structures Design Section.

**WisDOT policy item:**

Slab bridges, due to camber required to address future creep deflection, do not ride ideally for the first few years of their service life and present potential issues due to ponding. As such, if practical (e.g. not excessive financial implications), consideration of other structure types should be given for higher volume/higher speed facilities, such as the Interstate. Understanding these issues, the Regions have the responsibility to make the final decision on structure type with respect to overall project cost, with BOS available for consultation.

dead load deflection. Most of the excess camber is dissipated during the first year of service, which is the time period that the majority of creep and shrinkage deflection occurs. Noticeable excess deflection or structure sag can normally be attributed to falsework settlement. Use modulus of elasticity $E_c = 3800$ ksi, see 18.2.2 . The dead load deflection, $\Delta_{DL}$ , shall be calculated using factored loads described in 18.3.4.1 and 18.4.2. The factored resistance, $R_r$ , is described in 18.3.4.2.3.

**WisDOT exception to AASHTO:**

Calculating full camber as three times the dead load deflection, as stated in paragraph above, is an exception to **LRFD [5.6.3.5.2]**. This exception, used by the Bureau of Structures, is based on field observations using this method.

Then check that, $\Delta_{DL} \leq R_r$ is satisfied.

A “Camber and Slab Thickness Diagram”, “Top of Slab Elevations” table and “Survey Top of Slab Elevations” table are to be shown on the plans. See Standard 18.03 for details.

Simple-Span Concrete Slabs:

Maximum allowable camber for simple-span slabs is limited to 2 ½ inches. For simple-span slabs, Bureau of Structures practice indicates that using a minimum slab depth (ft) from the equation 1.1(S + 10) / 30, (where S is span length in feet), and meeting the live load deflection and dead load deflection (camber) limits stated in this section, provides an adequate slab section for most cases.

**WisDOT exception to AASHTO:**

The equation for calculating minimum slab depth for simple-spans, as stated in paragraph above, is an exception to **LRFD [Table 2.5.2.6.3-1]**. This exception, used by the Bureau of Structures, is based on past performance using this equation.

Continuous-Span Concrete Slabs:

Maximum allowable camber for continuous-span slabs is 1 ¾ inches.

Note: For buried structures, include dead load deflections from the permanent earth cover (overburden) and update the “Camber and Slab Thickness Diagram” note accordingly. An exception to the maximum allowable camber criteria may be given by the Bureau of Structures for buried structures.

## 18.4.5 Live Load Distribution

Live loads are distributed over an equivalent width, E, as calculated below. The equivalent distribution width applies for both live load moment and shear.

### 18.4.5.1 Interior Strip

Equivalent interior strip widths for slab bridges are covered in **LRFD [4.6.2.1.2, 4.6.2.3]**.

The live loads to be placed on these widths are axle loads (i.e., two lines of wheels) and the full lane load.

Single-Lane Loading: $E = 10.0 + 5.0\,(L_1\,W_1)^{1/2}$

Multi-Lane Loading: $E = 84.0 + 1.44(L_1\,W_1)^{1/2} \leq 12.0(W)/N_L$

Where:

| | | |
|---|---|---|
| $E$ | = | equivalent distribution width (in) |
| $L_1$ | = | modified span length taken equal to the lesser of the actual span or 60.0 ft (ft) |
| $W_1$ | = | modified edge to edge width of bridge taken to be equal to the lesser of the actual width or 60.0 ft for multi-lane loading, or 30.0 ft for single-lane loading (ft) |
| $W$ | = | physical edge to edge width of bridge (ft) |
| $N_L$ | = | number of design lanes as specified in **LRFD [3.6.1.1.1]** |

#### 18.4.5.1.1 Strength and Service Limit State

Use the smaller equivalent width (single-lane or multi-lane), when (HL-93) live load is to be distributed, for Strength I Limit State and Service I Limit State.

The distribution factor, DF, is computed for a design slab width equal to one foot.

$$DF = \frac{1}{E}$$

Where:

| | | |
|---|---|---|
| $E$ | = | equivalent distribution width (ft) |

The multiple presence factor, m, has been included in the equations for distribution width, E, and therefore aren't used to adjust the distribution factor, DF, **LRFD [3.6.1.1.2]**.

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.

#### 18.4.5.1.2 Fatigue Limit State

Use equivalent widths from single-lane loading to check fatigue stress range criteria. For the Fatigue Limit State only one design truck (Fatigue Truck) is present **LRFD [3.6.1.4]**. 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, **LRFD [3.6.1.1.2]**.

The distribution factor, DF, is computed for a design slab width equal to one foot.

$$DF = \frac{1}{E(1.20)}$$

Where:

E = equivalent distribution width (ft)

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.

### 18.4.5.2 Exterior Strip

Equivalent exterior strip widths for slab bridges are covered in **LRFD [4.6.2.1.4]**.

<u>For Exterior Strips without Raised Sidewalks</u>:

The exterior strip width, E, is assumed to carry one wheel line and a tributary portion of design lane load (located directly over the strip width) as shown in Figures 17.2-7 and 17.2-9.

E equals the distance between the edge of the slab and the inside face of the barrier, plus 12 inches, plus ¼ of the full strip width specified in **LRFD [4.6.2.3]**.

The exterior strip width, E, shall not exceed either ½ the full strip width or 72 inches.

Use the smaller equivalent width (single-lane or multi-lane), for full strip width, when (HL-93) live load is to be distributed, for Strength I Limit State and Service I Limit State.

The multiple presence factor, m, has been included in the equations for full strip width and therefore aren't used to adjust the distribution factor **LRFD [3.6.1.1.2]**.

<u>For Exterior Strips with Raised Sidewalks</u>:

The exterior strip width, E, is to carry a tributary portion of design lane load (when its located directly over the strip width) as in Live Load Case 1 or one wheel line as in Live Load Case 2, as shown in Figure 17.2-11.

The exterior strip width, E, shall be 72 inches.

#### 18.4.5.2.1 Strength and Service Limit State

The distribution factor, DF, is computed for a design slab width equal to one foot.

Compute the distribution factor associated with one truck wheel line, to be applied to axle loads:

$$DF = \frac{(1\text{ wheel line})}{(2\text{ wheel lines/lane})(E)}$$

Where:

E = equivalent distribution width (ft)

Look at the distribution factor (for axle loads) calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Compute the distribution factor associated with tributary portion of design lane load, to be applied to full lane load: **LRFD [3.6.1.2.4]**

$$DF = \frac{\left[\dfrac{(SWL)}{(10\text{ ft lane load width})}\right]}{(E)}$$

Where:

E = equivalent distribution width (ft)

SWL = Slab Width Loaded (with lane load) (ft) $\geq$ 0.

E – (distance from edge of slab to inside face of barrier) or

E – (distance from edge of slab to inside face of raised sidewalk)

Look at the distribution factor (for lane load) calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

### 18.4.6 Longitudinal Slab Reinforcement

The concrete cover on the top bars is 2 ½ inches, which includes a ½ inch wearing surface. The bottom bar cover is 1 ½ inches. Minimum clear spacing between adjacent longitudinal bars is 3 ½ inches. The maximum center to center spacing of adjacent bars shall not exceed 1.5 times the thickness of the slab or 18.0 inches **LRFD [5.10.3.2]**. When bundled bars are used, see **LRFD [5.10.3.1.5, 5.10.8.2.3, 5.10.8.4.2a]**.

### 18.4.6.1 Design for Strength

Strength Limit State considerations and assumptions are detailed in **LRFD [5.5.4, 5.6.2]**.

The area of longitudinal slab reinforcement, $A_s$ , should be designed for strength at maximum moment locations along the structure, and for haunched slab structures, checked for strength at the haunch/slab intercepts. The area should also be checked for strength at bar reinforcement cutoff locations. This reinforcement should be designed for interior and exterior strips (edge beams) in both positive and negative moment regions. The reinforcement in the exterior strip is always equal to or greater than that required for the slab in an interior strip. Compare the reinforcement to be used for each exterior strip and select the strip with the largest amount of reinforcement ($in^2/ft$). Use this reinforcement pattern for both exterior strips to keep the bar layout symmetrical. Concrete parapets, curbs, sidewalks and other appurtenances are not to be considered to provide strength to the edge beam **LRFD [9.5.1]**. The total factored moment, $M_u$ , shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. Then calculate the coefficient of resistance, $R_u$ :

$$R_u = M_u / \phi \, b \, d_s^2$$

Where:

| | | |
|---|---|---|
| $\phi$ | = | 0.90 (see 18.3.3.2) |
| b | = | 12 in (for a 1 foot design slab width) |
| $d_s$ | = | slab depth (excl. ½ inch wearing surface) – bar clearance – ½ bar diameter (in) |

Calculate the reinforcement ratio, $\rho$, using ($R_u$ vs. $\rho$) Table 18.4-3 .

Then calculate required area,

$$A_s = \rho \, (b) \, (d_s)$$

Area of bar reinforcement per foot of slab width can be found in Table 18.4-4 .

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

Then check that, $M_u \leq M_r$ is satisfied.

The area of longitudinal reinforcement, $A_s$ , should also be checked for moment capacity (factored resistance) along the structure, to make sure it can handle factored moments due to applied dead load (including future wearing surface) and the Wisconsin Standard Permit Vehicle (Wis-SPV) (with a minimum gross vehicle load of 190 kips) on an interior strip. This requirement is stated in 17.1.2.1. See Chapter 45 for details on checking the capacity of the structure for this Permit Vehicle.

## 18.4.6.2 Check for Fatigue

Fatigue Limit State considerations and assumptions are detailed in **LRFD [5.5.3, 5.6.1, 9.5.3]**

The area of longitudinal slab reinforcement, $A_s$ , should be checked for fatigue stress range at locations where maximum stress range occurs along the structure, and for haunched slab structures, checked at the haunch/slab intercepts. The area should also be checked for fatigue stress range at bar reinforcement cutoff locations using Fatigue I Limit State. Check the reinforcement in an interior strip, where the largest number of fatigue cycles will occur.

Fatigue life of reinforcement is reduced by increasing the maximum stress level, bending of the bars and splicing of reinforcing bars by welding.

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

In regions of compressive stress due to unfactored permanent loads, fatigue shall be considered only if this compressive stress is less than 1.75 times the maximum tensile live load stress from the fatigue truck. The section properties for fatigue investigations shall be based on cracked sections where the sum of stresses, due to unfactored permanent loads, and 1.75 times the fatigue load is tensile and exceeds $0.095\ \lambda\ (f'_c)^{1/2}$ ; $\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8].**

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

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

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

## 18.4.6.3 Check for Crack Control

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

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

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

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

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

**LRFD [5.6.7]**

in which:

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

Where:

| | | |
|---|---|---|
| $\gamma_e$ | = | 1.00 for Class 1 exposure condition (bottom reinforcement) |
| $\gamma_e$ | = | 0.75 for Class 2 exposure condition (top reinforcement) |
| $d_c$ | = | thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto, (in). For top reinforcement, $d_c$ , should not include the ½” wearing surface |
| $f_{ss}$ | = | tensile stress in steel reinforcement (ksi) $\leq$ 0.6$f_y$; use factored loads described in 18.3.4.1 at the Service I Limit State, to calculate ($f_{ss}$) |
| h | = | overall depth of the section (in) |

### 18.4.6.4 Minimum Reinforcement Check

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

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

$M_{cr}$ (or) 1.33 $M_u$

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

Where:

| | | |
|---|---|---|
| $f_r$ | = | $0.24\ \lambda\ (f'_c)^{1/2}$ modulus of rupture (ksi) **LRFD [5.4.2.6]** |
| $\gamma_1$ | = | 1.6 flexural cracking variability factor |
| $\gamma_3$ | = | 0.67 ratio of minimum yield strength to ultimate tensile strength; for A615 Grade 60 reinforcement |
| $I_g$ | = | gross moment of Inertia ($in^4$) |

$c$ = effective slab thickness/2 (in)

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

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

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

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

Then check that, $M_L \leq M_r$ is satisfied.

## 18.4.6.5 Bar Cutoffs

One-half of the bar steel reinforcement required for maximum moment can be cut off at a point, where the remaining one-half has the moment capacity, or factored resistance, $M_r$ , equal to the total factored moment, $M_u$ , at that point. This is called the theoretical cutoff point.

Select tentative cutoff point at theoretical cutoff point or at a distance equal to the development length from the point of maximum moment, whichever is greater. The reinforcement is extended beyond this tentative point for a distance equal to the effective depth of the slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. This cutoff point is acceptable, if it satisfies fatigue and crack control criteria. The continuing bars must be fully developed at this point **LRFD [5.10.8.1.2a]**.

### 18.4.6.5.1 Positive Moment Reinforcement

At least one-third of the maximum positive moment reinforcement in simple-spans and one-fourth of the maximum positive moment reinforcement in continuous-spans is extended along the same face of the slab beyond the centerline of the support **LRFD [5.10.8.1.2b]**.

### 18.4.6.5.2 Negative Moment Reinforcement

For negative moment reinforcement, the second tentative cutoff point is at the point of inflection. At least one-third of the maximum negative moment reinforcement must extend beyond this point for a distance equal to the effective depth of the slab, 12 bar diameters, or 1/16 of the clear span, whichever is greater **LRFD [5.10.8.1.2c]**.

## 18.4.7 Transverse Slab Reinforcement

### 18.4.7.1 Distribution Reinforcement

Distribution reinforcement is placed transversely in the bottom of the slab, to provide for lateral distribution of concentrated loads **LRFD [5.12.2.1]**. The criteria for main reinforcement parallel to traffic is applied. The amount of distribution reinforcement is to be determined as a percentage of the main reinforcing steel required for positive moment as given by the following formula:

$$\text{Percentage} = \frac{100\%}{\sqrt{L}} \leq \; 50\% \text{ maximum}$$

Where:

L = span length (ft)

The above formula is conservative when applied to slab structures. This specification was primarily drafted for the relatively thin slabs on stringers.

### 18.4.7.2 Reinforcement in Slab over Piers

If the concrete superstructure rests on a pier cap (with columns) or directly on columns, design of transverse slab reinforcement over the pier is required. A portion of the slab over the pier is designed as a continuous transverse slab member (beam) along the centerline of the substructure. The depth of the assumed section is equal to the depth of the slab or haunch when the superstructure rests directly on columns. When the superstructure rests on a pier cap and the transverse slab member and pier cap act as a unit, the section depth will include the slab or haunch depth plus the cap depth. For a concrete slab, the width of the transverse slab member is equal to one-half the center to center spacing between columns (or 8 foot maximum) for the positive moment zone. The width equals the diameter of the column plus 6 inches for negative moment zone when no pier cap is present. The width equals the cap width for negative moment zone when a pier cap is present. Reference is made to the design example in 18.5 of this chapter for computations relating to transverse reinforcement in slab over the piers.

### 18.4.8 Shrinkage and Temperature Reinforcement

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperature changes and in structural mass concrete.

The area, $A_s$ , of reinforcement per foot for shrinkage and temperature effects, on each face and in each direction shall satisfy: **LRFD [5.10.6]**

$$A_s \geq 1.30\,(b)\,(h) / 2\,(b+h)\,(f_y) \quad \text{and} \quad 0.11 \leq A_s \leq 0.60$$

Where:

$A_s$ = area of reinforcement in each direction and on each face ($in^2/ft$)

b = least width of component section (in)

h = least thickness of component section (in)

$f_y$ = specified yield strength of reinforcing bars (ksi) $\leq$ 75 ksi

Shrinkage and temperature reinforcement shall not be spaced farther apart than 3.0 times the component thickness or 18 inches. For components greater than 36 inches thick, the spacing shall not exceed 12 inches.

All longitudinal reinforcement and transverse reinforcement in the slab must exceed required $A_s$ (on each face and in each direction), and not exceed maximum spacing.

## 18.4.9 Shear Check of Slab

Slab bridges designed for dead load and (HL-93) live load moments in conformance with **LRFD [4.6.2.3]** may be considered satisfactory in shear **LRFD [5.12.2.1]**.

## 18.4.10 Longitudinal Reinforcement Tension Check

The tensile capacity check of longitudinal reinforcement on the flexural tension side of a member is detailed in **LRFD [5.7.3.5]**.

The area of longitudinal reinforcement (in bottom of slab), $A_s$ , should be checked for tensile capacity at the abutments, for dead load and (HL-93) live load on interior and exterior strips. The reinforcement at these locations shall have the capacity to resist the tension in the reinforcement produced by shear.

The factored shear, $V_u$ , shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. The factored tension force, $T_{fact}$ , from shear, to be resisted is from **LRFD [Eq'n. 5.7.3.5-2]**, where $V_s = V_p = 0$, is:

$$T_{fact} = [\ V_u / \phi_v\ ] \cot \theta$$

Assume a diagonal crack would start at the inside edge of the bearing area. Assume the crack angle, $\theta$, is 35 degrees. Calculate the distance from the bottom of slab to center of tensile reinforcement. Determine the distance $D_{crack}$ from the end of the slab to the point at which the diagonal crack will intersect the bottom longitudinal reinforcement. Find the development length, $\ell_d$ , from Table 9.9-2, Chapter 9.

The nominal tensile resistance, $T_{nom}$ , of the longitudinal bars at the crack location is:

$$T_{nom} = A_s f_y [\ D_{crack} - (\text{end cover})] / \ell_d \leq A_s f_y$$

Then check that, $T_{fact} \leq T_{nom}$ is satisfied.

If the values for $T_{fact}$ and $T_{nom}$ are close, the procedure for determining the crack angle, $\theta$, as outlined in **LRFD [5.7.3.4.2]** should be used.

## 18.4.11 Uplift Check

Check for uplift at the abutments for (HL-93) live loads **LRFD [C3.4.1, 5.5.4.3]**. Compare the factored dead load reaction to the factored live load reaction. The reactions shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. Place (HL-93) live loads in each design lane **LRFD [3.6.1.1.1]** and apply a multiple presence factor **LRFD [3.6.1.1.2]**.

## 18.4.12 Deflection Joints and Construction Joints

The designer should locate deflection joints in sidewalks and parapets on concrete slab structures according to the Standard *Vertical Face Parapet 'A'* in Chapter 30.

Refer to Standards *Continuous Haunched Slab* and *Continuous Flat Slab* in Chapter 18, for recommended construction joint guidelines.

## 18.4.13 Reinforcement Tables

Table 18.4-3 applies to: Rectangular Sections with Tension Reinforcement only

- Reinforcement Yield Strength ($f_y$) = 60,000 psi
- Concrete Compressive Strength ($f'_c$) = 4,000 psi

| $R_u$ | $\rho$ | $R_u$ | $\rho$ | $R_u$ | $\rho$ | $R_u$ | $\rho$ | $R_u$ | $\rho$ |
|---|---|---|---|---|---|---|---|---|---|
| 117.9 | 0.0020 | 335.6 | 0.0059 | 537.1 | 0.0098 | 722.6 | 0.0137 | 892.0 | 0.0176 |
| 123.7 | 0.0021 | 340.9 | 0.0060 | 542.1 | 0.0099 | 727.2 | 0.0138 | 896.1 | 0.0177 |
| 129.4 | 0.0022 | 346.3 | 0.0061 | 547.1 | 0.0100 | 731.7 | 0.0139 | 900.2 | 0.0178 |
| 135.2 | 0.0023 | 351.6 | 0.0062 | 552.0 | 0.0101 | 736.2 | 0.0140 | 904.4 | 0.0179 |
| 141.0 | 0.0024 | 357.0 | 0.0063 | 556.9 | 0.0102 | 740.7 | 0.0141 | 908.5 | 0.0180 |
| 146.7 | 0.0025 | 362.3 | 0.0064 | 561.8 | 0.0103 | 745.2 | 0.0142 | 912.5 | 0.0181 |
| 152.4 | 0.0026 | 367.6 | 0.0065 | 566.7 | 0.0104 | 749.7 | 0.0143 | 916.6 | 0.0182 |
| 158.1 | 0.0027 | 372.9 | 0.0066 | 571.6 | 0.0105 | 754.2 | 0.0144 | 920.7 | 0.0183 |
| 163.8 | 0.0028 | 378.2 | 0.0067 | 576.5 | 0.0106 | 758.7 | 0.0145 | 924.8 | 0.0184 |
| 169.5 | 0.0029 | 383.5 | 0.0068 | 581.4 | 0.0107 | 763.1 | 0.0146 | 928.8 | 0.0185 |
| 175.2 | 0.0030 | 388.8 | 0.0069 | 586.2 | 0.0108 | 767.6 | 0.0147 | 932.8 | 0.0186 |
| 180.9 | 0.0031 | 394.1 | 0.0070 | 591.1 | 0.0109 | 772.0 | 0.0148 | 936.9 | 0.0187 |
| 186.6 | 0.0032 | 399.3 | 0.0071 | 595.9 | 0.0110 | 776.5 | 0.0149 | 940.9 | 0.0188 |
| 192.2 | 0.0033 | 404.6 | 0.0072 | 600.8 | 0.0111 | 780.9 | 0.0150 | 944.9 | 0.0189 |
| 197.9 | 0.0034 | 409.8 | 0.0073 | 605.6 | 0.0112 | 785.3 | 0.0151 | 948.9 | 0.0190 |
| 203.5 | 0.0035 | 415.0 | 0.0074 | 610.4 | 0.0113 | 789.7 | 0.0152 | 952.9 | 0.0191 |
| 209.1 | 0.0036 | 420.2 | 0.0075 | 615.2 | 0.0114 | 794.1 | 0.0153 | 956.8 | 0.0192 |
| 214.8 | 0.0037 | 425.4 | 0.0076 | 620.0 | 0.0115 | 798.4 | 0.0154 | 960.8 | 0.0193 |
| 220.4 | 0.0038 | 430.6 | 0.0077 | 624.8 | 0.0116 | 802.8 | 0.0155 | 964.7 | 0.0194 |
| 225.9 | 0.0039 | 435.8 | 0.0078 | 629.5 | 0.0117 | 807.2 | 0.0156 | 968.7 | 0.0195 |
| 231.5 | 0.0040 | 441.0 | 0.0079 | 634.3 | 0.0118 | 811.5 | 0.0157 | 972.6 | 0.0196 |
| 237.1 | 0.0041 | 446.1 | 0.0080 | 639.0 | 0.0119 | 815.8 | 0.0158 | 976.5 | 0.0197 |
| 242.7 | 0.0042 | 451.3 | 0.0081 | 643.8 | 0.0120 | 820.1 | 0.0159 | 980.4 | 0.0198 |
| 248.2 | 0.0043 | 456.4 | 0.0082 | 648.5 | 0.0121 | 824.5 | 0.0160 | 984.3 | 0.0199 |
| 253.7 | 0.0044 | 461.5 | 0.0083 | 653.2 | 0.0122 | 828.8 | 0.0161 | 988.2 | 0.0200 |
| 259.3 | 0.0045 | 466.6 | 0.0084 | 657.9 | 0.0123 | 833.1 | 0.0162 | 992.1 | 0.0201 |
| 264.8 | 0.0046 | 471.7 | 0.0085 | 662.6 | 0.0124 | 837.3 | 0.0163 | 996.0 | 0.0202 |
| 270.3 | 0.0047 | 476.8 | 0.0086 | 667.3 | 0.0125 | 841.6 | 0.0164 | 999.8 | 0.0203 |
| 275.8 | 0.0048 | 481.9 | 0.0087 | 671.9 | 0.0126 | 845.9 | 0.0165 | 1003.7 | 0.0204 |
| 281.3 | 0.0049 | 487.0 | 0.0088 | 676.6 | 0.0127 | 850.1 | 0.0166 | 1007.5 | 0.0205 |
| 286.8 | 0.0050 | 492.1 | 0.0089 | 681.3 | 0.0128 | 854.3 | 0.0167 | 1011.3 | 0.0206 |
| 292.2 | 0.0051 | 497.1 | 0.0090 | 685.9 | 0.0129 | 858.6 | 0.0168 | 1015.1 | 0.0207 |
| 297.7 | 0.0052 | 502.2 | 0.0091 | 690.5 | 0.0130 | 862.8 | 0.0169 | 1018.9 | 0.0208 |
| 303.1 | 0.0053 | 507.2 | 0.0092 | 695.1 | 0.0131 | 867.0 | 0.0170 | 1022.7 | 0.0209 |
| 308.6 | 0.0054 | 512.2 | 0.0093 | 699.7 | 0.0132 | 871.2 | 0.0171 | 1026.5 | 0.0210 |
| 314.0 | 0.0055 | 517.2 | 0.0094 | 704.3 | 0.0133 | 875.4 | 0.0172 | 1030.3 | 0.0211 |
| 319.4 | 0.0056 | 522.2 | 0.0095 | 708.9 | 0.0134 | 879.5 | 0.0173 | 1034.0 | 0.0212 |
| 324.8 | 0.0057 | 527.2 | 0.0096 | 713.5 | 0.0135 | 883.7 | 0.0174 | 1037.8 | 0.0213 |
| 330.2 | 0.0058 | 532.2 | 0.0097 | 718.1 | 0.0136 | 887.9 | 0.0175 | ---- | ---- |

**Table 18.4-3**

$R_u$ (psi) vs. $\rho$

$R_u$ = coefficient of resistance (psi) = $M_u / \phi\ b\ d_s^2$ $\rho$ = reinforcement ratio = $A_s / b\ d_s$

Table 18.4-4 can be used to select bar size and bar spacing to provide an adequate area of reinforcement to meet design requirements.

| Bar Size Number | Nominal Dia. Inches | 4 1/2" | 5" | 5 1/2" | 6" | 6 1/2" | 7" | 7 1/2" | 8" | 8 1/2" | 9" | 10" | 12" |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 4 | 0.500 | 0.52 | 0.47 | 0.43 | 0.39 | 0.36 | 0.34 | 0.31 | 0.29 | 0.28 | 0.26 | 0.24 | 0.20 |
| 5 | 0.625 | 0.82 | 0.74 | 0.67 | 0.61 | 0.57 | 0.53 | 0.49 | 0.46 | 0.43 | 0.41 | 0.37 | 0.31 |
| 6 | 0.750 | 1.18 | 1.06 | 0.96 | 0.88 | 0.82 | 0.76 | 0.71 | 0.66 | 0.62 | 0.59 | 0.53 | 0.44 |
| 7 | 0.875 | 1.60 | 1.44 | 1.31 | 1.20 | 1.11 | 1.03 | 0.96 | 0.90 | 0.85 | 0.80 | 0.72 | 0.60 |
| 8 | 1.000 | 2.09 | 1.88 | 1.71 | 1.57 | 1.45 | 1.35 | 1.26 | 1.18 | 1.11 | 1.05 | 0.94 | 0.79 |
| 9 | 1.128 | -- | 2.40 | 2.18 | 2.00 | 1.85 | 1.71 | 1.60 | 1.50 | 1.41 | 1.33 | 1.20 | 1.00 |
| 10 | 1.270 | -- | 3.04 | 2.76 | 2.53 | 2.34 | 2.17 | 2.02 | 1.90 | 1.79 | 1.69 | 1.52 | 1.27 |
| 11 | 1.410 | -- | 3.75 | 3.41 | 3.12 | 2.88 | 2.68 | 2.50 | 2.34 | 2.21 | 2.08 | 1.87 | 1.56 |

**Table 18.4-4**
Area of Bar Reinf. ($in^2$ / ft) vs. Spacing of Bars (in)

## 18.5 Standard Concrete Slab Design Procedure

### 18.5.1 Local Bridge Improvement Assistance Program

The Local Bridge Program was established to rehabilitate and replace, on a cost-shared basis, the most seriously deteriorating local bridges on Wisconsin's local highway and road systems. Counties, cities, villages, and towns are eligible for bridge replacement funding in accordance with the requirements in Administrative Code Trans 213. As a part of the Local Bridge Replacement Program, BOS has developed a Standard Bridge Design Tool (SBDT) to efficiently design and draft single span concrete slab bridges.

More information on the Local Bridge Improvement Assistance Program can be found at the following link:

https://wisconsindot.gov/Pages/doing-bus/local-gov/astnce-pgms/highway/localbridge.aspx.

### 18.5.2 Selection of Applicable Projects

On a biennial basis, locals sponsors submit applications for prospective bridge replacement projects to the WisDOT Regional Offices. The BOS Design Section assists the Regional Local Program Managers with the reviews of the applications for the appropriateness of the requested estimated bridge replacement costs. At that time, the BOS Design Section will identify candidate bridges to utilize the SBDT to streamline the bridge replacement design process. Identification of candidate bridges is based on the existing structure size, configuration, inspection and maintenance history, and known stream characteristics and flood history.

Once projects are approved for funding, the WisDOT Local Program Managers reach out to local sponsors soliciting knowledge that would preclude the use of the SBDT on those individual projects that have been identified by the BOS Design Section as candidates. If sufficient information is presented, identifying issues that will preclude the use of the tool for an identified, candidate project; then the BOS Design Section will support the conventional bridge replacement design process. However, if sufficient information is not presented, then it is the expectation that the identified candidate projects will move forward into preliminary design with the assumption that the SBDT will be utilized.

### 18.5.3 Use Within Other Programs

While the main focus of the SBDT is on local program usage, there may also be projects on the state system that may benefit from its use. The BOS Design Section will look for opportunities within the structures certification process to identify candidate projects on the state system to utilize the SBDT.

## 18.5.4 Standard Bridge Design Tool

### 18.5.4.1 Requirements of Designer

While the SBDT will significantly increase the efficiency with which single span slab bridge designs and plans are completed, the consultant and in-house structure designers will continue to fulfill the critical function of preliminary structure design and layout. It is expected that a structure type alternatives analysis will continue to be completed in order to verify that a single span slab bridge is the most cost-effective structure type for each project location, and that the single span slab bridge meets all site design criteria and constraints. In the event that a box culvert can be utilized, significant consideration should be given to utilizing this structure type as it is generally a more economical structure type both from an initial cost and long-term maintenance standpoint. While there would be an increase in the design fees associated with not utilizing the SBDT to make this change, those would be far outweighed over the life of the structure.

Once the structure type is verified, the preliminary type, size, and location design; hydrology and hydraulic designs; and foundation support selection remain the responsibility of the consultant. When the preliminary design and analyses are complete, the SBDT can be used to assemble the preliminary plans for submittal to the BOS Consultant Review Unit for preliminary review following the guidelines included in 6.2 and 6.5. There are no changes to the preliminary structure e-submittal contents for projects utilizing the SBDT when compared to conventional projects.

After preliminary review comments are addressed, the full set of final bridge plans can be submitted to the BOS Consultant Review unit following the guidelines included in 6.3 and 6.5. Note that design computations are not required to be submitted to BOS with the final plans unless there is a unique design feature that is added to the bridge, separate from what is automatically compiled by the SBDT. For the final quantities submittal, only those quantities not automatically compiled by the SBDT need to be submitted for review. Additionally, for the special provisions submittal, only those that need to be added in unique cases need to be submitted. For example, if a wildlife corridor is requested within the riprap slope of a standard bridge plan, then that SPV should be included in the plans and submitted for review.

The following is a list of items that need to be submitted as a part of the final e-submittal to BOS for review:

- Final Structure Plans
- QA/QC Verification Sheet
- Inventory Data Sheet
- Quantity Computations (only those not assembled by the SBDT)
- Special Provisions (only those to be added to the SBDT generated bid items)

The following is a list of items that do not need to be submitted as a part of the final e-submittal to BOS for review:

- Design Computations (unless there is a unique design feature)
- Bridge Load Rating Summary Form

### 18.5.4.2 Location of Tool

The SBDT is a web-based application that can be found at the following location:

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

### 18.5.4.3 How to Utilize the Tool

The step-by-step user guide can be found at the following location:

https://wisconsindot.gov/Pages/doing-bus/local-gov/lpm/lp-standarized-bridge-plan-pilot.aspx.

## 18.6 Design Example

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## 24.5 Repetitive Loading and Toughness Considerations

*AASHTO LRFD* specifies requirements for repetitive loading and toughness considerations. Fatigue design and detail guidelines are provided, and material impact testing for fracture toughness is required. These requirements are based on performance evaluations over the past several decades on existing highways and bridges under the effects of repetitive vehicle loading.

The direct application of fatigue specifications to main load-carrying members has generally been apparent to most bridge designers. Therefore, main members have been designed with the appropriate details. However, fatigue considerations in the design of secondary members and connections have not always been so obvious. Many of these members interact with main members and receive more numerous cycles of load at a higher level of stress range than assumed. As a result, most of the fatigue problems surfacing in recent years have involved cracking initiated by secondary members.

### 24.5.1 Fatigue Strength

In *AASHTO LRFD*, fatigue is defined as the initiation and/or propagation of cracks due to repeated variation of normal stress with a tensile component. The fatigue life of a detail is defined as the number of repeated stress cycles that results in fatigue failure of a detail, and the fatigue design life is defined as the number of years that a detail is expected to resist the assumed traffic loads without fatigue cracking. In *AASHTO LRFD*, the fatigue design life is based on either Fatigue I for infinite load-induced fatigue life or Fatigue II for finite load-induced fatigue life.

**WisDOT Policy Item**

Only consider the Fatigue I limit state for steel design.

The main factors governing fatigue strength are the applied stress, the number of loading cycles and the type of detail. The designer has the option of either limiting the stress range to acceptable levels or choosing details which limit the severity of the stress concentrations.

Details involving connections that experience fatigue crack growth from weld toes and weld ends where there is high stress concentration provide the lowest allowable stress range. This applies to both fillet and groove welded details. Details which serve the intended function and provide the highest fatigue strength are recommended.

Generally, details involving failure from internal discontinuities such as porosity, slag inclusion, cold laps and other comparable conditions will have a high allowable stress range. This is primarily due to the fact that geometrical stress concentrations at such discontinuities do not exist, other than the effect of the discontinuity itself.

*AASHTO LRFD* provides the designer with eight basic design range categories for redundant and non-redundant load path structures. The stress range category is selected based on the highway type and the detail employed. The designer may wish to make reference to *Bridge Fatigue Guide Design and Details*, by John W. Fisher.

## 24.5.2 Charpy V-Notch Impact Requirements

Recognizing the need to prevent brittle fracture failures of main load-carrying structural components, AASHTO adopted provisions for Charpy V-Notch impact testing in 1974. Impact testing offers an important measure of material quality, particularly in terms of ductility. Brittleness is detected prior to placing the material in service to prevent member service failures. Wisconsin *Standard Specifications for Highway and Structure Construction* require Charpy V-Notch tests on all girder flange and web plates, flange splice plates, hanger bars, links, rolled beams and flange cover plates. Special provisions require higher Charpy V-Notch values for non-redundant structure types.

For the Charpy V-Notch impact test, small, notched steel specimens are loaded at very high strain rates as the specimen absorbs the impact from a pendulum. The maximum height the pendulum rises after impact measures the amount of energy absorbed in foot-pounds.

The AASHTO fracture control plan uses three different temperature zones (designated Zones 1, 2 and 3) to qualify the fracture toughness of bridge steels. The three zones are differentiated by their minimum operating (or service) temperatures, which are given in **LRFD [Table 6.6.2.1-2]**. In Wisconsin, use Zone 2 requirements.

Separate fracture toughness requirements are given in **LRFD [Table C6.6.2.1-1]** for load path redundant members (LPRMs), system redundant members (SRMs), and non-redundant steel tension members (NSTMs). A NSTM is a primary steel member fully or partially in tension, and without load path redundancy, system redundancy, or internal redundancy, whose failure may cause a portion of or the entire bridge to collapse. NSTMs and SRMs are subject to more stringent Charpy V-Notch fracture toughness requirements than LPRMs. For NSTMs and SRMs, High Performance Steel (HPS) shall be used with Zone 2 requirements.

According to **LRFD [6.6.2.2]**, the engineer has the responsibility to identify all bridge members or components that are NSTMs and clearly delineate their location on the contract plans. Examples of NSTMs in bridges include certain truss members in tension, suspension cables, tension components of girders in two-girder systems, pin and link systems in suspended spans, cross girders and welded tie girders in tied-arches. In addition, any attachment having a length in the direction of the tension stress greater than 4 inches and welded to the tension area of a component of a NSTM is also to be considered fracture critical.

## 24.5.3 Non-Redundant Type Structures

Previous AASHTO fatigue and fracture toughness provisions provided satisfactory fracture control for multi-girder structures when employed with good fabrication and inspection practices. However, concern existed that some additional factor of safety against the possibility of brittle fracture should be provided in the design of non-redundant type structures such as single-box and two-box girders, two-plate girders or truss systems where failure of a single element could cause collapse of the structure. A case in point was the collapse of the Point Pleasant Bridge over the Ohio River. HPS shall be used for non-redundant structures.

Primary factors controlling the susceptibility of non-redundant structures to brittle fracture are the material toughness, flaw size and stress level. One of the most effective methods of reducing brittle fracture is lowering the stress range imposed on the member. AASHTO provides an increased safety factor for non-redundant members by requiring a shift of one range of loading cycles for fatigue design with corresponding reduction of stress range for critical stress categories. The restrictive ranges for certain categories require the designer to investigate the use of details which do not fall in critical stress categories or induce brittle fracture. For LPRMs including bolted tie girders found in tied arch bridges, multiple box girder structures (3 boxes) and hanger plates, HPS shall also be used.

As per a FHWA directive, two-girder box girder structures are to be considered NSTMs unless adequate system redundancy has been demonstrated, in which case the members may be reclassified as SRMs per 24.15.

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

## 24.6 Design Approach - Steps in Design

### 24.6.1 Obtain Design Criteria

The first design step for a steel girder is to choose the correct design criteria. The design criteria include the following:

- Number of spans
- Span lengths
- Skew angles
- Number of girders
- Girder spacing
- Deck overhang
- Cross-frame spacing
- Flange and web yield strengths
- Deck concrete strength
- Deck reinforcement strength
- Deck thickness
- Dead loads
- Roadway geometry
- Haunch depth

For steel girder design, the following load combinations are generally considered:

- Strength I
- Service II
- Fatigue I

The extreme event limit state (including earthquake load) is generally not considered for a steel girder design.

The following steps are taken in determining the girder or beam spacing and the slab thickness:

1. The girder spacing (and the resulting number of girders) for a structure is determined by considering the desirable girder depth and the span lengths. Refer to 24.4.2 for design aids. Where depth or deflection limitations do not control the design, it is usually more economical to use fewer girders with a wider spacing and a thicker slab. Four girders are generally considered to be the minimum, and five girders are desirable to facilitate future redecking.

2. The slab overhang on exterior girders is limited to 3'-7" measured from the girder centerline to the edge of slab. The overhang is limited to prevent rotation and bending of the web during construction caused by the forming brackets. The overhang width is generally determined such that the moments and shears in the exterior girder are similar to those in the interior girder. In addition, the overhang is set such that the positive and negative moments in the deck slab are balanced. A common rule of thumb is to make the overhang approximately 0.28 to 0.5 times the girder spacing. For girders less than, or equal to 36-inches in depth, limit the overhang to the girder depth, and preferably no wider than 0.80 the girder depth. The limits for raised sidewalk overhangs on the Standard for *Median and Raised Sidewalk Details* are likely excessive for such shallow girders.

3. Check if a thinner slab and the same number of members can be used by slightly reducing the spacing.

## 24.6.2 Select Trial Girder Section

Before the dead load effects can be computed, a trial girder section must be selected. This trial girder section is selected based on previous experience and based on preliminary design. Based on this trial girder section, section properties and dead load effects will be computed. Then specification checks will be performed to determine if the trial girder section successfully resists the applied loads. If the trial girder section does not pass all specification checks or if the girder optimization is not acceptable, then a new trial girder section must be selected and the design process must be repeated.

The following tips are presented to help bridge designers in developing an economical steel girder for most steel girder designs. Other design tips are available in various publications from the *American Institute of Steel Construction (AISC)* and from steel fabricators.

- Girder depth – The minimum girder depth is specified in **LRFD [2.5.2.6.3]**. An estimate of the optimum girder depth can be obtained from trial runs using design software. The web depth may be varied by several inches more or less than the optimum without significant cost penalty. Refer to 24.4.2 for recommended girder depths for a given girder spacing and span length.

- Web thickness – A "nominally stiffened" web (approximately 1/16 inch thinner than "unstiffened") will generally provide the least cost alternative or very close to it. However, for web depths of approximately 50” or less, unstiffened webs may be more economical.

- Plate transitions – For rolled sections, a change in section should occur only at field splice locations. For plate girders, include the change in section at butt splices and check the maximum rolling lengths of plates to see if additional butt splices are required. The fabricator may assume the cost of extending the heavier plate and eliminating the butt splice; this option has been used by fabricators on numerous occasions. Shim plates are provided at the bearing to allow for either option. A common rule of thumb is to use no more than three plates (two shop splices) in the top or bottom flange of field sections up to 130 feet long. In some cases, a single flange plate size can be carried through the full length of the field section. Estimate field splice locations at approximately the 7/10 point of continuous spans.

- Flange widths – Flange widths should remain constant within field sections. The use of constant flange widths simplifies construction of the deck. The unsupported length in compression of the shipping piece divided by the minimum width of the compression flange in that piece should be less than approximately 85. High bearing reactions at the piers of continuous girders may govern the width of the bottom flange.

- Flange transitions – It is good design practice to reduce the flange cross-sectional area by no more than approximately one-half of the area of the heavier flange plate. This reduces the build-up of stress at the transition.

- Haunched girders – On haunched plate girders, the length of the parabolic haunch is approximately 1/4 of the span length. The haunch depth is 1 1/2 times the midspan depth.

It should be noted that during the optimization process, minor adjustments can be made to the plate sizes and transition locations without needing to recompute the analysis results. However, if significant adjustments are made, such that the moments and shears would change significantly, then a revised analysis is required.

### 24.6.3 Compute Section Properties

See 17.2.11 for determining composite slab width.

For a composite superstructure, several sets of section properties must be computed. The initial dead loads (or the non-composite dead loads) are applied to the girder-only section. The superimposed dead loads are applied to the composite section based on a modular ratio of 3n, as described in **LRFD [6.10.1.1.1]**. The live loads are applied to the composite section based on a modular ratio of n.

For girders with shear connectors provided throughout their entire length and with slab reinforcement satisfying the provisions of **LRFD [6.10.1.7]**, stresses due to loads applied to the composite section for the Fatigue I and Service II limit states may be computed using the short-term composite section, based on a modular ratio of n, assuming the concrete slab to be fully effective for both positive and negative flexure.

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.

For LRFD, Wisconsin places shear connectors in both the positive and negative moment regions of continuous steel girder bridges, and both regions are considered composite in analysis and design computations. Negative flexure concrete deck reinforcement is considered in the section property calculations.

## 24.6.4 Compute Dead Load Effects

The girder must be designed to resist the dead load effects, as well as the other load effects. Various types of dead loads and their corresponding load factors are described in 17.2.4 and 17.2.5.

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 analysis software. Since the actual plate sizes are entered as input, the moments and shears are computed based on the actual, varying plate sizes.

Distribution of dead load to the girders is described in 17.2.8.

The stiffness of the composite section is used for determining live load and composite dead load moments and shears. When computing live load values, the composite section is based on n, and when computing composite dead load values, the composite section is based on 3n. Non-composite dead load moments and shears are computed based on the stiffness of the non-composite steel section.

## 24.6.5 Compute Live Load Effects

The girder must also be designed to resist the live load effects. The live load consists of an HL-93 loading. Similar to the dead load, the live load moments and shears for an HL-93 loading can be obtained from an analysis computer program.

For all limit states other than fatigue and fracture, the dynamic load allowance, IM, is 0.33. 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.

Live load distribution factors must be computed as specified in **LRFD [4.6.2.2]**, as shown in Table 24.6-1.

**WisDOT Policy Item**

For beams with variable moment of inertia, the longitudinal stiffness parameter, Kg (**LRFD [Eq'n 4.6.2.2.1-1]**), shall be based on a weighted average of properties, over the entire length of the bridge.

In addition to computing the live load distribution factors, their ranges of applicability must also be checked. If they are not satisfied, then conservative assumptions must be made based on sound engineering judgment. Additional information about distribution of live load to the girders is presented in 17.2.8.

For skewed bridges, WisDOT does not consider skew correction factors for moment.

| Live Load Distribution Factor | AASHTO LRFD Reference |
|---|---|
| Moments in Interior Beams | **LRFD [Table 4.6.2.2.2b-1]** |
| Moments in Exterior Beams | **LRFD [C4.6.2.2.2d]** and **LRFD [Table 4.6.2.2.2d-1]** |
| Moment Reduction for Skew | Not Applicable for WisDOT |
| Shear in Interior Beams | **LRFD [Table 4.6.2.2.3a-1]** |
| Shear in Exterior Beams | **LRFD [C4.6.2.2.2d]** and **LRFD [Table 4.6.2.2.3b-1]** |
| Shear Correction for Skew | **LRFD [Table 4.6.2.2.3c-1]** |

**Table 24.6-1**
Live Load Distribution Factors

### 24.6.6 Combine Load Effects

The next step is to combine the load effects for each of the applicable limit states. Load effects are combined in accordance with **LRFD [Table 3.4.1-1]** and **LRFD [Table 3.4.1-2]**.

After combining load effects, the next ten design steps consist of verifying the structural adequacy of the steel girder using appropriate sections of *AASHTO LRFD*. For steel girder designs, specification checks are generally performed at the following locations:

- Span tenth points
- Locations of plate transitions
- Locations of stiffener spacing transitions

However, it should be noted that the maximum moment within a span may not necessarily occur at any of the above locations.

Check the loads of the interior and exterior members to see if one or both members are to be designed.

### 24.6.7 Check Section Property Limits

Several checks are required to ensure that the proportions of the girder section are within specified limits, as presented in **LRFD [6.10.2]**. The first section proportion check relates to the web slenderness, and the second set of section proportion checks relate to the general proportions of the section.

## 24.6.8 Compute Plastic Moment Capacity

For composite sections, the plastic moment, $M_p$, must be calculated as the first moment of plastic forces about the plastic neutral axis. The methodology for the plastic moment capacity computations is presented in **LRFD [Appendix D6.1]**.

## 24.6.9 Determine If Section is Compact or Non-compact

The next step in the design process is to determine if the section is compact or non-compact, as described in **LRFD [6.10.6.2.2]**. This, in turn, will determine which formulae should be used to compute the flexural capacity of the girder.

## 24.6.10 Design for Flexure – Strength Limit State

The next step is to compute the flexural resistance of the girder at each section. These computations vary, depending on whether the section is composite or non-composite, whether the section is compact or non-compact, and whether the section is in positive flexure or negative flexure. The following sections of *AASHTO LRFD* can be used:

- Compact, composite section in positive flexure – **LRFD [6.10.7.1]**
- Non-compact, composite section in positive flexure – **LRFD [6.10.7.2]**
- Composite sections in negative flexure – **LRFD [6.10.8]**
- Non-composite sections – **LRFD [6.10.8]**

**WisDOT Policy Item:**

Do not utilize optional **LRFD [Appendix B6]** for Moment Redistribution from Interior-Pier I-Sections in Straight Continuous-Span Bridges.

## 24.6.11 Design for Shear

Shear must be checked at each section of the girder. However, shear is generally maximum at or near the supports.

The first step in the design for shear is to check if the web must be stiffened. A "nominally stiffened" web (approximately 1/16 inch thinner than "unstiffened") will generally provide the least cost alternative or very close to it. However, for web depths of approximately 50 inches or less, unstiffened webs may be more economical.

It should be noted that in end panels, the shear is limited to either the shear yield or shear buckling in order to provide an anchor for the tension field in adjacent interior panels. Tension field is not allowed in end panels. The design procedure for shear in the end panel is presented in **LRFD [6.10.9.3.3]**.

## 24.6.12 Design Transverse Intermediate Stiffeners and/or Longitudinal Stiffeners

If transverse intermediate stiffeners and/or longitudinal stiffeners are used, they must be designed. The design of transverse intermediate stiffeners is described in 24.10, and the design of longitudinal stiffeners is described in 24.11.

## 24.6.13 Design for Flexure – Fatigue and Fracture

Load-induced fatigue must be considered in a steel girder design. Fatigue considerations may include:

- Welds connecting the shear studs to the girder
- Welds connecting the flanges and the web
- Welds connecting stiffeners to the girder

The specific fatigue considerations depend on the unique characteristics of the girder design. Specific fatigue details and detail categories are explained and illustrated in **LRFD [Table 6.6.1.2.3-1]**.

In addition to the nominal fatigue resistance computations, fatigue requirements for webs must also be checked. These checks are required to control out-of-plane flexing of the web due to flexure or shear under repeated live loading.

## 24.6.14 Design for Flexure – Service Limit State

The girder must be checked for service limit state control of permanent deflection. This check is intended to prevent objectionable permanent deflections due to expected severe traffic loadings that would impair rideability. Service II is used for this check.

In addition to the check for service limit state control of permanent deflection, the girder must also be checked for live load deflection, as described in 24.4.3.

## 24.6.15 Design for Flexure – Constructability Check

The girder must also be checked for flexure during construction. The girder has already been checked in its final condition when it behaves as a composite section. It is the responsibility of the contractor to ensure that allowable stresses aren't exceeded during steel erection. The engineer is to make certain allowable stresses aren't exceeded from the time the steel erection is complete through final service, including during the deck pour. In addition, check the lateral bracing without the deck slab.

Before constructability checks can be performed, the slab pouring sequence must be determined. Refer to Standard for *Slab Pouring Sequence*. Determine the maximum amount of concrete that can be poured in a day. Determine deflections based on the proposed pouring sequence. The effects of the deck pouring sequence will often control the design of the top flange in the positive moment regions of composite girders.

Lateral torsional buckling can occur when the compression flange is not laterally supported. The laterally unsupported compression flange tends to buckle out-of-plane between the points of lateral support. Because the tension flange is kept in line, the girder section twists when it moves laterally. This behavior is commonly referred to as lateral torsional buckling. Lateral torsional buckling is generally most critical for the moments induced during the deck pouring sequence. If lateral torsional buckling occurs, the plastic moment resistance, Mp, cannot be reached.

In addition to checking the nominal flexural resistance during construction, the nominal shear resistance must also be checked.

### 24.6.16 Check Wind Effects on Girder Flanges

The next step is to check wind effects on the girder flanges. Wind effects generally do not control a steel girder design, and they are generally considered for the exterior girders only.

### 24.6.17 Draw Schematic of Final Steel Girder Design

If all of the above specification checks are satisfied, then the trial girder section is acceptable and can be considered the final girder section. It is often useful to draw a schematic summarizing the design of the final girder section.

However, if any of the specification checks are not satisfied or if the design is found to be overly conservative, then the trial girder section must be revised appropriately, and the specification checks must be repeated for the new trial girder section.

### 24.6.18 Design Bolted Field Splices

If bolted field splices are used, they must be designed, as described in 24.8.

### 24.6.19 Design Shear Connectors

For a composite steel girder, the shear connectors must be designed, as described in 24.7.5. The shear connector spacing must be computed based on fatigue and strength limit states.

### 24.6.20 Design Bearing Stiffeners

The next step is to design the bearing stiffeners, as described in 24.9.

### 24.6.21 Design Welded Connections

Welded connections are required at several locations on the steel superstructure, and all welds must be designed. Base metal, weld metal and welding design details must conform to the requirements of the *ANSI/AASHTO/AWS Bridge Welding Code D1.5*.

In most cases, the minimum weld thickness provides a welded connection that satisfies all design requirements. Therefore, the minimum weld thickness is generally a good starting point when designing a fillet weld.

The designer shall investigate all welded connections to a tension flange. Calculate and show the tension zones on top and bottom flanges for all continuous steel girders on the contract plans. The defined tension zone will assist with inspection and prohibit field welding within the tension zone, unless noted otherwise (i.e. shear connectors). Field welding within the tension zone for construction purposes (i.e. deck form attachments) is prohibited. See Chapter 6-Plan Preparation for additional guidance.

## 24.6.22 Design Diaphragms, Cross-Frames and Lateral Bracing

Diaphragms and cross-frames must be designed in accordance with **LRFD [6.7.4]**. Diaphragms and cross-frames may be placed at the following locations along the bridge:

- At the end of the structure
- Across interior supports
- Intermittently along the span

When investigating the need for diaphragms or cross-frames and when designing them, the following must be considered:

- Transfer of lateral wind loads from the bottom of the girder to the deck and from the deck to the bearings
- Stability of the bottom flange for all loads when it is in compression
- Stability of the top flange in compression prior to curing of the deck
- Distribution of vertical dead and live loads applied to the structure

Diaphragms or cross-frames can be specified as either temporary (if they are required only during construction) or permanent (if they are required during construction and in the bridge's final condition).

At a minimum, *AASHTO LRFD* requires that diaphragms and cross-frames be designed for the following transfer of wind loads based on **LRFD [4.6.2.7]** and for applicable slenderness requirements in accordance with **LRFD [6.8.4]** or **LRFD [6.9.3]**. In addition, connection plates must satisfy the requirements of **LRFD [6.6.1.3.1]**.

Refer to Standards 24.03 through 24.06 for information about the design of lateral bracing and end diaphragms. Consideration must be given to connection details susceptible to fatigue crack growth.

## 24.6.23 Determine Deflections, Camber, and Elevations

Determine the dead load deflections, blocking, camber, top of steel elevations and top of slab elevations. Camber and blocking are described in 24.4.8.

## 24.7 Composite Design

### 24.7.1 Composite Action

Composite action is present in steel girder superstructures when the steel beams or girders feature shear connectors which are embedded within the concrete slab. The shear connectors prevent slip and vertical separation between the bottom of the slab and the top of the steel member. Unless temporary shoring is used, the steel members deflect under the dead load of the wet concrete before the shear connectors become effective. However, since temporary shoring is not used in Wisconsin, composite action applies only to live loads and to portions of dead load placed after the concrete deck has hardened.

In the positive moment region, the concrete deck acts in compression and the composite section includes the slab concrete. However, in the negative moment region, the concrete deck acts in tension and the composite section includes the bar steel reinforcement in the slab.

As previously described, for LRFD, Wisconsin places shear connectors in both the positive and negative moment regions of continuous steel girder bridges, and both regions are considered composite in analysis and design computations. Negative flexure concrete deck reinforcement is considered in the section property calculations.

**WisDOT policy item:**

For rehabilitation projects, do not add shear studs in the negative moment region if none exist. Likewise, do not add additional studs in the positive moment region if shear connectors are provided and were designed for shear (not slab anchors on approximately 3'-0" to 4'-0" spacing).

If slab anchors are provided, consider as non-composite and add shear connectors if necessary for rating purposes. If adequate shear connector embedment into the deck is not achieved, additional reinforcement should be provided as per Figure 17.5-1.

### 24.7.2 Values of n for Composite Design

The effective composite concrete slab is converted to an equivalent steel area by dividing by n. For $f'_c$ = 4 ksi, use n = 8.

$f'_c$ = Minimum ultimate compressive strength of the concrete slab at 28 days

n = Ratio of modulus of elasticity of steel to that of concrete

The actual calculation of creep stresses in composite girders is theoretically complex and not necessary for the design of composite girders. Instead, a simple approach has been adopted for design in which a modular ratio appropriate to the duration of the load is used to compute the corresponding elastic section properties. As specified in **LRFD [6.10.1.1.1b]**, for transient loads applied to the composite section, the so-called ″short-term″ modular ratio, n, is used. However, for permanent loads applied to the composite section, the so-called ″long-term″ modular ratio, 3n, is used. The short-term modular ratio is based on the initial tangent modulus, $E_c$, of the concrete, while the long-term modular ratio is based on an effective apparent modulus, $E_c/k$, to account for the effects of creep. In U.S. practice, a value of k equal to 3 has been accepted as a reasonable value.

## 24.7.3 Composite Section Properties

The minimum effective slab thickness is equal to the nominal slab thickness minus 1/2" for wearing surface. The maximum effective slab width is defined in **LRFD [4.6.2.6]**.

## 24.7.4 Computation of Stresses

### 24.7.4.1 Non-composite Stresses

For non-composite sections, flexural stresses are computed using only non-composite (steel-only) section properties, as follows:

$$f_b = \frac{DLM\,(DC1)}{S(\text{steel only})} + \frac{DLM\,(DC2\ \&\ DW)}{S(\text{steel only})} + \frac{LLM\,(\text{Traffic})}{S(\text{steel only})} + \frac{LLM\,(\text{Pedestrian})}{S(\text{steel only})}$$

### 24.7.4.2 Composite Stresses

For composite sections, flexural stresses in the steel girder subjected to positive flexure are computed using appropriate non-composite (steel-only) and composite section properties, as follows:

$$f_b = \frac{DLM\,(DC1)}{S(\text{steel only})} + \frac{DLM\,(DC2\ \&\ DW)}{S(\text{composite},3n)} + \frac{LLM\,(\text{Traffic})}{S(\text{composite},n)} + \frac{LLM\,(\text{Pedestrian})}{S(\text{composite},n)}$$

For composite sections, flexural stresses in the concrete deck subjected to positive flexure are computed as follows:

$$f_b = \frac{DLM(DC2 + DW)}{S(\text{composite},n)} + \frac{LLM(\text{Traffic})}{S(\text{composite},n)} + \frac{LLM(\text{Pedestrian})}{S(\text{Composite},n)}$$

Where:

| | | |
|---|---|---|
| $f_b$ | = | Computed steel flexural stress |
| DLM | = | Dead lead moment |
| LLM | = | Live load moment |
| S | = | Elastic section modulus |
| DC1 | = | DC dead load resisted by the steel section only (for example, steel girder, concrete deck, concrete haunch, cross-frames and stiffeners) |

| | | |
|---|---|---|
| DC2 | = | DC dead load resisted by the composite section (for example, concrete parapets) |
| DW | = | Dead load due to future wearing surface and utilities |

## 24.7.5 Shear Connectors

Refer to Standard for *Plate Girder Details* for shear connector details. Three shop or field welded 7/8" diameter studs at a length of 5" are placed on the top flange. The studs are equally spaced with a minimum clearance of 1 1/2" from the edge of the flange. On girders having thicker haunches where stud embedment is less than 2" into the slab, longer studs should be used to obtain the minimum embedment of 2".

Connectors which fall on the flange field splice plates should be repositioned near the ends of the splice plate. The maximum spacing of shear connectors is 2'. Connector spacings should begin a minimum of 9" from the centerline of abutments.

To determine the locations of shear connectors along the length of the girder, two general requirements must be satisfied:

- Spacing (or pitch) requirements governed by fatigue, as presented in **LRFD [6.10.10.1]**
- Number of connector requirements governed by strength, as presented in **LRFD [6.10.10.4]**

For the fatigue limit state, the pitch, p, of the shear connectors must satisfy the following equation:

$$p \le \frac{nZ_r}{V_{sr}}$$

Where:

| | | |
|---|---|---|
| N | = | Number of shear connectors in a cross section |
| $V_{sr}$ | = | Horizontal fatigue shear range per unit length (kips/in.) |
| $Z_r$ | = | Shear fatigue resistance of an individual shear connector determined as specified in **LRFD [6.10.10.2]** (kips) |

When computing the value for $V_{sr}$, the maximum value of composite moment of inertia in the span can be used.

For the strength limit state, the minimum number of required shear connectors, n, is computed for a given region according to the following equation:

$$n = \frac{P}{Q_r}$$

Where:

| | | |
|---|---|---|
| P | = | Total nominal shear force determined as specified in **LRFD [6.10.10.4.2]** (kips) |
| $Q_r$ | = | Factored shear resistance of one shear connector (kips) |

The given regions over which the required number of shear connectors is distributed are defined based on the point of maximum moment due to live load plus dynamic load allowance. This value is used because it applies to the composite section and is easier to locate than a maximum of the sum of all the moments acting on the composite section.

In most cases, the connector spacing (using three connectors per row) based on fatigue requirements is more than adequate for the strength design requirements. However for relatively long spans, additional shear connectors may be required to satisfy the strength design requirements.

In addition to the above general requirements, special shear connector requirements at points of permanent load contraflexure are presented in **LRFD [6.10.10.3]**.

Additional information and equations used for LRFD design of shear connectors are presented in **LRFD [6.10.10]**. In addition, a design example for shear connectors is also provided in this *Bridge Manual*.

## 24.7.6 Continuity Reinforcement

For continuous steel girder bridges, continuity reinforcement in the concrete deck must be considered in regions of negative flexure, as specified in **LRFD [6.10.1.7]**. Continuity reinforcement consisting of small bars with close spacing is intended to control concrete deck cracking.

If the longitudinal tensile stress in the concrete deck due to either the factored construction loads or the Service II load combination exceeds $\phi f_r$, then the following continuity reinforcement requirements must be satisfied:

- The total cross-sectional area of the longitudinal reinforcement in the deck shall be greater than or equal to one percent of the total cross-sectional area of the concrete deck.

- The required reinforcement shall be placed in two layers uniformly distributed across the deck width, with two-thirds being in the top layer and one-third in the bottom layer.

- The specified minimum yield strength, fy, of the reinforcing steel shall not be less than 60 ksi.

- The size of the reinforcement bars shall not exceed No. 6 bars.

- The spacing of the reinforcement bars shall not exceed 12 inches.

Tables 17.5-3 and 17.5-4 meet the criteria specified above.

In computing $\phi f_r$, $f_r$ shall be taken as the modulus of rupture of the concrete (see **LRFD [5.4.2.6]**) and $\phi$ shall be taken as 0.90, which is the appropriate resistance factor for concrete in tension (see **LRFD [5.5.4.2]**). The longitudinal stresses in the concrete deck are computed as specified in **LRFD [6.10.1.1.1d]**. Superimposed dead loads and live loads are considered to be resisted by the composite section using the short-term modular ratio, n. Non-composite dead loads are supported by the girders alone.

Terminate the continuity reinforcement at the point of non-composite dead load contraflexure plus a development length. The bars are lapped to No. 4 bars.

For non-composite slabs in the negative moment region (on rehabilitation projects), extend the longitudinal reinforcement in Tables 17.5-3 and 17.5-4 a development length beyond the first shear connectors.

## 24.8 Field Splices

### 24.8.1 Location of Field Splices

Field splices shall be placed at the following locations whenever it is practical:

- At or near a point of dead load contraflexure for continuous spans
- Such that the maximum rolling length of the flange plates is not exceeded, thus eliminating one or more butt splices
- At a point where the fatigue in the net section of the base metal is minimized
- Such that section lengths between splices are limited to 120', unless special conditions govern

### 24.8.2 Splice Material

For homogeneous girders, the splice material is the same as the members being spliced. Generally, 3/4" diameter high-strength A325 bolted friction-type connectors, conforming to ASTM F3125, are used unless the proportions of the structure warrant larger diameter bolts.

### 24.8.3 Design

The following is a general description of the basic steps required for field splice design. These procedures and the accompanying equations are described in greater detail in **LRFD [6.13.6]**.

#### 24.8.3.1 Obtain Design Criteria

The first design step is to identify the appropriate design criteria. This includes defining material properties, identifying relevant superstructure information and determining the splice location based on the criteria presented in 24.8.1.

Resistance factors used for field splice design are as presented in 17.2.6.

When calculating the nominal slip resistance of a bolt in a slip-critical connection, the value of the surface condition factor, $K_s$, shall be taken as follows for the surfaces in contact (faying):

- For steel with fully painted surfaces, use Ks = 0.30.
- For unpainted, blast-cleaned steel or steel with organic zinc paint, use Ks = 0.50.

Where a section changes at a splice, the smaller of the two connected sections should be used in the design, as specified in **LRFD [6.13.6.1.1]**.

##### 24.8.3.1.1 Section Properties Used to Compute Stresses

The section properties used to compute stresses are described in **LRFD [6.10.1.1.1]**.

For calculating flexural stresses in sections subjected to positive flexure, the composite sections for short-term (transient) and long-term (permanent) moments shall be based on n and 3n, respectively.

For calculating flexural stresses in sections subjected to negative flexure, the composite section for both short-term and long-term moments shall consist of the steel section and the longitudinal reinforcement within the effective width of the concrete deck, except as specified otherwise in **LRFD [6.6.1.2.1]**, **LRFD [6.10.1.1.1d]** or **LRFD [6.10.4.2.1]**.

**WisDOT policy item:**

When computing composite section properties based on the steel section and the longitudinal reinforcement within the effective width of the concrete deck, only the top layer of reinforcement shall be considered.

Where moments due to short-term and long-term loads are of opposite sign at the strength limit state, the associated composite section may be used with each of these moments if the resulting net stress in the concrete deck due to the sum of the factored moments is compressive. Otherwise, the provisions of **LRFD [6.10.1.1.1c]** shall be used to determine the stresses in the steel section. Stresses in the concrete deck shall be determined as specified in **LRFD [6.10.1.1.1d]**.

However, for members with shear connectors provided throughout their entire length that also satisfy the provisions of **LRFD [6.10.1.7]**:

- Flexural stresses caused by Service II loads applied to the composite section may be computed using the short-term or long-term composite section, as appropriate, assuming the concrete deck is effective for both positive and negative flexure, as described in **LRFD [6.10.4.2.1]**.

- Live load stresses and stress ranges for fatigue design may be computed using the short-term composite section assuming the concrete deck to be effective for both positive and negative flexure, as described in **LRFD [6.6.1.2.1]**.

**WisDOT policy item:**

When stresses at the top and bottom of the web are required for web splice design, the flange stresses at the mid-thickness of the flanges can be conservatively used. This allows use of the same stress values for both the flange and web splices, which simplifies the calculations.

### 24.8.3.1.2 Constructability

As described in **LRFD [6.13.6.1.3a]**, splice connections shall be proportioned to prevent slip during the erection of the steel and during the casting of the concrete deck.

## 24.8.3.2 Compute Flange Splice Design Loads

Commercially available software programs can be used to obtain the design dead loads and live loads at the splice. The live loads should include dynamic load allowance and distribution factors.

Splices are typically designed for the Strength I, Service II and Fatigue I load combinations. The load factors for these load combinations are presented in 17.2.5. The stresses corresponding to these load combinations should be computed at the mid-thickness of the top and bottom flanges.

### 24.8.3.2.1 Factored Loads

For the Strength I and Service II load combinations, factored loads must be computed for the following two cases:

- Case 1: Dead load + Positive live load
- Case 2: Dead load + Negative live load

For the Fatigue I load combination, the following two load cases are used to compute the factored loads:

- Case 1: Positive live load
- Case 2: Negative live load

Minimum and maximum load factors are applied as appropriate to compute the controlling loading.

### 24.8.3.2.2 Section Properties

Section properties based on the gross area of the steel girder are used for computation of the maximum flexural stresses due to the factored loads for the Strength I, Service II and Fatigue I load combinations, as described in **LRFD [6.13.6.1.3a,b]** and **LRFD [C6.13.6.1.3a,b]**.

### 24.8.3.2.3 Factored Stresses

After the factored loads and section properties have been computed, factored stresses must be computed for each of the following cases:

- Strength I load combination – Dead load + Positive live load
- Strength I load combination – Dead load + Negative live load
- Service II load combination – Dead load + Positive live load
- Service II load combination – Dead load + Negative live load

- Fatigue I load combination – Positive live load
- Fatigue I load combination – Negative live load

Factored stresses are computed by dividing the factored moments by the appropriate section moduli.

### 24.8.3.2.4 Controlling Flange

As described in **LRFD [C6.13.6.1.3a,b]**, the controlling flange is defined as either the top or bottom flange for the smaller section at the point of splice, whichever flange has the maximum ratio of the elastic flexural stress at its mid-thickness due to the factored loads for the loading condition under investigation to its factored flexural resistance. The other flange is termed the non-controlling flange. In areas of stress reversal, the splice must be checked independently for both positive and negative flexure. For composite sections in positive flexure, the controlling flange is typically the bottom flange. For sections in negative flexure, either flange may qualify as the controlling flange.

### 24.8.3.2.5 Flange Splice Design Forces

After the factored stresses have been computed, the flange splice design forces can be computed as specified in **LRFD [6.13.6.1.3a,b]**. The design forces are computed for both the top and bottom flange for each load case (positive and negative live load). For the Strength I load combination, the design force is computed as the design stress times the smaller effective flange area on either side of the splice. When a flange is in compression, the gross flange area is used.

Service II load combination design forces must also be computed. As specified in **LRFD [6.13.6.1.3a,b]**, bolted connections for flange splices should be designed as slip-critical connections for the service level flange design force. This design force is computed as the Service II design stress multiplied by the smaller gross flange area on either side of the splice.

The flange slip resistance must exceed the larger of the following:

- Service II flange forces
- Factored flange forces from the moments at the splice due to constructability (erection and/or deck pouring sequence), as described in **LRFD [6.13.6.1.3a,b]**

For the Fatigue I load combination, the stress range at the mid-thickness of both flanges must be computed.

### 24.8.3.3 Design Flange Splice Plates

The next step is to design the flange splice plates. The width of the outside plate should be at least as wide as the width of the narrowest flange at the splice. The width of the inside plate must allow sufficient clearance for the web and for inserting and tightening the web and flange splice bolts. Fill plates are used when the flange plate thickness changes at the splice location. A typical flange splice configuration is presented in Figure 24.8-1.

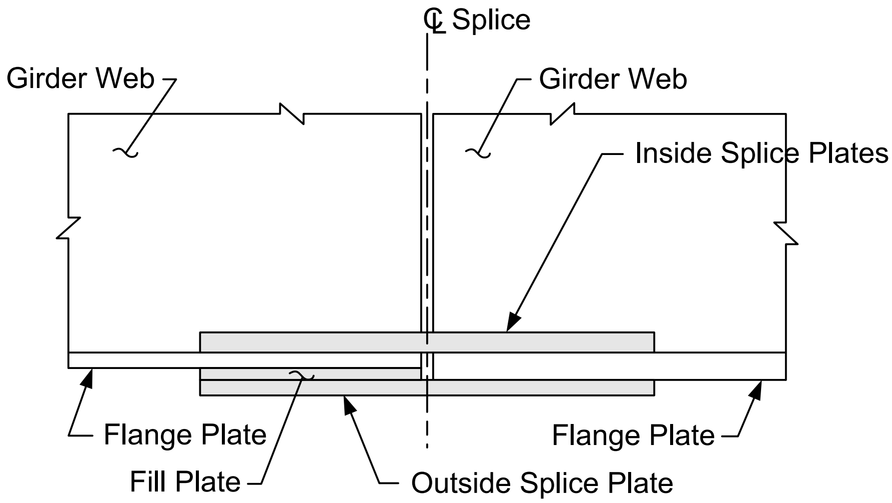


**Figure 24.8-1**
Bottom Flange Splice Configuration

If the combined area of the inside splice plates is within ten percent of the area of the outside splice plate, then both the inside and outside splice plates may be designed for one-half the flange design force, as described in **LRFD [C6.13.6.1.3a,b]**. However, if the areas of the inside and outside splice plates differ by more than ten percent, then the flange design force should be proportioned to the inside and outside splice plates. This is calculated by multiplying the flange design force by the ratio of the area of the splice plate under consideration to the total area of the inner and outer splice plates.

### 24.8.3.3.1 Yielding and Fracture of Splice Plates

The design force in the splice plates at the Strength I load combination shall not exceed the factored resistances for yielding and fracture, as described in **LRFD [6.13.5.2]** and **LRFD [6.8.2]**.

For a tension member, the net width shall be determined for each chain of holes extending across the member along any transverse, diagonal or zigzag line. This is determined by subtracting from the width of the element the sum of the width of all holes in the chain and adding the quantity $s^2/4g$ for each space between consecutive holes in the chain. For non-staggered holes, the minimum net width is the width of the element minus the width of bolt holes in a line straight across the width.

For a compression member, the gross area is used for these design checks.

24.8.3.3.2 Block Shear

All tension connections, including connection plates, splice plates and gusset plates, shall be investigated to ensure that adequate connection material is provided to develop the factored resistance of the connection. Block shear rupture resistance is described in **LRFD [6.13.4]**. A bolt pattern must be assumed prior to checking an assumed block shear failure mode.

Block shear rupture will usually not govern the design of splice plates of typical proportion.

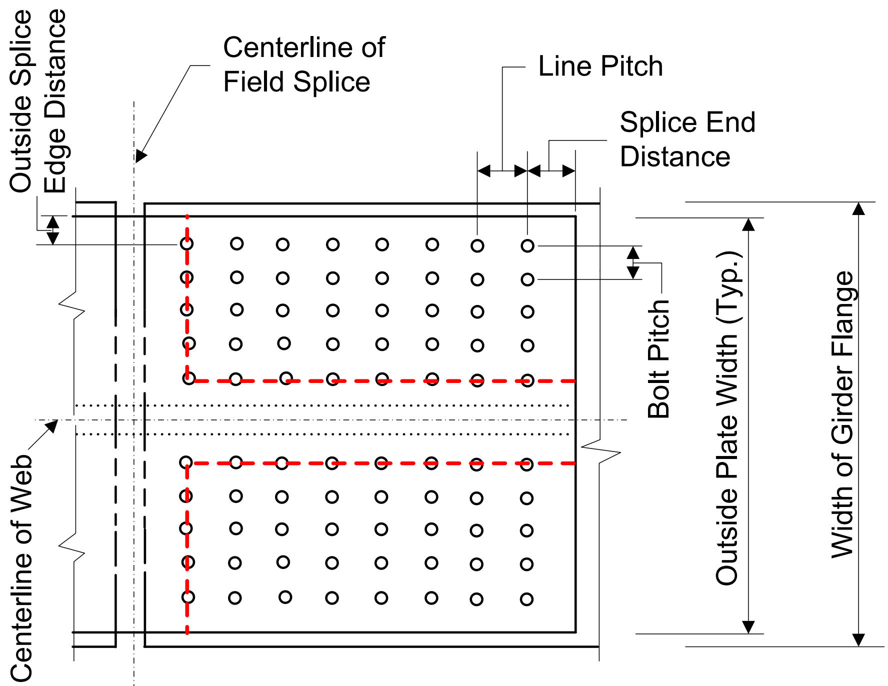


**Figure 24.8-2**
Double – L  Block Shear Path, Flange and Splice Plates

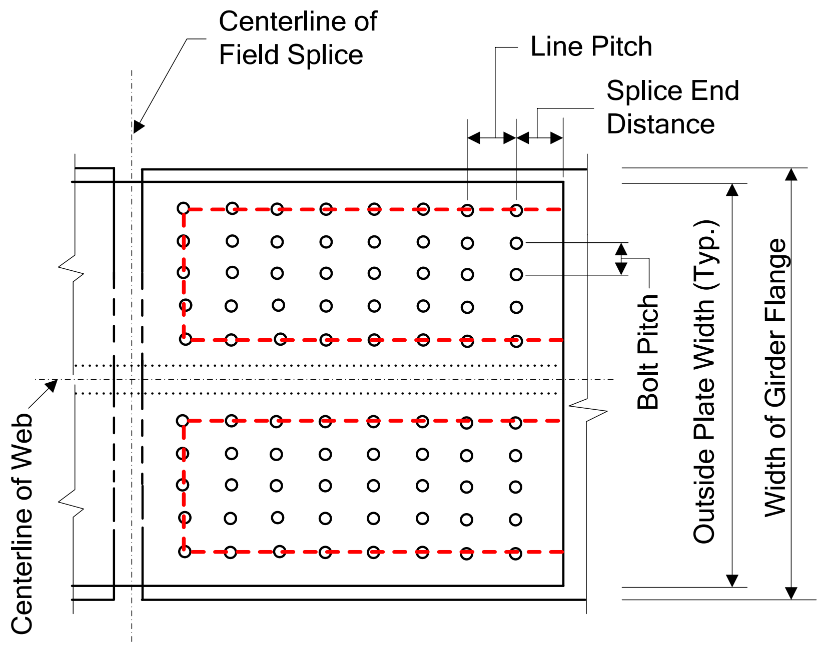


**Figure 24.8-3**
Double – U Block Shear Path, Flange and Splice Plates

## 24.8.3.3.3 Net Section Fracture

When checking flexural members at the Strength I load combination or for constructability, all cross sections containing holes in the tension flange must satisfy the fracture requirements of **LRFD [6.10.1.8]**.

## 24.8.3.3.4 Fatigue of Splice Plates

Check the fatigue stresses in the base metal of the flange splice plates adjacent to the slip-critical connections. Fatigue normally does not govern the design of the splice plates. However, a fatigue check of the splice plates is recommended whenever the area of the flange splice plates is less than the area of the smaller flange to which they are attached.

The fatigue detail category under the condition of Mechanically Fastened Connections for checking the base metal at the gross section of high-strength bolted slip-resistant connections is Category B.

## 24.8.3.3.5 Control of Permanent Deformation

A check of the flexural stresses in the splice plates at the Service II load combination is not explicitly specified in *AASHTO LRFD*. However, whenever the combined area of the inside and outside flange splice plates is less than the area of the smaller flange at the splice, such a check is recommended.

## 24.8.3.4 Design Flange Splice Bolts

After the flange splice plates have been designed, the flange splice bolts must be designed for shear, slip resistance, spacing, edge distance and bearing requirements.

### 24.8.3.4.1 Shear Resistance

Shear resistance computations for bolted connections are described in **LRFD [6.13.2.7]**. The first step is to determine the number of bolts for the flange splice plates that are required to develop the Strength I design force in the flange in shear, assuming the bolts in the connection have slipped and gone into bearing. A minimum of two rows of bolts should be provided to ensure proper alignment and stability of the girder during construction.

The factored resistance of the bolts in shear must be determined, assuming the threads are excluded from the shear planes. For the flange splice bolts, the number of bolts required to provide adequate shear strength is determined by assuming the design force acts on two shear planes, known as double shear.

Requirements for filler plates are presented in **LRFD [6.13.6.1.4]**. When bolts carrying loads pass through fillers 0.25 inches or more in thickness in axially loaded connections, including girder flange splices, either of the following is required:

- The fillers shall be extended beyond the gusset or splice material and shall be secured by enough additional bolts to distribute the total stress in the member uniformly over the combined section of the member and the filler.
- The fillers need not be extended and developed provided that the factored resistance of the bolts in shear at the Strength I load combination is reduced by the factor presented in **LRFD [6.13.6.1.4]**.

### 24.8.3.4.2 Slip Resistance

As specified in **LRFD [6.13.6.1.3a,b]**, bolted connections for flange splices shall be designed as slip-critical connections for the Service II flange design force or the flange design force from constructability, whichever governs. Slip resistance computations for bolted connections are described in **LRFD [6.13.2.8]**.

When checking for slip of the bolted connection for a flange splice with inner and outer splice plates, the slip resistance should always be determined by dividing the flange design force equally to the two slip planes, regardless of the ratio of the splice plate areas. Slip of the connection cannot occur unless slip occurs on both planes.

### 24.8.3.4.3 Bolt Spacing

The minimum spacing between centers of bolts in standard holes shall be no less than three times the diameter of the bolt.

The maximum spacing for sealing must be checked to prevent penetration of moisture in the joints, in accordance with **LRFD [6.13.2.6.2]**. Sealing must be checked for a single line adjacent to a free edge of an outside plate or shape (for example, when the bolts along the edges of the plate are parallel to the direction of the applied force) and along the free edge at the end of the splice plate.

### 24.8.3.4.4 Bolt Edge Distance

Edge distance requirements must be checked as specified in **LRFD [6.13.2.6.6]**. The minimum required edge distance is measured as the distance from the center of any bolt in a standard hole to an edge of the plate.

The maximum edge distance shall not be more than eight times the thickness of the thinnest outside plate or 5.0 inches.

### 24.8.3.4.5 Bearing at Bolt Holes

Finally, bearing at the bolt holes must be checked, as specified in **LRFD [6.13.2.9]**. The flange splice bolts are checked for bearing of the bolts on the connected material under the maximum Strength I design force. The design bearing strength of the connected material is calculated as the sum of the smaller of the nominal shear resistance of the individual bolts and the nominal bearing resistance of the individual bolt holes parallel to the line of the applied force. Nominal shear resistance of the bolt is found in **LRFD [6.13.2.7]**.

If the bearing resistance controls and is not adequate, it is recommended that the edge distance be increased slightly, in lieu of increasing the number of bolts or thickening the flange splice plates.

### 24.8.3.5 Compute Web Splice Design Loads

The next step is to compute the web splice design loads for each of the following cases:

- Strength I load combination – Dead load + Positive live load
- Strength I load combination – Dead load + Negative live load
- Service II load combination – Dead load + Positive live load
- Service II load combination – Dead load + Negative live load
- Fatigue I load combination – Positive live load
- Fatigue I load combination – Negative live load

As specified in **LRFD [6.13.6.1.3a,c]**, web splice plates and their connections shall be designed for the following loads:

- Girder shear forces at the splice location

- Moment due to the eccentricity of the shear at the point of splice
- The portion of the flexural moment assumed to be resisted by the web at the point of the splice

#### 24.8.3.5.1 Girder Shear Forces at the Splice Location

As previously described, any number of commercially available software programs can be used to obtain the design dead loads and live loads at the splice. The live loads must include dynamic load allowance and distribution factors.

#### 24.8.3.5.2 Web Moments and Horizontal Force Resultant

Because the portion of the flexural moment assumed to be resisted by the web is to be applied at the mid-depth of the web, a horizontal design force resultant must also be applied at the mid-depth of the web to maintain equilibrium. The web moment and horizontal force resultant are applied together to yield a combined stress distribution equivalent to the unsymmetrical stress distribution in the web. For sections with equal compressive and tensile stresses at the top and bottom of the web (that is, with the neutral axis located at the mid-depth of the web), the horizontal design force resultant will equal zero.

In the computation of the portion of the flexural moment assumed to be resisted by the web and the horizontal design force resultant in the web, the flange stresses at the midthickness of the flanges can be conservatively used, as described in **LRFD [C6.13.6.1.3c]**. This allows use of the same stress values for both the flange and web splices, which simplifies the calculations.

The moment due to the eccentricity of the design shear is resisted solely by the web and always acts about the mid-depth of the web (that is, the horizontal force resultant is zero). This moment is computed as the design shear times the distance from the centerline of the splice to the centroid of the connection on the side of the joint under consideration.

The total web moment for each load case is computed as the sum of these two moments.

In general, the web splice is designed under the conservative assumption that the maximum moment and shear at the splice will occur under the same loading condition.

### 24.8.3.6 Design Web Splice Plates

After the web splice design forces are computed, the web splice must be designed. First, a preliminary web splice bolt pattern is determined. The outermost rows of bolts in the web splice plate must provide sufficient clearance from the flanges to provide clearance for assembly (see the *AISC Manual of Steel Construction* for required bolt assembly clearances). The web is spliced symmetrically by plates on each side with a thickness not less than one-half the thickness of the web. A typical web splice configuration is presented in Figure 24.8-4.

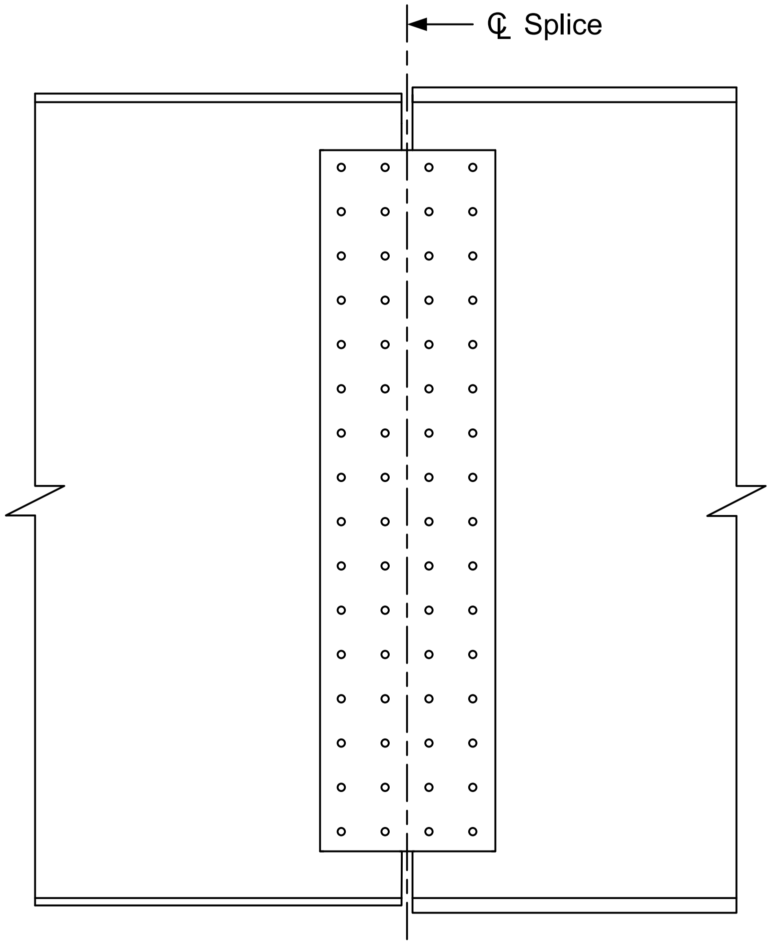


**Figure 24.8-4**
Web Splice Configuration

The web splice plates should be extended as near as practical the full depth of the web between flanges without impinging on bolt assembly clearances. Also, at least two vertical rows of bolts in the web on each side of the splice should be used. This may result in an over-designed web splice, but it is considered good engineering practice.

### 24.8.3.6.1 Shear Yielding of Splice Plates

Shear yielding on the gross section of the web splice plates must be checked under the Strength I design shear force, as specified in **LRFD [6.13.6.1.3a,c]**.

### 24.8.3.6.2 Fracture and Block Shear Rupture of the Web Splice Plates

Fracture must be investigated on the net section extending across the full plate width, in accordance with **LRFD [6.13.6.1.3a,c]**. In addition, block shear rupture resistance must be checked in accordance with **LRFD [6.13.4]**. Connection plates, splice plates and gusset plates shall be investigated to ensure that adequate connection material is provided to develop the factored resistance of the connection. Strength I load combination checks for fracture on the net section of web splice plates and block shear rupture normally do not govern for plates of typical proportion.

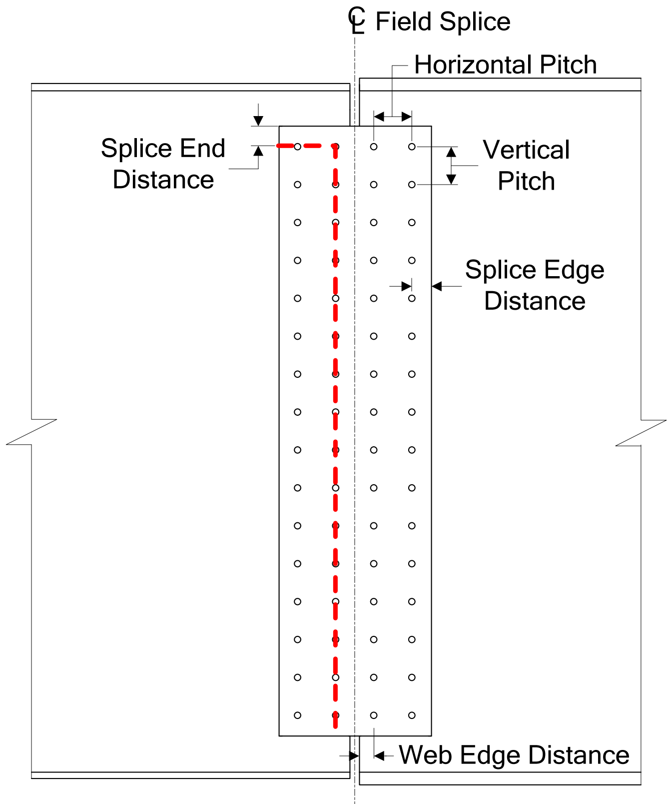


**Figure 24.8-5**
Block Shear Path, Web Splice

### 24.8.3.6.3 Flexural Yielding of Splice Plates

Flexural yielding on the gross section of the web splice plates must be checked for the Strength I load combination due to the total web moment and the horizontal force resultant. Flexural yielding must be checked for dead load and positive live load, as well as dead load and negative live load. Flexural yielding of splice plates is checked in accordance with **LRFD [6.13.6.1.3a,c]**.

### 24.8.3.6.4 Fatigue of Splice Plates

In addition, fatigue of the splice plates must be checked. Fatigue is checked at the edge of the splice plates which is subject to a net tensile stress. The normal stresses at the edge of the splice plates due to the total positive and negative fatigue load web moments and the corresponding horizontal force resultants are computed.

Check the fatigue stresses in the base metal of the web splice plates adjacent to the slip-critical connections. Fatigue normally does not govern the design of the splice plates. However, a fatigue check of the splice plates is recommended whenever the area of the web splice plates is less than the area of the web at the splice.

The fatigue detail category under the condition of Mechanically Fastened Connections for checking the base metal at the gross section of high-strength bolted slip-resistant connections is Category B.

| **WisDOT policy item:** |
| --- |
| For the Fatigue I load combination, the stress range at the mid-thickness of both flanges may be used when checking fatigue in the web. |

### 24.8.3.7 Design Web Splice Bolts

Similar to the flange splice bolts, the web splice bolts must be designed for shear, slip resistance, spacing, edge distance and bearing requirements. These bolt requirements are described in 24.8.3.4.

#### 24.8.3.7.1 Shear in Web Splice Bolts

Shear in the web splice bolts is checked in accordance with **LRFD [6.13.6.1.3a,c]**. The polar moment of inertia, $I_p$, of the bolt group on each side of the web centerline with respect to the centroid of the connection is computed as follows:

$$I_p = \frac{n \cdot m}{12} \cdot \left[s^2 \cdot \left(n^2 - 1\right) + g^2 \cdot \left(m^2 - 1\right)\right]$$

Where:

| | | |
| --- | --- | --- |
| n | = | Number of bolts in each vertical row |
| m | = | Number of vertical rows of bolts |
| s | = | Vertical pitch of bolts (inches) |
| g | = | Horizontal pitch of bolts (inches) |

The polar moment of inertia is required to determine the shear force in a given bolt due to the applied web moments. Shear in the web splice bolts is checked for each of the following cases:

- Strength I load combination – Dead load + Positive live load
- Strength I load combination – Dead load + Negative live load
- Service II load combination – Dead load + Positive live load

- Service II load combination – Dead load + Negative live load

Under the most critical combination of the design shear, moment and horizontal force, it is assumed that the bolts in the web splice have slipped and gone into bearing. The shear strength of the bolts are computed assuming double shear and assuming the threads are excluded from the shear planes.

Since the bolt shear strength for both the flange and web splices is based on the assumption that the threads are excluded from the shear planes, an appropriate note should be placed on the drawings to ensure that the splice is detailed to exclude the bolt threads from the shear planes.

### 24.8.3.7.2 Bearing Resistance at Bolt Holes

Bearing of the web splice bolts on the connected material must be checked for the Strength I load combination, assuming the bolts have slipped and gone into bearing, as specified in **LRFD [6.13.2.9]**. The design bearing strength of the girder web at the location of the extreme bolt in the splice is computed as the minimum resistance along the two orthogonal shear failure planes shown in Figure 24.8-6. The maximum force (vector resultant) acting on the extreme bolt is compared to this calculated strength, which is conservative since the components of this force parallel to the failure surfaces are smaller than the maximum force.

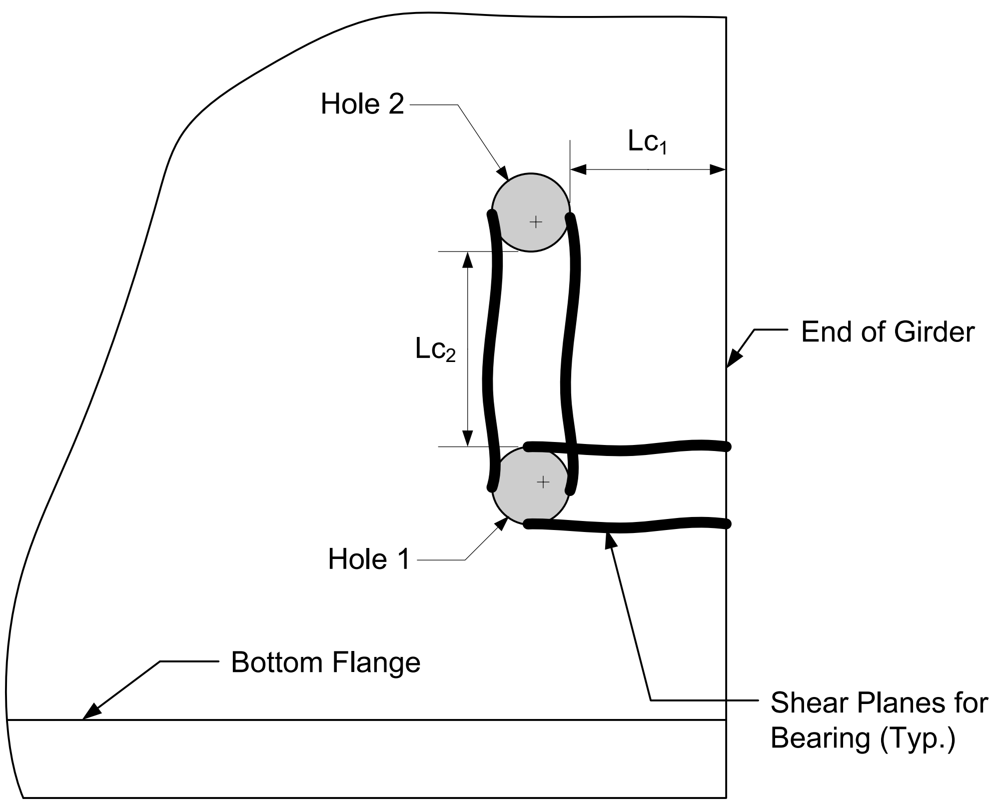


**Figure 24.8-6**
Bearing Resistance at Girder Web Bolt Holes

To determine the applicable equation for the calculation of the nominal bearing resistance, the clear distance between holes and the clear end distance must be calculated and compared to the value of two times the nominal diameter of the bolt. Calculate the bearing resistance at bolt holes using the appropriate equations in **LRFD [6.13.2.9]**. The design bearing strength of the connected material is calculated as the sum of the smaller of the nominal shear resistance of the individual bolts and the nominal bearing resistance of the individual bolt holes. Nominal shear resistance of the bolt is found in **LRFD [6.13.2.7]**. If the bearing resistance controls and is not adequate, it is recommended that the edge distance be increased slightly, in lieu of increasing the number of bolts or thickening the web splice plates.

### 24.8.3.8 Schematic of Final Splice Configuration

After the flange splice plates, flange splice bolts, web splice plates and web splice bolts have been designed and detailed, a schematic of the final splice configuration can be developed. A sample schematic of a final splice configuration is presented in Figure 24.8-7.

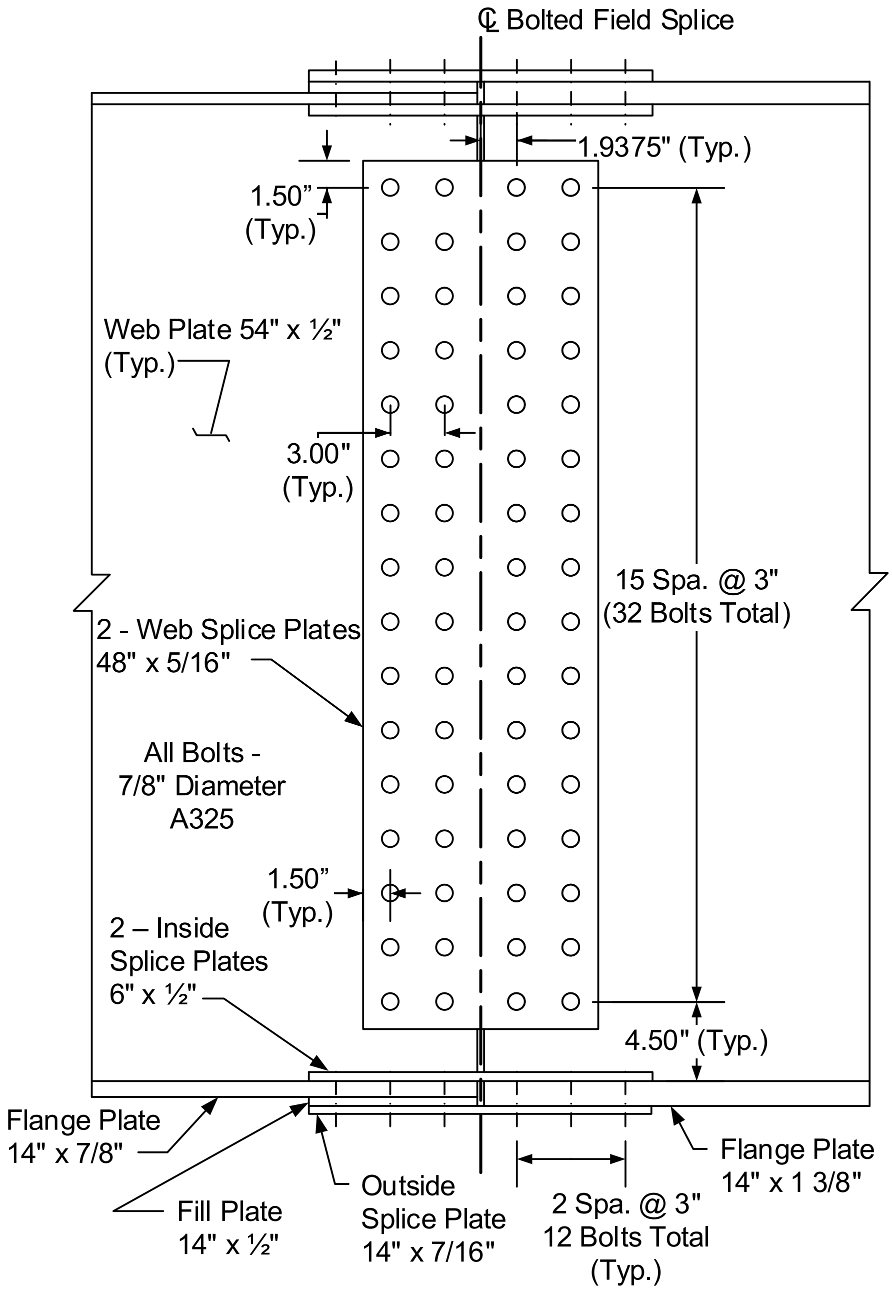


**Figure 24.8-7**
Sample Schematic of Final Splice Configuration

The schematic includes all plates, dimensions, bolt spacings, edge distances and bolt material and diameter.

A design example for field splices is provided in this *Bridge Manual*.

## 24.9 Bearing Stiffeners

For skew angles greater than 15°, bearing stiffeners are placed normal to the web of the girder. However, for skew angles of 15° or less, they may be placed parallel to the skew at the abutments and piers to support the end diaphragms or cross framing.

For structures on grades of 3 percent or greater, the end of the girder section at joints is to be cut vertical. This eliminates the large extension and clearance problems at the abutments.

### 24.9.1 Plate Girders

As specified in **LRFD [6.10.11.2.1]**, bearing stiffeners must be placed on the webs of built-up sections at all bearing locations. Bearing stiffeners are placed over the end bearings of welded plate girders and over the intermediate bearings of continuous welded plate girders. The bearing stiffeners extend as near as practical to the outer edges of the flange plate. They consist of two or more plates placed on both sides of the web. They are ground to a tight fit and fillet welded at the top flange, welded to the web on both sides with the required fillet weld and attached to the bottom flange with full penetration groove welds.

### 24.9.2 Rolled Beams

At bearing locations on rolled shapes and at other locations on built-up sections or rolled shapes subjected to concentrated loads, where the loads are not transmitted through a deck or deck system, either bearing stiffeners must be provided or else the web must satisfy the provisions of **LRFD [D6.5]** (Appendix D to Section 6). According to the provisions of **LRFD [D6.5]**, webs without bearing stiffeners at the indicated locations are to be investigated for the limit states of web local yielding and web crippling. The section must either be modified to comply with these requirements or else bearing stiffeners must be placed on the web at the locations under consideration.

### 24.9.3 Design

The design of bearing stiffeners is covered in **LRFD [6.10.11.2]**. Bearing stiffeners, which are aligned vertically on the web, are designed as columns to resist the reactions at bearing locations and at other locations subjected to concentrated loads where the loads are not transmitted through a deck or deck system.

#### 24.9.3.1 Projecting Width

As specified in **LRFD [6.10.11.2.2]**, the projecting width, $b_t$, of each bearing stiffener element must satisfy the following requirement in order to prevent local buckling of the bearing stiffener plates:

$$b_t \leq 0.48 t_p \sqrt{\frac{E}{F_{ys}}}$$

Where:

$t_p$ = Thickness of the projecting stiffener element (in.)

E = Modulus of elasticity of stiffener (ksi)

$F_{ys}$ = Specified minimum yield strength of the stiffener (ksi)

The projecting width and thickness of the projecting stiffener element are illustrated in Figure 24.9-1.

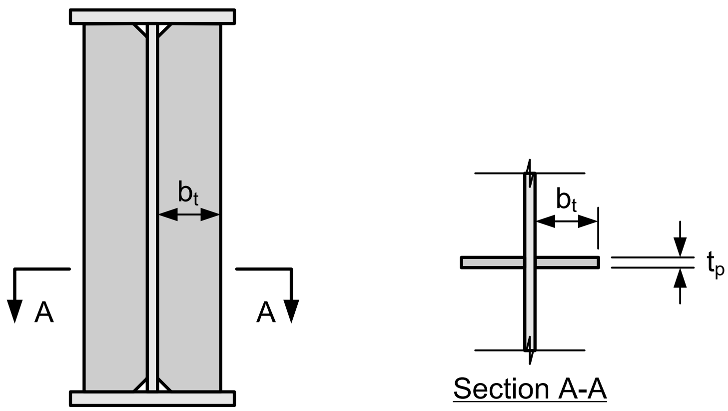


**Figure 24.9-1**
Projecting Width of a Bearing Stiffener

### 24.9.3.2 Bearing Resistance

Bearing stiffeners must be clipped to clear the web-to-flange fillet welds and to bring the stiffener plates tight against the flange through which they receive their load. As a result, the area of the plates in direct bearing on the flange is less than the gross area of the plates. As specified in **LRFD [6.10.11.2.3]**, the factored bearing resistance, $(R_{sb})_r$, of the fitted ends of bearing stiffeners is to be taken as:

$$(R_{sb})_r = \phi_b (R_{sb})_n$$

Where:

$\phi_b$ = Resistance factor for bearing on milled surfaces specified in **LRFD [6.5.4.2]** (= 1.0)

$(R_{sb})_n$ = Nominal bearing resistance for the fitted ends of bearing stiffeners (kips) = 1.4 $A_{pn}F_{ys}$ (**LRFD [Eq'n 6.10.11.2.3-2]**)

$A_{pn}$ = Area of the projecting elements of the stiffener outside of the web-to-flange fillet welds but not beyond the edge of the flange ($in^2$)

$F_{ys}$ = Specified minimum yield strength of the stiffener (ksi)

### 24.9.3.3 Axial Resistance

As previously mentioned, bearing stiffeners are designed as columns. As specified in **LRFD [6.10.11.2.4a]**, the factored axial resistance of the stiffeners, $P_r$, is to be determined as specified in **LRFD [6.9.2.1]** using the specified minimum yield strength of the stiffener plates, $F_{ys}$, in order to account for the effect of any early yielding of lower strength stiffener plates. The factored resistance of components in axial compression is given in **LRFD [6.9.2.1]** as:

$$P_r = \phi_c P_n$$

Where:

| | | |
|---|---|---|
| $\phi_c$ | = | Resistance factor for axial compression specified in **LRFD [6.5.4.2]** (= 0.95) - (axial compression - steel only) |
| $P_n$ | = | Nominal compressive resistance specified in **LRFD [6.9.4.1]** (kips) |

For bearing stiffeners, the nominal compressive resistance, $P_n$, is computed as follows, based on **LRFD [6.9.4.1]**:

If $\lambda \leq 2.25$, then: $P_n = 0.658^{\lambda} \cdot F_{ys} \cdot A_s$

If $\lambda > 2.25$, then: $P_n = (0.877 \cdot F_{ys} \cdot A_s) / \lambda$

Where:

$$\lambda = P_o / P_e = (K\ell / r_s \cdot \pi)^2 \cdot F_{ys} / E \;;\quad P_e = \pi^2 \cdot E \cdot A_s / (K\ell / r_s)^2 \;;\quad P_o = F_{ys} \cdot A_s$$

| | | |
|---|---|---|
| $E$ | = | Modulus of elasticity of steel (ksi) |
| $P_o$ | = | nominal yield resistance (kip) |
| $P_e$ | = | elastic critical buckling resistance (kip) **LRFD [6.9.4.1.2]** |
| $F_{ys}$ | = | Specified minimum yield strength of the stiffener (ksi) |
| $A_s$ | = | Area of effective column section of the bearing stiffeners (see below) (in.$^2$) |
| $K\ell$ | = | Effective length of the effective column taken as 0.75D, where D is the web depth (refer to **LRFD [6.10.11.2.4a]**) (in.) |
| $r_s$ | = | Radius of gyration of the effective column about the plane of buckling computed about the mid-thickness of the web (refer to **LRFD [6.10.11.2.4a]**) (in.) |

### 24.9.3.4 Effective Column Section

The effective column section of the bearing stiffeners is defined in **LRFD [6.10.11.2.4b]**. For stiffeners bolted to the web, the effective column section is to consist of only the stiffener elements. For stiffeners consisting of two plates welded to the web, the effective column section is to consist of the two stiffener plates, plus a centrally located strip of web extending not more than $9t_w$ on each side of the stiffeners, as illustrated in Figure 24.9-2.

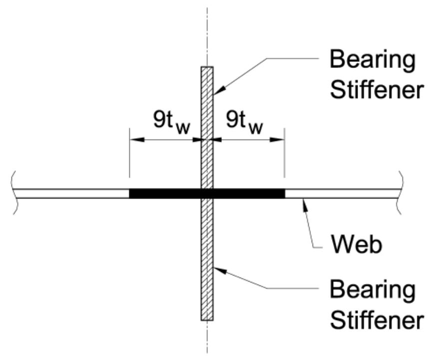


**Figure 24.9-2**
Effective Column Section for Welded Bearing Stiffener Design (One Pair of Stiffeners)

If more than one pair of stiffeners is used, the effective column section is to consist of all the stiffener plates, plus a centrally located strip of web extending not more than $9t_w$ on each side of the outer projecting elements of the group.

Additional information and equations used for LRFD design of bearing stiffeners are presented in **LRFD [6.10.11.2]**. In addition, a design example for bearing stiffeners is also provided in this *Bridge Manual*.

## 24.10 Transverse Intermediate Stiffeners

The design of transverse web stiffeners is specified in **LRFD [6.10.11.1]**. Transverse stiffeners are used to increase the shear resistance of a girder and are aligned vertically on the web.

The term connection plate is given to a transverse stiffener to which a cross-frame or diaphragm is connected. A connection plate can serve as a transverse stiffener for shear design calculations.

As specified in **LRFD [6.10.11.1.1]**, stiffeners used as connection plates must be attached to both flanges. According to **LRFD [6.6.1.3.1]**, attachment of the connection plate to the flanges must be made by welding or bolting. When the diaphragms are connected to the transverse intermediate stiffeners, the stiffeners are welded to both the tension and compression flanges. Flange stresses are usually less than the Category C allowable fatigue stresses produced by this detail which the designer should verify.

Stiffeners in straight girders not used as connection plates are to be welded to the compression flange and tight fit to the tension flange. A tight fit can help straighten the flange tilt without the application of heat. According to **LRFD [6.10.11.1.1]**, single-sided stiffeners on horizontally curved girders should be attached to both flanges to help retain the cross-sectional shape of the girder when subjected to torsion and to avoid high localized bending within the web, particularly near the top flange due to the torsional restraint of the concrete deck. For the same reason, it is required that pairs of transverse stiffeners on horizontally curved girders be tight fit or attached to both flanges.

Indicate on the plans the flange to which stiffeners are welded. The stiffeners are attached to the web with a continuous fillet weld. See 24.6.21 for additional information on welded connections.

In the fabrication of tub sections, webs are often joined to top flanges and the connection plates and transverse stiffeners (not serving as connection plates) are installed, and then these assemblies are attached to a common box flange. The details in this case must allow the welding head to clear the bottom of the connection plates and stiffeners so the webs can be welded continuously to the box flange inside the tub section. A detail must also be provided to permit the subsequent attachment of the connection plates to the box flange (and any other transverse stiffeners that are to be attached to the box flange).

In Wisconsin, if longitudinal stiffeners are required, the transverse stiffeners are placed on one side of the web of the interior member and the longitudinal stiffener on the opposite side of the web. Place intermediate stiffeners on one side of interior members when longitudinal stiffeners are not required. Transverse stiffeners are placed on the inside web face of exterior members. If longitudinal stiffeners are required, they are placed on the outside web face of exterior members as shown on Standard for *Plate Girder Details*.

Transverse stiffeners can be eliminated by increasing the thickness of the web. On plate girders under 50" in depth, consider thickening the web to eliminate all transverse stiffeners. Within the constant depth portion of haunched plate girders over 50" deep, consider thickening the web to eliminate the longitudinal stiffener and most, but likely not all, of the transverse stiffeners within the span. The minimum size of transverse stiffeners is 5 x ½".

Transverse stiffeners are placed on the inside face of all exterior girders where the slab overhang exceeds 1'-6" as shown on Standard for *Plate Girder Details*. The stiffeners are to prevent web bending caused by construction of the deck slab where triangular overhang brackets are used to support the falsework.

If slab overhang is allowed to exceed the recommended 3'-7" on exterior girders, the web and stiffeners should be analyzed to resist the additional bending during construction of the deck. Overhang construction brackets may overstress the stiffeners. It may also be necessary to provide longitudinal bracing between stiffeners to prevent localized web deformations which did occur on a structure having 5' overhangs.

## 24.10.1 Proportions

As specified in **LRFD [6.10.11.1.2]**, the width, $b_t$, of each projecting transverse stiffener element must satisfy requirements related to the web depth, the flange width and the thickness of the projecting stiffener elements. The width, $b_t$, is illustrated in Figure 24.10-1.

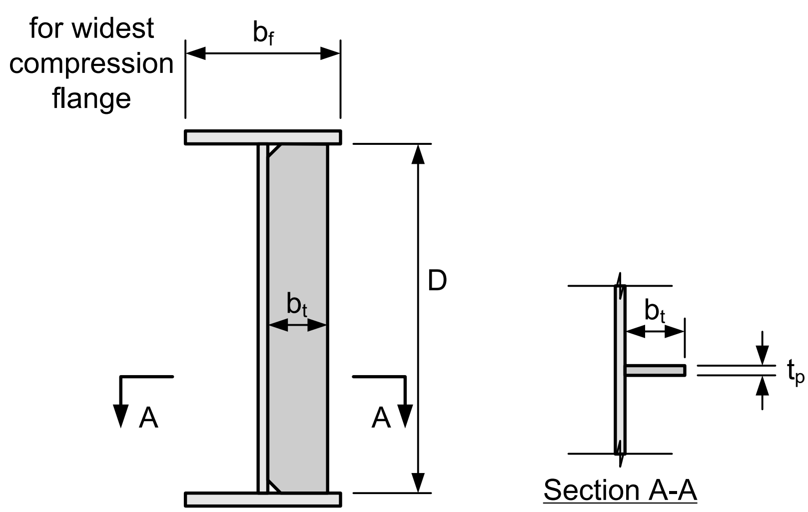


**Figure 24.10-1**
Projecting Width of Transverse Stiffeners

Fabricators generally prefer a ½" minimum thickness for stiffeners and connection plates.

## 24.10.2 Moment of Inertia

For the web to adequately develop the shear-buckling resistance, or the combined shear-buckling and post-buckling tension-field resistance, the transverse stiffener must have sufficient rigidity to maintain a vertical line of near zero lateral deflection of the web along the line of the stiffener. Therefore, the bending rigidity (or moment of inertia) is the dominant parameter governing the performance of transverse stiffeners.

As specified in **LRFD [6.10.11.1.3]**, for transverse stiffeners adjacent to web panels in which neither panel supports shear forces larger than the shear-buckling resistance, the moment of inertia of the transverse stiffener, $I_t$, must satisfy the smaller of the following two equations:

$$I_t \geq b t_w^3 J$$

and

$$I_t \geq \frac{D^4 \rho_t^{1.3}}{40}\left(\frac{F_{yw}}{E}\right)^{1.5}$$

Where:

| | | |
|---|---|---|
| $I_t$ | = | Moment of inertia of the transverse stiffener taken about the edge in contact with the web for single stiffeners and about the mid-thickness of the web for stiffener pairs (in.$^4$) |
| B | = | Smaller of $d_o$ and D (in.) |
| $d_o$ | = | Smaller of the adjacent panel widths (in.) |
| D | = | Web depth (in.) |
| $t_w$ | = | Web thickness (in.) |
| J | = | Stiffener bending rigidity parameter taken as follows: |

$$J = \frac{2.5}{\left(d_o/D\right)^2} - 2.0 \geq 0.5$$

| | | |
|---|---|---|
| $\rho_t$ | = | Larger of $F_{yw}/F_{crs}$ and 1.0 |
| $F_{yw}$ | = | Specified minimum yield strength of the web (ksi) |
| $F_{crs}$ | = | Local buckling stress for the stiffener (ksi) taken as follows: |

$$Fcrs = \frac{0.31E}{\left(b_t/t_p\right)^2} \leq F_{ys}$$

| | | |
|---|---|---|
| $F_{ys}$ | = | Specified minimum yield strength of the stiffener (ksi) |
| $b_t$ | = | Projecting width of the stiffener (in.) |

$t_p$ = Thickness of the projecting stiffener element (in.)

If the shear force in one of both panels is such that the web post-buckling or tension-field resistance is required, the moment of inertia of the transverse stiffener need only satisfy the second equation presented above.

For single-sided stiffeners, a significant portion of the web is implicitly assumed to contribute to the bending rigidity so that the neutral axis of the stiffener is assumed to be located close to the edge in contact with the web. Therefore, for this case, the moment of inertia is taken about this edge and the contribution of the web to the moment of inertia about the neutral axis is neglected for simplicity.

Transverse stiffeners used in panels with longitudinal web stiffeners must also satisfy the following relationship:

$$I_t \geq \left(\frac{b_t}{b_\ell}\right)\left(\frac{D}{3d_o}\right)I_\ell$$

Where:

$I_t$ = Moment of inertia of the transverse web stiffener taken about the edge in contact with the web for single stiffeners and about the mid-thickness of the web for stiffener pairs ($in.^4$)

$b_t$ = Projecting width of the transverse stiffener (in.)

$b_\ell$ = Projecting width of the longitudinal stiffener (in.)

$D$ = Web depth (in.)

$d_o$ = Smaller of the adjacent web panel widths (in.)

$I_\ell$ = Moment of inertia of the longitudinal stiffener determined as specified in **LRFD [6.10.11.3.3]** ($in.^4$)

Additional information and equations used for LRFD design of transverse intermediate stiffeners are presented in **LRFD [6.10.11.1]**. In addition, a design example for transverse intermediate stiffeners is also provided in this *Bridge Manual*.

## 24.11 Longitudinal Stiffeners

The design of longitudinal web stiffeners is specified in **LRFD [6.10.11.3]**. Longitudinal stiffeners are aligned horizontally on the web along the length of the girder and divide the web panel into smaller sub-panels. As specified in **LRFD [6.10.2.1]**, longitudinal stiffeners are required whenever the web slenderness $D/t_w$ exceeds 150. They are used to provide additional bend-buckling resistance to the webs of deeper girders. Longitudinal stiffeners, where required, are to consist of a plate welded to one side of the web or a bolted angle.

As specified in **LRFD [6.10.11.3.1]**, longitudinal stiffeners are to be located vertically on the web such that adequate web bend-buckling resistance is provided for constructability and at the service limit state. It also must be verified that the section has adequate nominal flexural resistance at the strength limit state with the longitudinal stiffener in the selected position.

At composite sections in negative flexure and non-composite sections, it is recommended that the longitudinal stiffener initially be located at $0.4D_c$ from the inner surface of the compression flange. For composite sections in negative flexure, $D_c$ would be conservatively calculated for the section consisting of the steel girder plus the longitudinal reinforcement. For non-composite sections, $D_c$ would be based on the section consisting of the steel girder alone. As a preliminary approximation, a distance of 1/5 of the depth of the web may be used as the distance from the longitudinal stiffener to the inner surface of the compression flange.

On the exterior members, the longitudinal stiffeners are placed on the outside face of the web as shown on Standard for *Plate Girder Details*. If the longitudinal stiffener is required throughout the length of span on an interior member, the longitudinal stiffener is placed on one side of the web and the transverse stiffeners on the opposite side of the web. Longitudinal stiffeners are normally used in the haunch area of long spans and on a selected basis in the uniform depth section.

Where longitudinal stiffeners are used, place intermediate transverse stiffeners next to the web splice plates at a field splice. The purpose of these stiffeners is to prevent web buckling before the girders are erected and spliced.

In some cases, particularly in regions of stress reversal, it may be necessary or desirable to use two longitudinal stiffeners on the web. It is possible to have an overlap of longitudinal stiffeners near the top flange and near the bottom flange due to the variation between maximum positive and maximum negative moment.

It is preferred that longitudinal stiffeners be placed on the opposite side of the web from transverse stiffeners. At bearing stiffeners and connection plates where the longitudinal stiffener and transverse web element must intersect, a decision must be made as to which element to interrupt. According to **LRFD [6.10.11.3.1]**, wherever practical, longitudinal stiffeners are to extend uninterrupted over their specified length, unless otherwise permitted in the contract documents, since longitudinal stiffeners are designed as continuous members to improve the web bend buckling resistance. In such cases, the interrupted transverse elements must be fitted and attached to both sides of the longitudinal stiffener with connections sufficient to develop the flexural and axial resistance of the transverse element. If the longitudinal stiffener is interrupted instead, it should be similarly attached to all transverse elements. All interruptions must be carefully designed with respect to fatigue, especially if the longitudinal stiffener is not attached to the transverse web elements, as a Category E or E′ detail may exist at the termination points of each longitudinal stiffener-to-web weld. Copes should always be provided to avoid intersecting welds.

Longitudinal stiffeners are subject to the same flexural strain as the web at their vertical position on the web. As a result, the stiffeners must have sufficient strength and rigidity to resist bend buckling of the web (at the appropriate limit state) and to transmit the stresses in the stiffener and an effective portion of the web as an equivalent column. Therefore, as specified in **LRFD [6.10.11.3.1]**, the flexural stress in the longitudinal stiffener due to the factored loads, $f_s$, must satisfy the following at the strength limit state and when checking constructability:

$$f_s \le \phi_f R_h F_{ys}$$

Where:

| | | |
|---|---|---|
| $\phi_f$ | = | Resistance factor for flexure specified in **LRFD [6.5.4.2]** (= 1.0) |
| $R_h$ | = | Hybrid factor specified in **LRFD [6.10.1.10.1]** |
| $F_{ys}$ | = | Specified minimum yield strength of the longitudinal stiffener (ksi) |

## 24.11.1 Projecting Width

As specified in **LRFD [6.10.11.3.2]**, the projecting width, $b_\ell$, of the longitudinal stiffener must satisfy the following requirement in order to prevent local buckling of the stiffener plate:

$$b_\ell \le 0.48 t_s \sqrt{\frac{E}{F_{ys}}}$$

Where:

| | | |
|---|---|---|
| $t_s$ | = | Thickness of the longitudinal stiffener (in.) |
| $F_{ys}$ | = | Specified minimum yield strength of the stiffener (ksi) |

## 24.11.2 Moment of Inertia

As specified in **LRFD [6.10.11.3.3]**, to ensure that a longitudinal stiffener will have adequate rigidity to maintain a horizontal line of near zero lateral deflection in the web to resist bend buckling of the web (at the appropriate limit state), the moment of inertia of the stiffener acting in combination with an adjacent strip of web must satisfy the following requirement:

$$I_\ell \ge D t_w^3 \left[ 2.4 \left( \frac{d_o}{D} \right)^2 - 0.13 \right] \beta$$

Where:

$I_\ell$ = Moment of inertia of the longitudinal stiffener including an effective width of the web equal to $18t_w$ taken about the neutral axis of the combined section (in.$^4$). If $F_{yw}$ is smaller than $F_{ys}$, the strip of web included in the effective section must be reduced by the ratio of $F_{yw}/F_{ys}$.

D = Web depth (in.)

$t_w$ = Web thickness (in.)

$d_o$ = Transverse stiffener spacing (in.)

$\beta$ = Curvature correction factor for longitudinal stiffener rigidity (equal to 1.0 for longitudinal stiffeners on straight webs)

Longitudinal stiffeners on horizontally curved webs require greater rigidity than on straight webs because of the tendency of curved webs to bow. This is reflected by including the factor $\beta$ in the above equation, which is a simplification of a requirement for longitudinal stiffeners on curved webs. For longitudinal stiffeners on straight webs, $\beta$ equals 1.0.

The moment of inertia (and radius of gyration) of the longitudinal stiffener is taken about the neutral axis of an equivalent column cross section consisting of the stiffener and an adjacent strip of web with a width of $18t_w$.

## 24.11.3 Radius of Gyration

As specified in **LRFD [6.10.11.3.3]**, to ensure that the longitudinal stiffener acting in combination with an adjacent strip of web as an effective column section can withstand the axial compressive stress without lateral buckling, the radius of gyration, r, of the effective column section must satisfy the following requirement:

$$r \geq \frac{0.16 d_o \sqrt{\frac{F_{ys}}{E}}}{\sqrt{1 - 0.6\frac{F_{yc}}{R_h F_{ys}}}}$$

Where:

r = Radius of gyration of the longitudinal stiffener including an effective width of the web equal to $18t_w$ taken about the neutral axis of the combined section (in.)

$d_o$ = Transverse stiffener spacing (in.)

$F_{ys}$ = Specified minimum yield strength of the longitudinal stiffener (ksi)

$F_{yc}$ = Specified minimum yield strength of the compression flange (ksi)

$R_h$ = Hybrid factor determined as specified in **LRFD [6.10.1.10.1]**

Additional information and equations used for LRFD design of longitudinal stiffeners are presented in **LRFD [6.10.11.3]**.

## 24.12 Construction

When the deck slab is poured, the exterior girder tends to rotate between the diaphragms. This problem may result if the slab overhang is greater than recommended and/or if the girders are relatively shallow in depth. This rotation causes the rail supporting the finishing machine to deflect downward and changes the roadway grade unless the contractor provides adequate lateral timber bracing.

Stay-in-place steel forms are not recommended for use. Steel forms have collected water that permeates through the slab and discharges across the top flanges of the girders. As a result, flanges frequently corrode. Since there are cracks in the slab, this is a continuous problem.

Where built-up box sections are used, full penetration welds provide a stronger joint than fillet welds and give a more aesthetically pleasing appearance. However, they are also more costly.

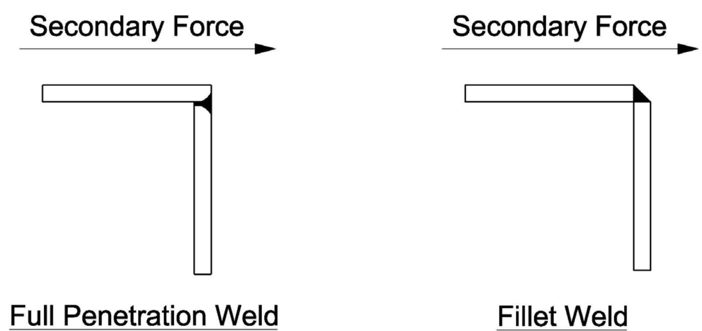


**Figure 24.12-1**
Welds for Built-up Box Sections

The primary force of the member is tension or compression along the axis of the member. The secondary force is a torsional force on the member cross section which produces a shearing force across the weld.

During construction, holes may be drilled in the top flanges in the compression zone to facilitate anchorage of posts for safety lines. The maximum hole size is 3/4" diameter, and prior to pouring the concrete deck, a bolt must be placed in each hole.

**LRFD [6.10.3]** describes the constructability design requirements for a steel girder bridge. Provisions are provided for the following constructability checks:

- Nominal yielding
- Reliance on post-buckling resistance
- Potential uplift at bearings
- Webs without bearings stiffeners
- Holes in tension flanges

- Load-resisting bolted connections
- Flexure in discretely braced flanges
- Flexure in continuously braced flanges
- Shear in interior panels of webs with transverse stiffeners
- Dead load deflections

## 24.12.1 Web Buckling

The buckling behavior of a slender web plate subject to pure bending is similar to the buckling behavior of a flat plate. Through experimental tests, it has been observed that web bend-buckling behavior is essentially a load-deflection rather than a bifurcation phenomenon; that is, a distinct buckling load is not observed.

Since web plates in bending do not collapse when the theoretical buckling load is reached, the available post-buckling strength can be considered in determining the nominal flexural resistance of sections with slender webs at the strength limit state. However, during the construction condition, it is desirable to limit the bending deformations and transverse displacements of the web.

The advent of composite design has led to a significant reduction in the size of compression flanges in regions of positive flexure. As a result, more than half of the web of the non-composite section will be in compression in these regions during the construction condition before the concrete deck has hardened or is made composite. As a result, the web is more susceptible to bend-buckling in this condition.

To control the web plate bending strains and transverse displacements during construction, *AASHTO LRFD* uses the theoretical web bend-buckling load as a simple index. The web bend-buckling resistance, $F_{crw}$, is specified in **LRFD [6.10.1.9.1]** as follows:

$$Fcrw = \frac{0.9Ek}{\left(D/t_w\right)^2}$$

Where:

| | | |
|---|---|---|
| E | = | Modulus of elasticity of the steel (ksi) |
| K | = | Bend-buckling coefficient (see below) |
| D | = | Depth of web (in.) |
| $t_w$ | = | Thickness of web (in.) |

For webs without longitudinal stiffeners, the bend-buckling coefficient, k, is as follows:

$$k = \frac{9}{(D_c/D)^2}$$

Where:

$D_c$ = Depth of web in compression in the elastic range (in.)

$F_{crw}$ is not to exceed the smaller of $R_hF_{yc}$ and $F_{yw}/0.7$, where $F_{yc}$ and $F_{yw}$ are the specified minimum yield strengths of the compression flange and web, respectively, and $R_h$ is the hybrid factor.

According to **LRFD [6.10.3.2]**, the maximum compression-flange stress in a non-composite I-section due to the factored loads, calculated without consideration of flange lateral bending, must not exceed the resistance factor for flexure, $\phi_f$, times $F_{crw}$ for all critical stages of construction. This requirement also applies at sections where top flanges of tub girders are subject to compression during construction. For closed-box sections, **LRFD [6.11.3.2]** states that the maximum longitudinal flange stress due to the factored loads, calculated without consideration of longitudinal warping, must not exceed $\phi_fF_{crw}$ at sections where non-composite box flanges are subject to compression during construction. (A box flange is defined in *AASHTO LRFD* as a flange connected to two webs.) For tub or closed-box sections with inclined webs, $D_c$ should be taken as the depth of the web in compression measured along the slope (that is, $D_c$ divided by the cosine of the angle of inclination of the web plate with respect to the vertical) when computing $F_{crw}$. Should $F_{crw}$ be exceeded for the construction condition, the engineer has several options to consider:

- Provide a larger compression flange or a smaller tension flange to reduce $D_c$.
- Adjust the deck-placement sequence to reduce the compressive stress in the web.
- Provide a thicker web.
- As a last resort, should the previous options not prove practical or cost-effective, provide a longitudinal web stiffener.

## 24.12.2 Deck Placement Analysis

Depending on the length of the bridge, the construction of the deck may require placement in sequential stages. Therefore, certain sections of the steel girders will become composite before other sections. If certain placement sequences are followed, temporary moments induced in the girders during the deck placement can be significantly higher than the final non-composite dead load moments after the sequential placement is complete.

Therefore, **LRFD [6.10.3.4]** requires that sections in positive flexure that are non-composite during construction but composite in the final condition must be investigated for flexure according to the provisions of **LRFD [6.10.3.2]** during the various stages of the deck placement. Furthermore, changes in the load, stiffness and bracing during the various stages are to be considered in the analysis.

**Example:**

Consider the sample deck placement shown in Figure 24.12-2 for a three-span continuous bridge. The deck placement sequence is based on Standard for *Slab Pouring Sequence*.

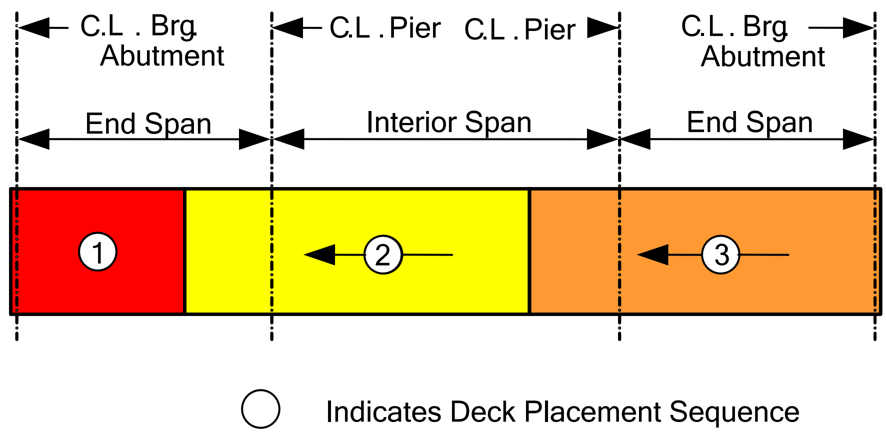


**Figure 24.12-2**
Deck Placement Sequence

Figure 24.12-3 through Figure 24.12-6 show elevation views of a girder which will be used to show the results for each stage of the deck placement sequence assumed for this example in Figure 24.12-2. In Figure 24.12-3, the girders are in place but no deck concrete has yet been placed. The entire girder length is non-composite at this stage. Before the deck is placed, the non-composite girder must resist the moments due to the girder self-weight and any additional miscellaneous steel weight. The moments due to these effects are shown at Location A, which is the location of maximum positive moment in the first end span.

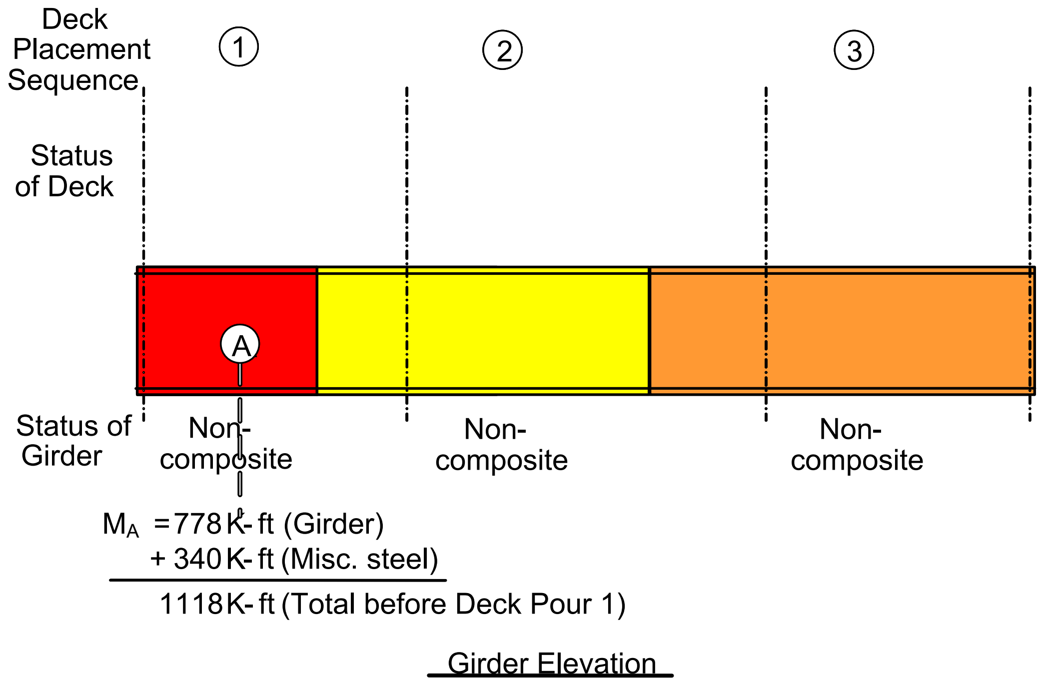


**Figure 24.12-3**
Girder Elevation View

Figure 24.12-4 shows the first deck placement (Cast 1), which is cast in the first portion of the first span. The moment due to the wet concrete load, which consists of the weight of the deck and deck haunches, is added to the moments due to the girder self-weight and miscellaneous steel weight. Since the concrete in this first placement has not yet hardened, the moment due to the first deck placement is resisted by the non-composite girder. The cumulative positive moment in the girder at Location A after the first deck placement is +3,565 kip-ft, which is the maximum positive moment this section will experience during the assumed placement sequence. This moment is larger than the moment of +3,542 kip-ft that would be computed at this location assuming a simultaneous placement of the entire deck (that is, ignoring the sequential stages).

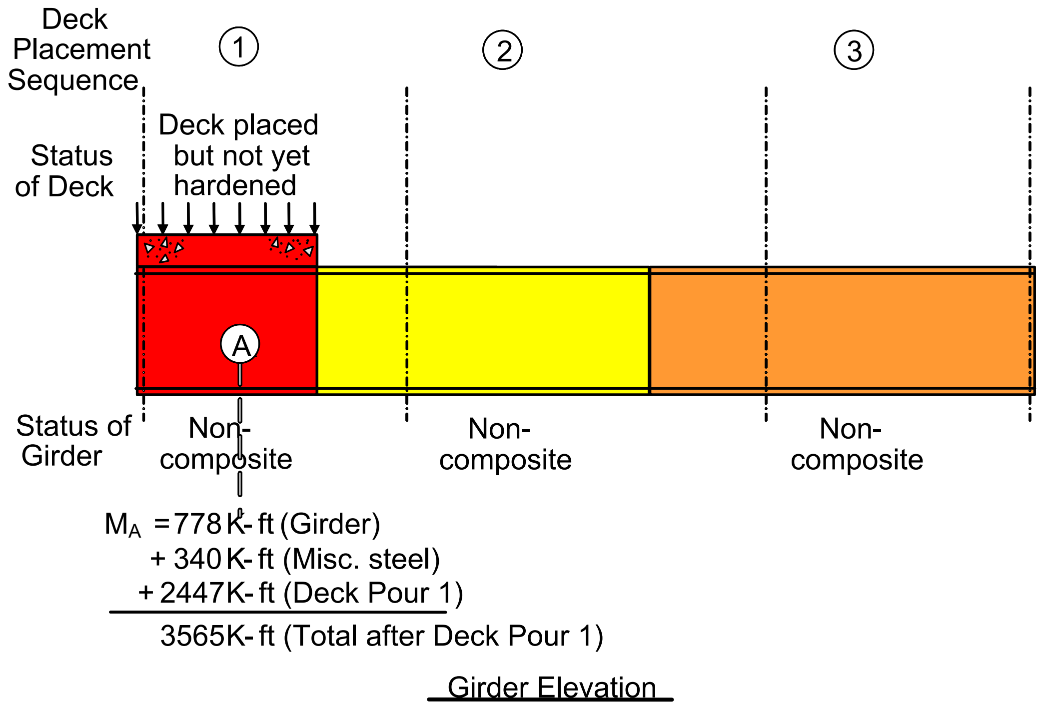


**Figure 24.12-4**
Deck Placement Analysis 1

The next deck placement (Cast 2) is located immediately adjacent to Cast 1, as shown in Figure 24.12-5. The concrete in the first placement is now assumed to be hardened so that those portions of the girder are now composite. Therefore, as required in **LRFD [6.10.3.4]**, those portions of the girder are assumed composite in the analysis for this particular deck placement. The remainder of the girder is non-composite. Since the deck casts are relatively short-term loadings, the short-term modular ratio, n, is used to compute the composite stiffness. The previous casts are assumed to be fully hardened in this case, but adjustments to the composite stiffness to reflect the actual strength of the concrete in the previous casts at the time of this particular placement could be made, if desired. The cumulative moment at Location A has decreased from +3,565 kip-ft after Cast 1 to +3,449 kip-ft after Cast 2, because the placement in Cast 2 causes a negative moment in the end spans.

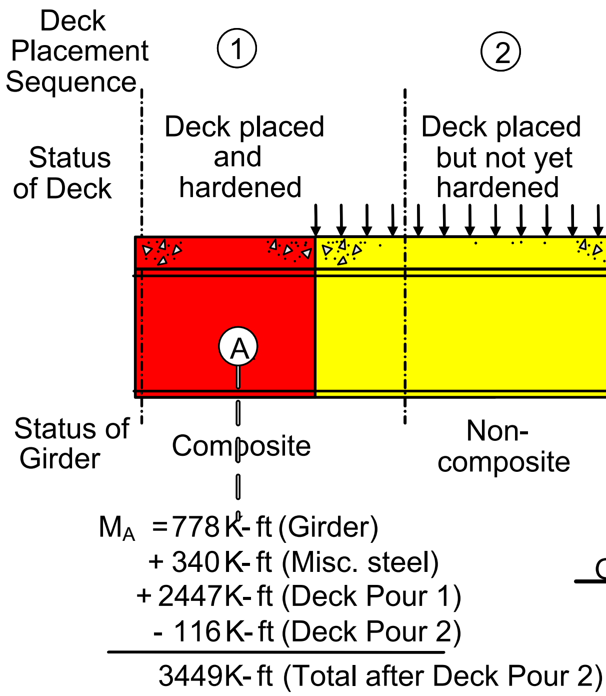


**Figure 24.12-5**
Deck Placement Analysis 2

The last deck placement (Cast 3) is located immediately adjacent to Cast 2, as presented in Figure 24.12-6. Again, the concrete in Casts 1 and 2 is assumed to be fully hardened in the analysis for Cast 3. The cumulative moment at Location A has increased slightly from +3,449 kip-ft to +3,551 kip-ft, which is less than the moment of +3,565 kip-ft experienced at Location A after Cast 1.

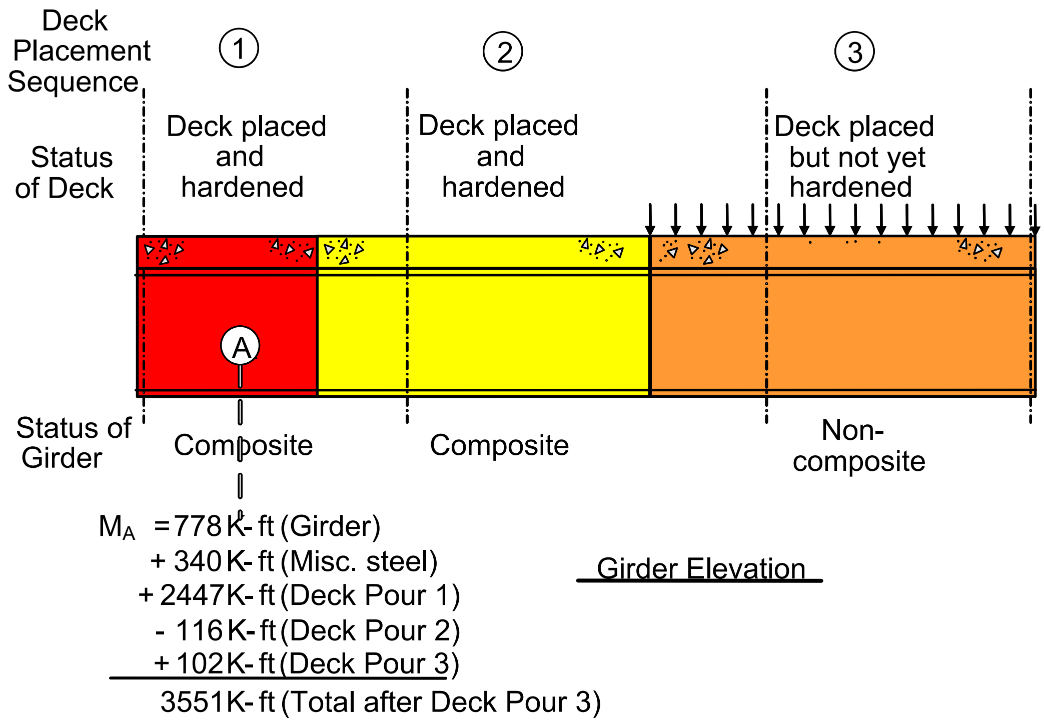


**Figure 24.12-6**
Deck Placement Analysis 3

Table 24.12-1 shows a more complete set of the unfactored dead-load moments in the end span (Span 1) from the abutment to the end of Cast 1 computed from the example deck placement analysis. Data are given at 19.0-foot increments along the span, measured from the abutment. The end of Cast 1 is located 102.5 feet from the abutment, based on the requirements of Standard for *Slab Pouring Sequence*. Location A is 76.0 feet from the abutment. In addition to the moments due to each of the individual casts, Table 24.12-1 gives the moments due to the steel weight and the additional miscellaneous steel. Also included are the sum of the moments due to the three casts and the moments due to the weight of the concrete deck and haunches assuming that the concrete is placed simultaneously on the non-composite girders instead of in sequential steps. The maximum moment occurs after Cast 1.

| Length (ft) | 0.0 | 19.0 | 38.0 | 57.0 | 76.0 | 95.0 |
|---|---|---|---|---|---|---|
| Steel Weight | 0 | 400 | 663 | 789 | 778 | 630 |
| Additional Miscellaneous Steel | 0 | 166 | 278 | 336 | 340 | 290 |
| Cast 1 | 0 | 1190 | 1994 | 2413 | 2447 | 2096 |
| Cast 2 | 0 | -29 | -58 | -87 | -116 | -145 |
| Cast 3 | 0 | 25 | 51 | 76 | 102 | 127 |
| Sum of Casts | 0 | 1186 | 1987 | 2402 | 2433 | 2078 |
| Deck & Haunches (Simultaneous Cast) | 0 | 1184 | 1983 | 2396 | 2424 | 2067 |

**Table 24.12-1**
Moments from Deck Placement Analysis (K-ft)

The slight differences in the moments on the last line of Table 24.12-1 (assuming a simultaneous placement of the entire slab) and the sum of the moments due to the three casts are due to the changes in the girder stiffness with each sequential cast. The principle of superposition does not apply directly in the deck-placement analyses, since the girder stiffness changes at each step of the analysis. Although the differences in the moments are small in this example, they can be significantly greater depending on the span configuration. The effects of the deck placement sequence must be considered during design.

In regions of positive flexure, the non-composite girder should be checked for the effect of the maximum accumulated deck-placement moment. This moment at 76 feet from Abutment 1 is computed as:

$$M = 778 + 340 + 2{,}447 = 3{,}565 \text{ kip-ft}$$

This value agrees with the moment at this location shown in Figure 24.12-4.

In addition to the dead load moments during the deck placement, unfactored dead load deflections and reactions can also be investigated similarly during the construction condition.

When investigating reactions during the construction condition, if uplift is found to be present during deck placement, the following options can be considered:

- Rearrange the concrete casts.
- Specify a temporary load over that support.
- Specify a tie-down bearing.

- Perform another staging analysis with zero bearing stiffness at the support experiencing lift-off.

## 24.13 Painting

The final coat of paint on all steel bridges shall be an approved color. Exceptions to this policy may be considered on an individual basis for situations such as scenic river crossings, unique or unusual settings or local community preference. The Region is to submit requests for an exception along with the Structure Survey Report. The *AMS Standard Color Numbers* available for use on steel structures are shown in Chapter 9 - Materials.

Unpainted weathering steel is used on bridges over streams and railroads. All highway grade separation structures require the use of painted steel, since unpainted steel is subject to excessive weathering from salt spray distributed by traffic. On weathering steel bridges, the end 6' of any steel adjacent to either side of an expansion joint and/or hinge is required to have two shop coats of paint. The second coat is to be brown color similar to rusted steel. Do not paint the exterior face of the exterior girders for aesthetic reasons, but paint the hanger bar on the side next to the web. Additional information on painting is presented in Chapter 9 - Materials.

For painted steel plate I-girders utilize a three-coat system defined by the Standard Specification bid item “Painting Epoxy System (Structure)”. For painted tub girders use a two-coat system defined by the STSP “Painting Polysiloxane System (Structure)”, which includes painting of the inside of the tubs.

Paint on bridges affects the slip resistance of bolted connections. Since faying surfaces that are not galvanized are typically blast-cleaned as a minimum, a Class A surface condition should only be used to compute the slip resistance when Class A coatings are applied or when unpainted mill scale is left on the faying surface. Most commercially available primers will qualify as Class B coatings.

## 24.14 Floor Systems

In the past, floor systems utilizing two main girders were used on long span structures. Current policy is to use multiple plate girder systems for bridges having span lengths up to 400'. Multiple girder systems are preferred since they are redundant; that is, failure of any single member will not cause failure of the structure.

In a two-girder system, the main girders are designed equally to take the dead load and live load unless the roadway cross section is unsymmetrical. The dead load and live load carried by the intermediate stringers is transferred to the floor beams, which transmit the load to the main girders. In designing the main girders, it is an acceptable practice to assume the same load distribution along the stringers as along the girder and ignore the concentrated loads at the floor beam connections.

The design criteria used for such girders is the same as the criteria used for plate girders and rolled sections. Particular attention should be paid to the sufficiency of the girder connection details and to the lateral bracing requirements and connections.

### 24.14.1 Redundancy in Floor Systems

Per the Wisconsin Structures Inspection Manual (SIM) section 1.3.5 floor beams that are spaced greater than 14 feet shall receive NSTM-level inspections. Per the FHWA memo “Inspection of Nonredundant Steel Tension Members” dated May 9th, 2022, state transportation departments may develop procedures in accordance with the NBIS to demonstrate that a member without load path redundancy has system or internal redundancy such that it is not considered a NSTM.

BOS selectively applies the *AASHTO Guide Specification for Analysis and Identification of Fracture Critical Members and System Redundant Members* (SRM Guide Spec) for the evaluation of floor beams spaced greater than 14 feet, for the purpose of demonstrating that adequate system redundancy exists to relieve the NSTM inspection requirements. Additional requirements for inspection and classification can be found in the SIM 1.3.5.7-2. BOS approval is required prior to undertaking any redundancy analyses.

As a supplement to the SRM Guide Spec, BOS has developed and gained FHWA approval on the use of a simplified analysis method to assist in the evaluation of floor beams for system redundancy.

### 24.14.2 Simplified System Redundancy Analysis

The simplified system redundancy analysis shall only apply to Girder-Floor beam-Stringer (GFS) systems with the following:

1. The stringers are continuous over the floor beams, and
2. The floor beams are not composite with the deck

The analysis shall adhere to the provisions within the SRM Guide Spec. with the following exceptions or analytical simplifications provided herein.

Floor beam fracture differs from other fracture types, such as the fracture of a longitudinal girder, in that the effects of the fracture tend to be more localized and rely less on system redistribution. The fracture of a floor beam effectively removes or reduces the support provided to the stringers and deck and increases the effective span length of those elements. It is those elements that require evaluation to verify sufficient strength exists in the faulted state. The remaining intact floor beams should be investigated as to whether the demand from an adjacent fracture exceeds that of the original design demand. The mainline girders in a GFS system are largely unaffected, as the loads will shift to the adjacent floor beams or directly through the deck. Therefore, they are not a focus of the system redundancy evaluation.

GFS systems commonly exhibit out-of-plane distortion cracking, particularly at the floor beam connections to the main girders. Floor beam fracture may lead to increased distortion in the web gap region and may initiate or accelerate cracking. However, the risk of fracture in the girder is low as these cracks tend to be along the direction of primary stresses initially, and when they do change direction, it is orientated away from the flanges. The existence of distortion induced cracking is not an automatic disqualifier for SRM status if the procedures for the detection and mitigation of cracking is in place (refer to SIM 1.3.5.7-2).

In situations where the floor beam is not directly connected to the deck and acts as a support for the stringers, then explicit consideration of contact friction and shear studs within the analytical model are not required. Primarily because the member that is faulted does not rely on those interactions to transfer load. Instead, the analysis considers the stringer support to be greatly reduced or removed entirely due to the fracture of the floor beam and the load transfer mechanism is broken. At this point it is sufficient to consider member-force effects in the stinger similar to how one might consider shored construction with removal of supports.

Fracturing an exterior bay floor beam produces the most severe response due to the relative flexibility of an exterior girder to rotate compared to an interior girder. For floor beams carrying a single stringer, the fracture should be modeled such that the floorbeam is considered a complete loss of support to the stringer. When multiple stringers are carried by a floorbeam, the fracture may be modeled such that the floor beam is not completely removed but acts as two cantilevers. In this scenario, consideration should be given to the floor beam to girder connection and the potential for the floor beam to buckle. If those resulting forces are significant, then consideration should be given for the full removal of the floor beam in the analysis.

In situations where the bridge cross section varies along the length of the bridge and the number and/or position of striped lanes changes, Redundancy I load combination may consider the use of *design lanes* in determining the controlling load effect.

The Concrete Damage Plasticity (CDP) Model accounts for residual strains due to plastic deformation as well as the loss of elastic stiffness (decay in modulus of elasticity or "damage") in the load-unload cycle from cracks developing in the concrete as depicted below. This applies to both the tensile and compressive response. However, the CDP model is only supported by high end FEA software. Other concrete material models may be used depending on software capabilities. A nonlinear elastic model with monotonic loading (following smooth black line below) is a reasonable approach provided the concrete model accounts for little or no tensile resistance from the concrete and the compressive strains from loading are below the compressive strain of concrete at a uniaxial compressive stress equal to f'c (peak stress). The concrete compressive stress-strain curve is developed per SRM 4.2.1.

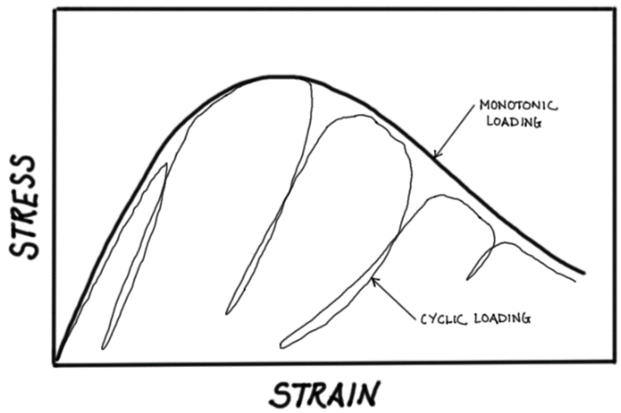


**Figure 24.14-1**
Typical Concrete Compressive Stress-Strain Curve

The explicit modeling of shear studs is not required. If the stringers are composite with the deck, full composite behavior in the analysis may be assumed if the shear stud design satisfies:

a) The shear stud design detail screening per SRM A3.2, or
b) Section 6.10.10.4 of the current AASHTO BDS

The analysis should include a 5 PSF future wearing surface to allow for a future epoxy overlay or any other minor increases to the structure's dead load.

The flexibility and type of substructure has very little impact on the behavior of the components in the locally impacted area and does not need to be explicitly modeled. The displacement degree-of-freedom from bearings should be considered if the floor beam fracture and affected components being evaluated are near a support (i.e. negative moment region).

Because floor beam fracture tends to be more localized, member evaluation based on force-effect criteria may be more practical than stress-strain criteria when evaluating the structurally intact members, specifically the stringers.

The fracture of a floor beam does not influence the behavior of an adjacent floor beam significantly. This is because the design loading for a floor beam is controlled by axle loading directly above the particular floor beam. And while service loading in the faulted state will always be greater than design service loading, the consideration of strength versus redundancy load factors makes this difference less significant. A simple moving load analysis should be used to confirm that the factored demands in the faulted state do not exceed that of the design loads for intact floor beams. If demands in the faulted state are higher than the design load demands, then the adjacent floor beams shall be evaluated to ensure they have adequate strength.

If using force-effect criteria, determining the effective widths for composite stringers should be done following a procedure similar to that described in the FHWA *Manual for Refined Analysis in Bridge Design and Evaluation* section 8.2.3. Depending on mesh size, the user should ensure the analytical results (or demand) consider a section cut that is equal to or larger than the effective width used for resistance calculations. The deck may be modeled with shell elements utilizing Gauss integration.

If using force-effect criteria, member resistance should be determined using the current AASHTO BDS. Careful attention should be given to the increased unbraced length of the stringer for compression in the bottom flange. The calculation of Cb via AASHTO will likely result in a very low compressive stress resistance. Because this factor was developed and intended for uniform moment gradients (plate girders with cross bracing), more refined methods for calculating Cb are recommended. The current AISC Steel Construction Manual Chapter F commentary provides a number of alternative methods, specifically equation C-F1-5, that may be used when the top flange of rolled W-shapes are continuously braced.

The calculated resistance of steel components should include a 10% reduction of section in the critical component to represent future section loss.

For stringers, special consideration should be given to bolted field splice connections. At field splices, the model should consider only the flanges and deck as capable of providing flexural resistance (shear only for web elements). Because the shear and moment diagram changes due to the loss of support of the floor beam, the original design assumption of placing a field splice near points of dead load contraflexure may no longer be valid. If the flange splice alone is insufficient to meet the flexural demands, the web resistance may be considered as well per LRFD 6.13.6.1.3. The analyst may consider additional load paths beyond failure of a bolted field splice, but explicit modeling of shear studs and contact friction is required.

If the concrete material model does not consider damage plasticity or an equivalent model, the concrete compression strain shall be conservatively limited to that strain corresponding to the uniaxial compressive stress equal to f'c, determined from equation 4.2.1-4 (SRM). This ensures a load redistribution mechanism remains largely unrealized in the concrete deck.

Special consideration should be given to the lapping of reinforcement bars within the concrete deck. Similar to that of the stringers, the demands in the deck have shifted and/or increased. The placement of bar laps in the original design may have been placed intentionally in areas of low stress. In addition, the length of lap required has changed over time and existing bar laps may not be fully developed. In these cases, a reduced allowable strain should be used proportional to the actual lap versus lap required for the bar to reach yield stress.

## 24.15 Box Girders

Box girders present a smooth, uncluttered appearance under the bridge deck due to the lack of transverse bracing and due to their closed section. Enhanced torsional rigidity can make box girders a favorable choice for horizontally curved bridges. However, due to redundancy concerns, use of single-box and two-box girder bridges should be avoided unless absolutely necessary.

In the design of box girders, the concrete slab is designed as a portion of the top flange and also as the support between the two girder webs which satisfies the requirement for being considered a closed box section.

Current experience shows that box girders may require more material than conventional plate girders. On longer spans, additional bracing between girders is required to transfer lateral loads.

Several requirements in *AASHTO LRFD* are specific to box girders. For box girders, sections in positive flexure shall not have a yield strength in excess of 70 ksi. The following web slenderness requirement from **LRFD [6.11.6.2.2]** must also be satisfied:

$$\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yc}}}$$

Where:

| | | |
|---|---|---|
| $D_{cp}$ | = | Depth of web in compression at plastic moment (in.) |
| $t_w$ | = | Web thickness (in.) |
| $F_{yc}$ | = | Specified minimum yield strength of the compression flange (ksi) |

Other requirements for positive flexure in box girders are presented in **LRFD [6.11.6.2.2]**. Steel sections in negative flexure must not use the provisions in Appendices A or B of the *AASHTO LRFD* specifications.

When computing effective flange widths for closed-box sections, the distance between the outside of the webs at the tops is to be used in lieu of the web thickness in the general requirements. For closed box sections, the spacing should be taken as the spacing between the centerlines of the box sections.

When computing section properties for closed-box sections with inclined webs, the moment of inertia of the webs about a horizontal axis at the mid-depth of the web should be adjusted for the web slope by dividing by the cosine of the angle of inclination of the web plate to the vertical. Also, inspection manholes are often inserted in the bottom flanges of closed-box sections near supports. These manholes should be subtracted from the bottom-flange area when computing the elastic section properties for use in the region of the access hole. If longitudinal flange stiffeners are present on the closed-box section, they are often included when computing the elastic section properties.

When investigating web bend-buckling resistance for closed-box sections, **LRFD [6.11.3.2]** states that the maximum longitudinal flange stress due to the factored loads, calculated without consideration of longitudinal warping, must not exceed $\phi_f F_{crw}$ at sections where non-composite box flanges are subject to compression during construction. For more information about the web bend-buckling resistance of box girders, refer to 24.12.1. In *AASHTO LRFD*, a box flange is defined as a flange connected to two webs.

Torsion in structural members is generally resisted through a combination of St. Venant torsion and warping torsion. For closed cross-sections such as box girders, St. Venant torsion generally dominates. Box girders possess favorable torsional characteristics which make them an attractive choice for horizontally curved bridges. However, due to redundancy concerns, use of single-box and two-box girder bridges should be avoided unless absolutely necessary.

**WisDOT policy item:**

Certain criteria must be met to consider a trapezoidal steel box girder bridge to be a System Redundant Member (SRM), as outlined in *A Simplified Approach for Designing SRMs in Composite Continuous Twin-Tub Girder Bridges* (as summarized in Appendix A – the full report is available upon request from BOS) by Robert J. Connor, et. al., Purdue University. A summary of these steps required by WisDOT are outlined below this policy item box.

It is required to design twin-tub girders to meet SRM criteria. BOS approval is required for all box girders.

Summary of Appendix A

Approach
For a multi-span twin-tub girder bridge to be considered an SRM, the bridge must meet certain screening criteria. If the criteria are met, design must be in accordance with the provisions set forth in the subject report. Figure A-1 is a flowchart for describing the proposed guideline steps.

Screening
To consider a twin-tub girder an SRM, certain criteria must be met, which require continuous spans, composite section with specific shear stud design, maximum bridge width, maximum girder spacing, web depth range, interior span length limits, exterior span length limits, ratio of unfractured to fractured span length limits, ratio of radius of curvature to longest span length limit, skew limit, maximum number of design lanes, and maximum dead load displacement limit at both interior and exterior spans.

Design Methodology
If the screening criteria are met, the design then needs to meet specific design requirements for shear studs, intermediate diaphragms, bottom flange buckling resistance, and positive moment flexural resistance.

Additional information regarding design and rating includes:

New twin steel tub girder designs should continue to include the redundancy load factor (**LRFD [1.3.4]**) for nonredundant members, $\eta_R$ = 1.05 under the strength limit state, regardless of the structure's final redundant related classification (e.g. FCM or SRM). The continued use of this load factor, even if a structure is determined to be redundant via system redundant classification is to maintain consistency in design with the original group of structures evaluated and documented in the report by Purdue University.

However, the load redundancy factor shall not be considered when checking the *Redundancy I and II* limit states described in the aforementioned report.

For load ratings, the *Manual for Bridge Evaluation*, section 6A.4.2.4 applies a system factor $\phi_S$ = 0.85 to the resistance of welded members in two-girder systems (i.e. twin steel tub girders). If a twin steel tub girder bridge has achieved SRM classification the system factor should be taken as 1.0 for load rating purposes.

## 24.16 Design Examples

## Table of Contents

## 28.1 Introduction

Many structures have joints that must be properly designed and installed to insure their integrity and serviceability. Bridges as well as highway pavements, airstrips, buildings, etc. need joints to take care of expansion and contraction caused by temperature changes. However, bridges expand and contract more than pavement slabs or buildings and have their own special types of expansion devices.

Current practice is to limit the number of bridge expansion joints. This practice results in more movement at each joint. There are so many potential problems associated with joints that fewer joints are recommended practice. Expansion joints are placed on the high end of a bridge if only one joint is placed on the bridge. This is done to prevent the bridge from creeping downhill and to minimize the amount of water passing over the joint.

Open joints generally lead to future maintenance. Water and debris fall through the joint. Water running through an open joint erodes the soil under the structure, stains the bent cap and columns, and leads to corrosion of adjacent girders, diaphragms, and bearings. During freeze-thaw conditions, large icicles may form under the structure or ice may form on the roadway presenting a traffic hazard. Debris acts with water in staining the substructure units and plugs the drainage systems.

In the past, open steel finger type joints were used on long span bridges where large movements encountered. Finger joints were placed in the span near the point of contraflexure and were placed on the structure where they are required structurally. Drains were located to prevent drainage across the joint if feasible. In some areas, they were provided with a drainage trough to collect the water passing through.

Sliding steel plate joints are semi-open joints since water and light debris can pass through. A sealant placed in the joint prevents some water from passing through. It also prevents the accumulation of debris which can keep the joint from moving as it was designed. To date, considerable maintenance has occurred with sealants and neoprene troughs have been added to collect the water at some sites.

Currently finger and sliding plate details are maintained for joint maintenance and retrofitting but are not used for new structures. Watertight expansion devices such as strip seals and modular types are recommended for new structures. Although these expansion joints are not completely watertight; they have been effective in reducing damage to adjacent girders, diaphragms, bearings and substructure units.

The neoprene compression seal is a closed joint which is watertight if it is properly installed and an adequate adhesive is employed. Compression seals are only used for fixed joints. Strip-seals are watertight joints which are used in place of sliding plate joints or finger joints in an attempt to keep water and debris on the bridge deck surface.

Refer to Figure 12.7-1 for placement of expansion devices. Criteria for placement of expansion devices is described in the following sections.

## 28.6 Joint Performance

Currently the approved modular expansion devices with continuous neoprene seals and individual bearing support bars have performed well. From the maintenance standpoint, they are preferred over steel finger joints with troughs that require periodic cleaning. Galvanizing modular expansion joints is required due to the number of steel components subjected to chlorides and potential for corrosion. Strip seal joints require galvanizing too.

Joint cleaning and inspection/repair of the neoprene glands is imperative to insure long-term joint performance.

## 28.7 Joint Armoring

In the past, concrete edges at joint openings were protected or “armored” with steel angles. This previously included armoring compression seal joints to increase the joint durability by preventing spalling of the concrete. However, the practice of armoring concrete edges has largely been discontinued due to the installation challenges and long-term performance issues due to snowplow damage, leakage, and corrosion. Note: prior to 2008, Figure 28.2-1 included armoring angles.

The current practice is to limit the number of bridge joints (especially joints with steel hardware) and to use armor-less joints. As such, protection angle armor should not be used on paved (concrete or asphalt) approach roadways with or without paving notches. Protection angle amor may be considered on unpaved (gravel) approach roadways on a project-by-project basis.