DISCLAIMER

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1.1 Introduction

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

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

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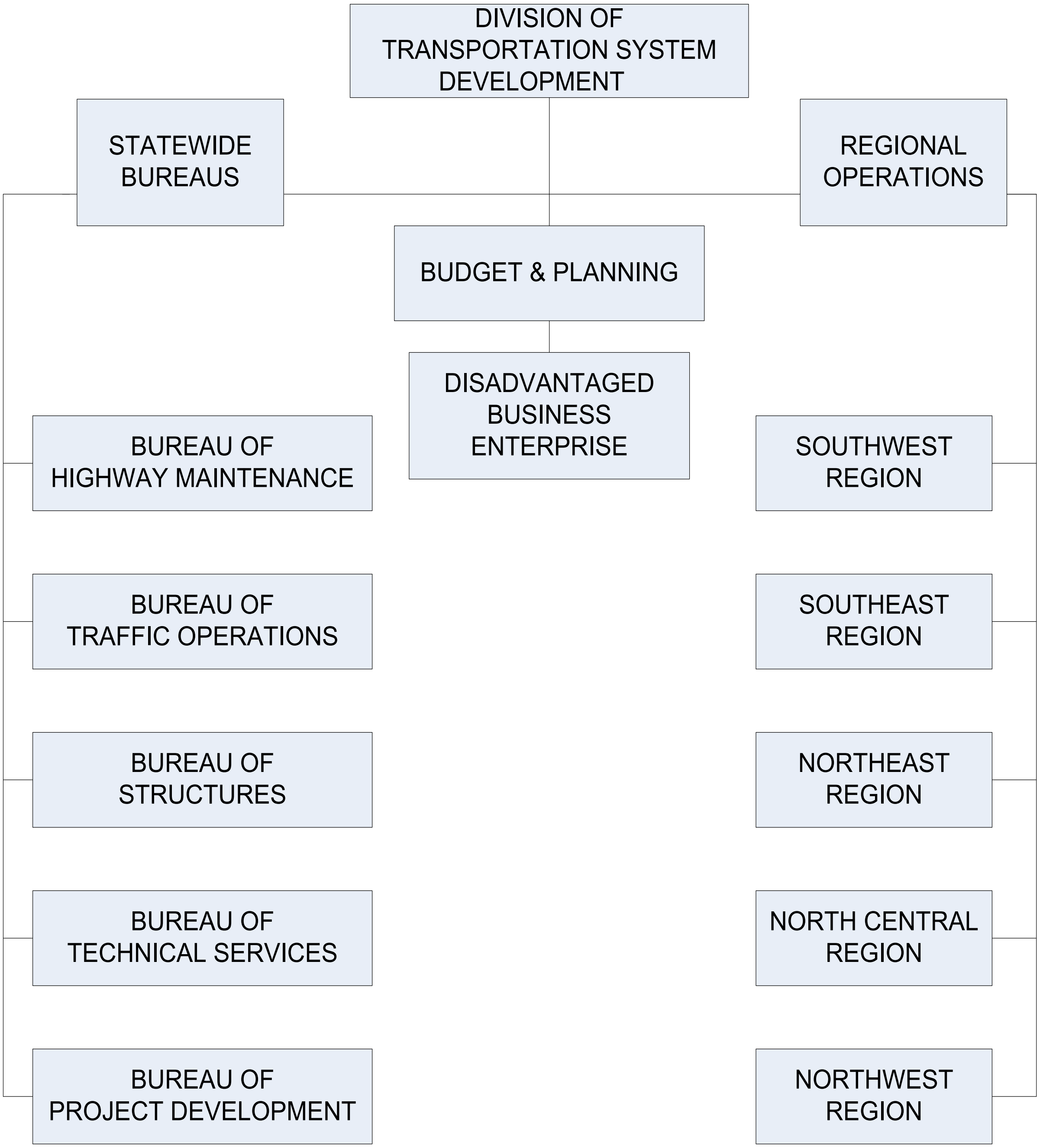


Figure 2.1-1
Division of Transportation System Development

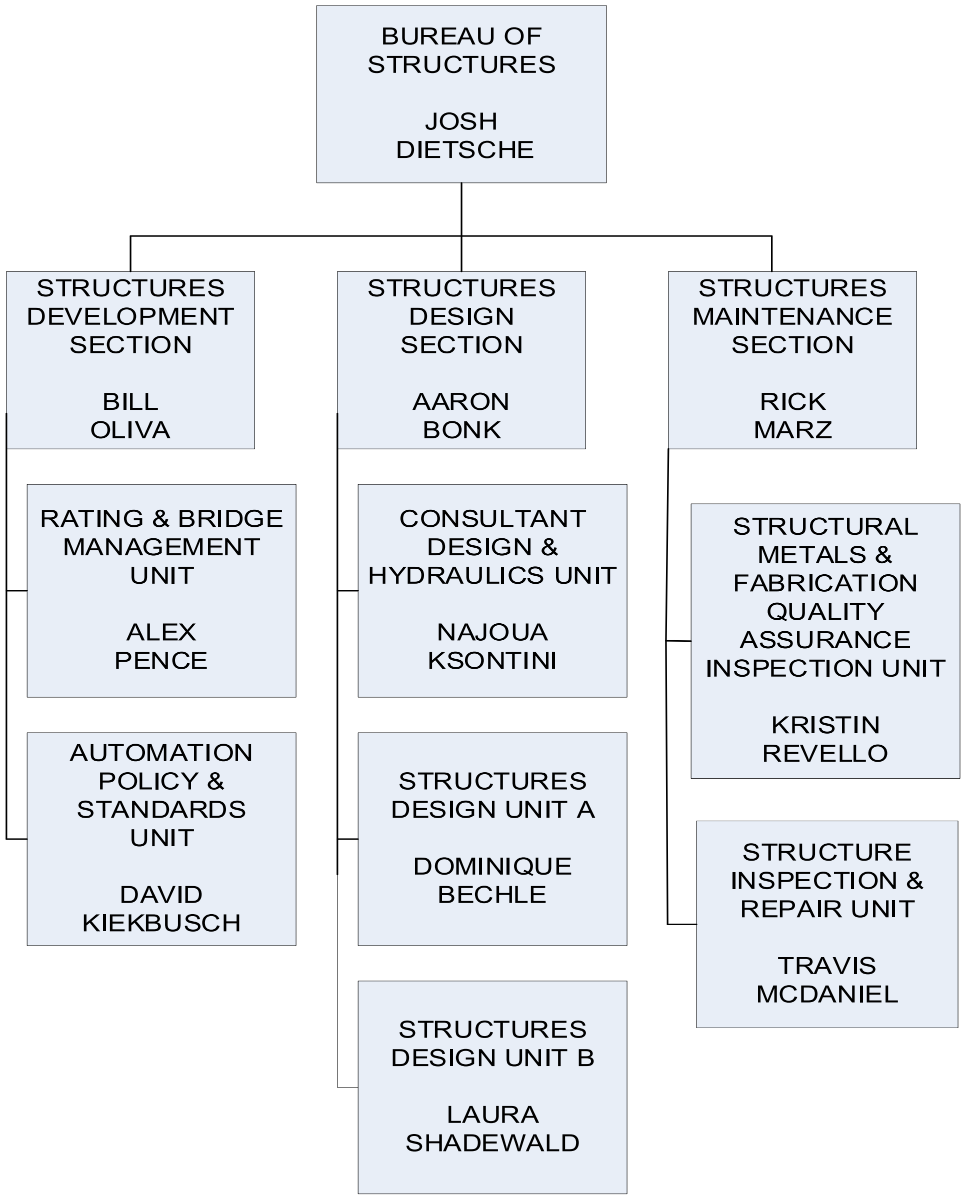
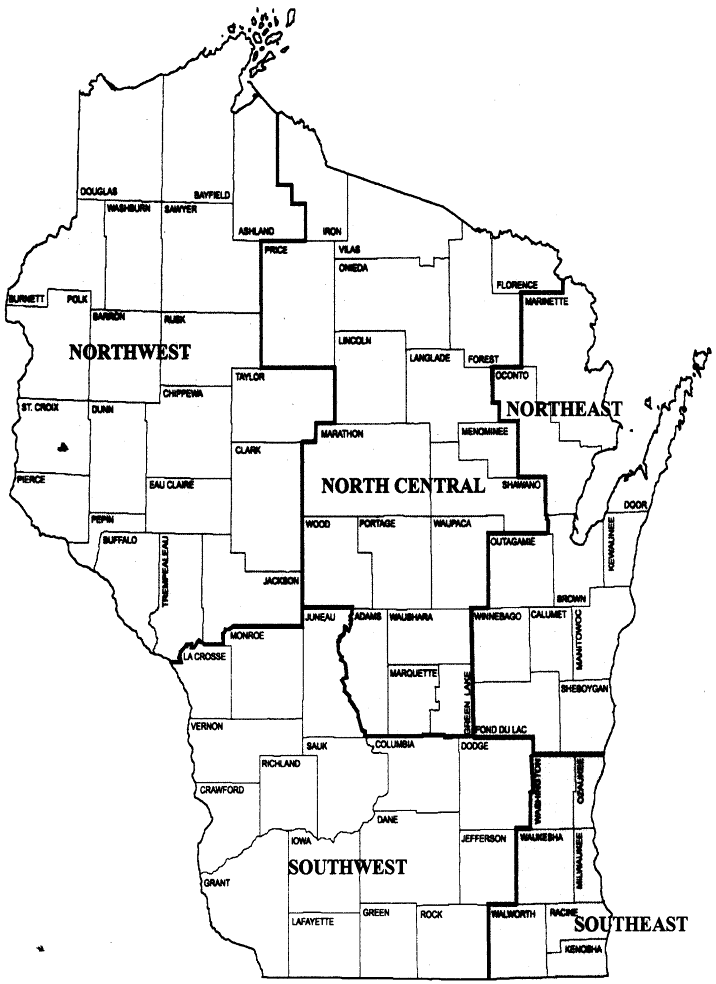


Figure 2.1-2
Bureau of Structures



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
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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. Contact the Regional Ancillary Program Manager for assigning structure numbers and structure unit numbers. For inspection purposes, structure unit numbers are beneficial and should be coordinated with the Region once determined needed. 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. 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. Unit numbers may be assigned to long bridges or complex interchanges where it is desirable to have only one bridge number for the site.

  Pedestrian only bridge structures are assigned a B-Structure if they are over 20 ft in structure length and 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.

- 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.

<u>WisDOT Policy Item:</u>

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.

- 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:

  - <u>Proprietary retaining walls</u> (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.

  - <u>Non-proprietary walls</u> (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 bridges or complex interchanges where it is desirable to have
- 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.

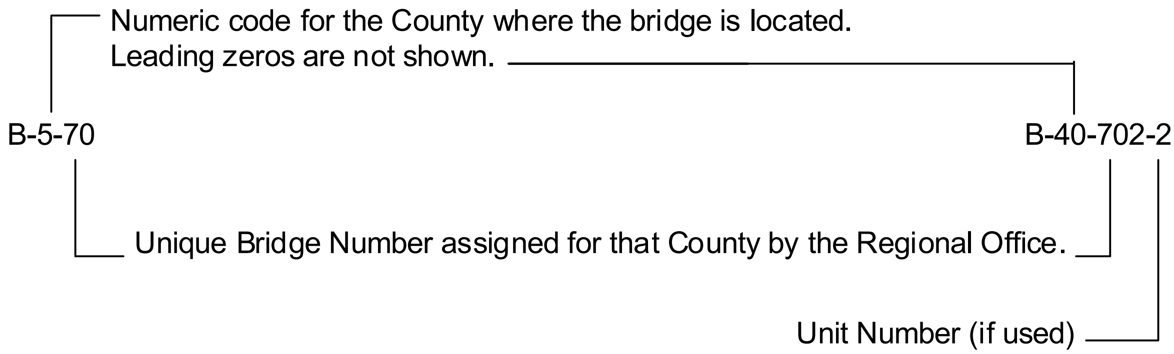

Figure 2.5-1
Bridge Number Detail

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.

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.

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.

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).
Holddown Device	A device used on bridge abutments to prevent girders from lifting off their bearings as a result of the passage of liveload 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.

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.

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.

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

	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 Holddown 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.

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

Table 2.10-1
Unique Structures

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

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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.

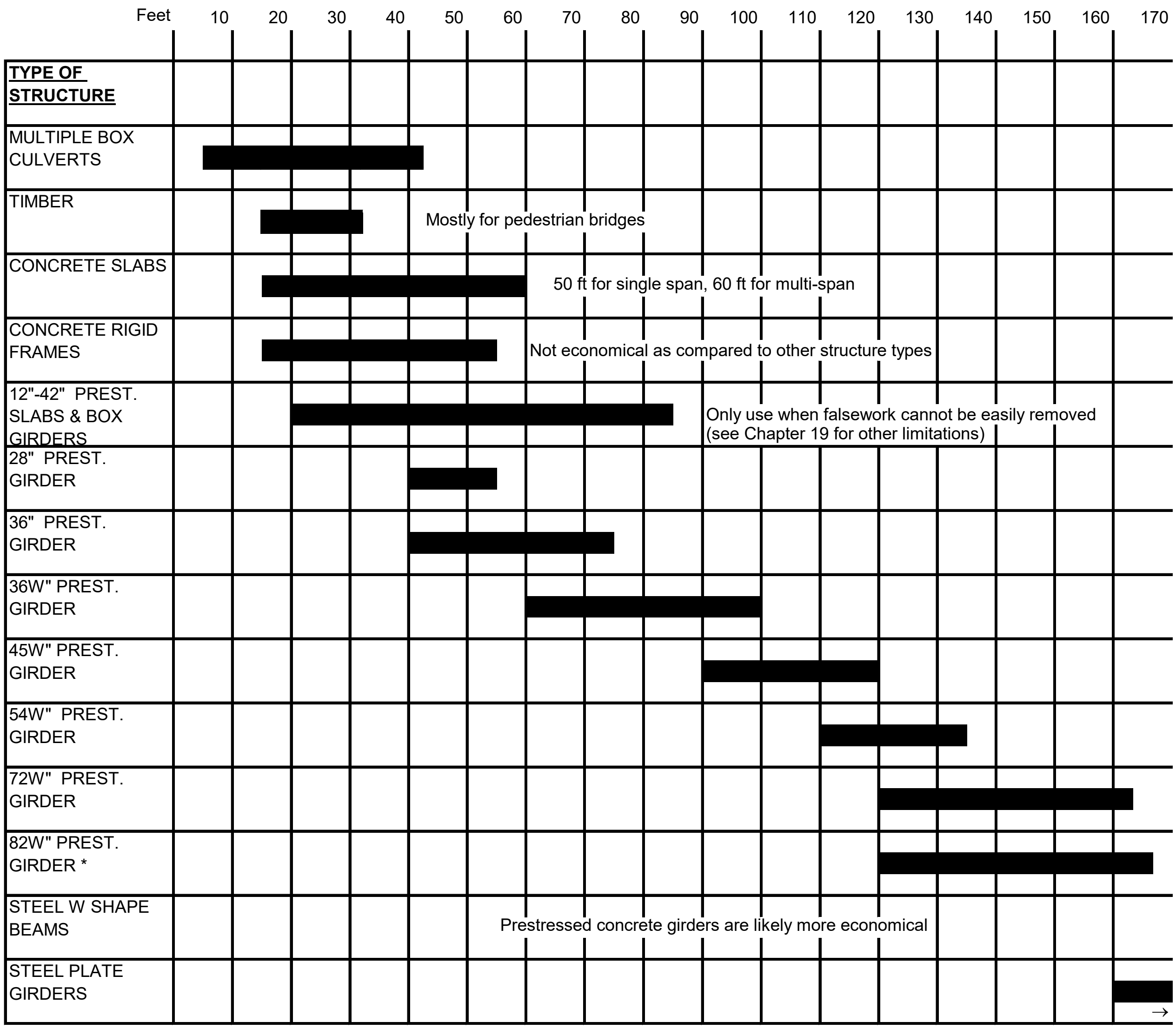
In these reports prestressed girder costs are grouped together because there is a small cost difference between girder sizes. Refer to unit costs. Concrete slab costs are also grouped together for this reason.

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

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

5.4 Bid Letting Cost Data

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

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

5.4.1 2015 Year End Structure Costs

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

Table 5.4-1
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	58	768,458	102,067,913	66.04	132.82
Reinf. Conc. Slabs (Flat)	2	8,566	922,866	46.36	107.74
Reinf. Conc. Slabs (Haunched)	1	6,484	868,845	41.26	133.99
Steel Plate Girders	4	100,589	20,248,653	137.13	201.30
Trapezoidal Steel Box Girders	4	305,812	79,580,033	189.24	260.23
Rigid Frames	2	7,657	2,730,308	--	356.58
Timber	1	16,800	1,982,669	--	118.02
Pre-Fab Pedestrian Bridges	1	1,851	449,475	--	242.83

Table 5.4-2
Grade Separation Structures

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

Table 5.4-3
Box Culverts

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

Table 5.4-4
Retaining Walls

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

Table 5.4-5
Sign Structures

5.4.2 2016 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	19	199,367	26,157,660	57.97	131.20
Reinf. Conc. Slabs (Flat)	36	72,066	10,985,072	63.40	152.43
Reinf. Conc. Slabs (Haunched)	5	22,144	2,469,770	50.63	111.53
Prestressed Box Girders	3	4,550	773,098	80.85	169.91

Table 5.4-6
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	25	343,165	40,412,805	60.62	117.76
Reinf. Conc. Slabs (Haunched)	5	33,268	4,609,286	59.21	138.55
Steel Plate Girders	3	127,080	18,691,714	90.78	147.09
Pedestrian Bridges	1	4,049	846,735	91.35	209.13

Table 5.4-7
Grade Separation Structures

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

Table 5.4-8
Box Culverts

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

Table 5.4-9
Retaining Walls

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

Table 5.4-10
Sign Structures

5.4.3 2017 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super Only Cost per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	24	238,956	33,970,344.86	60.05	142.16
Reinf. Conc. Slabs (Flat)	44	69,095	11,063,299.53	57.75	160.12
Reinf. Conc. Slabs (Haunched)	8	48,434	6,759,897.64	55.41	139.57
Prestressed Box Girders	2	2,530	691,474.35	117.93	273.31

Table 5.4-11
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	28	302,672	37,247,580.94	52.67	123.10
Reinf. Conc. Slabs (Flat)	25	58,076	9,561,823.06	42.14	164.64
Reinf. Conc. Slabs (Haunched)	6	49,160	9,444,012.75	43.73	192.11
Steel Plate Girders	0	--	--	--	--
Pedestrian Bridges	2	12,864	2,141,133.01	53.53	166.44

Table 5.4-12
Grade Separation Structures

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

Table 5.4-13
Box Culverts

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

Table 5.4-14
Retaining Walls

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

Table 5.4-15
Sign Structures

5.4.4 2018 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	45	276,820	45,260,979	66.45	163.50
Reinf. Conc. Slabs (Flat)	49	72,180	12,259,362	68.04	169.85
Reinf. Conc. Slabs (Haunched)	8	34,732	6,437,911	70.04	185.36
Prestressed Box Girder	1	1,864	419,175	113.39	224.88

Table 5.4-16
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	52	727,872	124,665,613	59.90	171.30
Reinf. Conc. Slabs (Haunched)	6	56,580	10,858.579	57.14	191.92
Steel Plate Girders	0	--	--	--	--
Trapezoidal Steel Box Girders	0	--	--	--	--

Table 5.4-17
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	13	1,948
Twin Cell	6	2,941
Three Cell	1	6,354

Table 5.4-18
Box Culverts

Bridge Type	Cost
Twin Pipe Culvert	2,292 Lin. Ft.

Table 5.4-19
Miscellaneous Bridges

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
CIP Cantilever	0	--	--	--
CIP Facing (MSE)	0	--	--	--
MSE Block Walls	3	4,693	567,547	120.93
MSE Panel Walls	49	378,371	44,841,726	118.51
Modular Walls	3	2,402	204,002	84.93
Precast Panel and Wire Faced	1	5,945	948,347	159.53
Soldier Pile Walls	4	8,531	1,570,107	184.05

Steel Sheet Pile Walls	2	16,620	1,639,380	98.64

Table 5.4-20
Retaining Walls

Sign Structure Type		No. of Structures	Total Lineal Ft. of Arm	Total Costs	Cost per Lin. Ft.
Butterfly (1-Sign)	Conc. Col.	6	118	273,756	2,319.97
	1-Steel Col.	0	--	--	--
Butterfly (2-Signs)	Conc. Col.	5	88	277,787	3,156.67
	1-Steel Col.	4	73	326,652	4,474.68
Cantilever	Conc. Col	8	234	588,676	2,515.71
	1-Steel Col	32	850.83	1,380,710	1,622.78
Cantilever Full Span	Conc. Col.	16	1267	2,909,973	2,296.74
	1-Steel Col.	2	184.2	279,115	1,515.28
	2-Steel Col.	17	1469	2,236,464	1,522.44
Full Span	1-Steel Col.	10	675.5	513,623	760.36
	2-Steel Col.	0	--	--	--

Table 5.4-21
Sign Structures

5.4.5 2019 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	25	128,141	21,357,588	66.55	166.67
Reinf. Conc. Slabs (Flat)	44	69,664	12,974,370	70.13	186.24
Reinf. Conc. Slabs (Haunched)	10	43,057	7,035,245	100.04	163.39
Prestressed Box Girder	1	1,253	292,643	101.17	233.55

Table 5.4-16
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	19	170,986	30,850,532	75.00	180.40
Reinf. Conc. Slabs (Haunched)	3	18,772	3,335,053	60.01	177.76
Steel Beams	1	7,964	1,897,388	95.77	238.25
Steel Plate Girders	3	130,986	30,430,624	144.97	232.32

Table 5.4-17
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	8	2,611
Twin Cell	5	3,559
Three Cell	1	3,444

Table 5.4-18
Box Culverts

Bridge Type	Cost
(none)	--

Table 5.4-19
Miscellaneous Bridges

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
CIP Cantilever	0	--	--	--
CIP Facing (MSE)	0	--	--	--
MSE Block Walls	7	17,195	2,490,957	144.87
MSE Panel Walls	27	85,496	10,517,536	123.02
Modular Walls	0	--	--	-
Precast Panel and Wire Faced	0	--	--	--
Soldier Pile Walls	3	6,290	1,378,911	219.22
Steel Sheet Pile Walls	1	1,940	92,512	47.69

Table 5.4-20
Retaining Walls

Sign Structure Type		No. of Structures	Total Lineal Ft. of Arm	Total Costs	Cost per Lin. Ft.
Butterfly (1-Sign)	Conc. Col.	0	--	--	--
	1-Steel Col.	0	--	--	--
Butterfly (2-Signs)	Conc. Col.	0	--	--	--
	1-Steel Col.	0	--	--	--
Cantilever	Conc. Col	0	--	--	--
	1-Steel Col	2	56	42,520	1,518
Cantilever Full Span	Conc. Col.	--	--	--	--
	1-Steel Col.	--	--	--	--
	2-Steel Col.	10	735.5	126,495	1,719.86
Full Span	1-Steel Col.	3	187	45,069	723.04
	2-Steel Col.	0	--	--	--

Table 5.4-21
Sign Structures

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10.1 General

The purpose of the Geotechnical Investigation is to provide subsurface information for the plans and to develop recommendations for the construction of the structure at reasonable costs versus short and long term performance. The level of Geotechnical Investigation is a function of the type of the structure and the associated performance. For example, a box culvert under a low ADT roadway compared to a multi-span bridge on a major interstate would require a different level of Geotechnical Investigation. The challenge for the geotechnical engineer is to gather subsurface information that will allow for a reasonable assessment of the soil and rock properties compared to the cost of the investigation.

The geotechnical engineer and the structure engineer need to work collectively when evaluating the loads on the structures and the resistance of the soil and rock. The development of the geotechnical investigation and evaluation of the subsurface information requires a degree of engineering judgment. A guide for performing the Geotechnical Investigation is provided in WisDOT Geotechnical Bulletin No. 1, **LRFD [10.4]** and Geotechnical Engineering Circular #5 – Evaluation of Soil and Rock Properties (Sabatini, 2002).

The following structures will require a Geotechnical Investigation:

- Bridges
- Box Culverts
- Retaining Walls
- Non-Standard Sign Structures Foundations
- High Mast Lighting Foundations
- Noise Wall Foundations

10.2 Subsurface Exploration

The Geotechnical Engineering Unit (or geotechnical consultant) prepares the Site Investigation Report (SIR) and the Subsurface Exploration (SE) sheet. The SIR describes the subsurface investigation, laboratory testing, analyses, computations and recommendations for the structure. All data relative to the underground conditions which may affect the design of the proposed structure's foundation are reported. Further information describing this required investigation can be found in the Department's "Geotechnical Bulletin #1" document. The Subsurface Exploration sheet is a CADDS drawing that illustrates the soil boring locations and is a graphical representation of the driller's findings. This sheet is included in the structure plans. If the Department is not completing the geotechnical work on the project, the SIR and SE sheet(s) are the responsibility of the consultant.

The subsurface investigation is composed of two areas of investigation: the Surface Survey and the detailed Site Investigation.

Surface Surveys include studies of the site geology and air-photo review, and they can include geophysical methods of exploration. This work should include a review of any existing structure foundations and any existing geotechnical information. Surface Surveys provide valuable data indicating approximate soil conditions during the reconnaissance phase.

Based on the results of the Surface Survey information, the plans for a Detailed Site Investigation are made. The subsurface investigation needs to provide the following information:

- Depth, extent and thickness of each soil or rock stratum
- Soil texture, color, mottling and moisture content
- Rock type, color and condition
- In-situ field tests to determine soil and rock parameters
- Laboratory samples for determining soil or rock parameters
- Water levels, water loss during drilling, utilities and any other relevant information

The number and spacing of borings is controlled by the characteristics and sequence of subsurface strata and by the size and type of the proposed structure. Depending upon the timing of the Geotechnical Investigation the required information may not be available and the geotechnical engineer may have to develop a subsurface investigation plan based on the initial design. The Department understands that additional investigation may be required once the preliminary design is completed. The challenge for the Department and the consultant is to develop a geotechnical investigation budget without knowing the subsurface conditions that will be encountered. Existing subsurface information from previous work can help this situation, but the plans should be flexible to allow for some unforeseen subsurface conditions.

One particular subsurface condition is the presence of shallow rock. In some cases, borings should be made at a frequency of one per substructure unit to adequately define the subsurface conditions. However, with shallow rock two or more borings may be necessary to define the rock line below the foundation. Alternatively, where it is apparent the soil is uniform, fewer borings are needed. For example, a four span bridge with short (less than30 foot) spans at each end of a bridge may only require three borings versus the five borings (one per substructure).

Borings are typically advanced to a depth where the added stress due to the applied load is 10 percent of the existing stress due to overburden or extended beyond the expected pile penetration depths. Where rock is encountered, borings are advanced by diamond bit coring according to ASTM D2113 to determine rock quality according to ASTM D6032.

LRFD [Table 10.4.2-1] Minimum Number of Exploration Points and Depth of Exploration (modified after Sabatini et al., 2002) provides guidelines for an investigation of bridges (shallow foundations and deep foundations) and retaining walls. The following presents the typical subsurface investigation guidelines for the other structures:

- Box Culverts: A minimum of two soil borings (generally located near the proposed culvert aprons) are recommended for box culvert lengths up to 150 feet, with one additional boring for each additional 100 feet of culvert length. These additional borings along the culvert length, should be spaced approximately equally between the apron borings. The number of additional borings can also be adjusted based on the uniformity of subsurface conditions, and knowledge of site geology. All borings should have a minimum of 15 feet of continuous SPT samples below the base of the box culvert.

- Box Culvert Extensions: The recommended borings depend on the extension length at the culvert end, available information from the existing box culvert, and proposed loads (i.e. traffic lanes over the extension). In general, one boring is recommended at each extension. If an extension length exceeds 150 feet, a minimum of two borings may be warranted.

- Non-Standard Sign Structure Foundations: The recommended spacing would be one for each sign structure site. If the sign structure is a bridge with two foundations then one boring may still be adequate. The borings should have 20 feet of continuous SPT samples and a SPT sample at 25 feet and 30 feet below the ground surface at the sign structure site.

- High Mast Lighting Foundations: The recommended spacing would be one for each site. The borings should have 15 feet of continuous SPT samples and a SPT sample every 5 feet to a depth of 40 feet below the ground surface at the site.

- Noise Wall Foundations: The recommended spacing would be one for every 200 feet to 300 feet of wall. The borings should have 20 feet of continuous SPT samples below the ground surface.

The Department generally follows AASHTO laboratory testing procedures. Any or all of the following soil tests may be considered necessary or desirable at a given site:

Table of Contents

14.1 Introduction

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

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

WisDOT policy item:

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

14.1.1 Wall Development Process

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

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

Based on the SSR, a Geotechnical site investigation (see Chapter 10 – Geotechnical Investigation) may be required to determine foundation and retained soil properties. A hydraulic analysis is also conducted, if required, to asses scour potential. The Geotechnical investigation generally includes a subsurface and laboratory investigation. For the departmental-designed walls, the Bureau of Technical Services, Geotechnical Engineering Unit can recommend the scope of soil exploration needed and provide/recommend bearing resistance, overall stability, and settlement of walls based on the geotechnical exploration results. These Geotechnical recommendations are presented in a Site Investigation Report.

The SSR is sent to the wall designer (Structures Design Section or WisDOT's Consultant) for wall selection, design and contract plan preparation. Based on the wall selection criteria discussed in 14.3, either a proprietary or a non-proprietary wall system is selected.

Proprietary walls, as defined in 14.2, are pre-approved by the WisDOT's Bureau of Structures. Preapproval process for the proprietary walls is explained in 14.16. The structural design, internal and final external stability of proprietary wall systems are the responsibility of the supplier/contractor. The design and shop drawing computations of the proprietary wall systems are also reviewed by the Bureau of Structures in accordance with the plans and special provisions. The preliminary external stability, overall stability and settlement computations of these walls are performed by the Geotechnical Engineering Unit or the WisDOT's Consultant in the project design phase. Design and shop drawings must be accepted by the Bureau of Structures prior to start of the construction. Design of all temporary walls is the responsibility of the contractor.

Non-proprietary retaining walls are designed by WisDOT or its Consultant. The internal stability and the structural design of such walls are performed by the Structures Design Section or WisDOT's Consultant. The external and overall stability is performed by the Geotechnical Engineering Unit or Geotechnical Engineer of record.

The final contract plans of retaining walls include final plans, details, special provisions, contract requirements, and cost estimate for construction. The Subsurface Exploration sheet depicting the soil borings is part of the final contract plans.

The wall types and wall selection criteria to be used in wall selection are discussed in 14.2 and 14.3 of this chapter respectively. General design concepts of a retaining wall system are discussed in 14.4. Design criteria for specific wall systems are discussed in sections 14.5 thru 14.11. The plan preparation process is briefly described in Chapter 2 – General and Chapter 6 – Plan Preparation. The contract documents and contract requirements are discussed in 14.14 and 14.15 respectively.

For further information related to wall selection, design, approval process, pre-approval and review of proprietary wall systems please contact Structures Design Section of the Bureau of Structures at 608-266-8489. For questions pertaining to geotechnical analyses and geotechnical investigations please contact the Geotechnical Engineering Unit at 608-246-7940.

14.1.1.1 Wall Numbering System

Refer to 2.5 for assigning structure numbers.

14.2 Wall Types

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

Conventional Walls

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

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

Permanent or Temporary Walls

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

Fill Walls or Cut Walls

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

Bottom-up or Top-down Constructed Walls

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

Proprietary or Non-Proprietary

Some retaining walls have prefabricated modules or components that are proprietary in nature. Based on the use of proprietary components, walls can be divided into the categories of proprietary and non-proprietary wall systems as defined in 14.1.1.

A proprietary retaining wall system is considered as a patented or trademarked retaining wall system or a wall system comprised of elements/components that are protected by a trade name, brand name, or patent and are designed and supported by the manufacturer. MSE walls, modular block gravity walls, bin, and crib walls are considered proprietary walls because these walls have components which are either patented or have trademarks.

Proprietary walls require preapproval and appropriate special provisions. The preapproval requirements are discussed in 14.16 of this chapter. Proprietary walls also have special design requirements for the structural components, and are discussed in further detail within each specific wall design section. Most MSE, modular block, bin or crib walls require pre-approval and/or special provisions.

A non-proprietary retaining wall is fully designed and detailed by the designer or may be design-build. A non-proprietary retaining wall system may contain proprietary elements or components as well as non-proprietary elements and components. CIP cantilever walls, rock walls, soil nail walls and non-gravity walls fall under this category.

Wall classification is shown in Table 14.2-1 and is based on wall type, project function category, and method of construction.

14.2.1 Gravity Walls

Gravity walls are considered externally stabilized walls as these walls use self weight to resist lateral pressures due to earth and water. Gravity walls are generally subdivided into mass gravity, semi-gravity, modular gravity, mechanically stabilized reinforced earth (MSE), and in-situ reinforced earth wall (soil nailing) categories. A schematic diagram of the various types of gravity walls is included in Figure 14.2-1.

14.2.1.1 Mass Gravity Walls

A mass gravity wall is an externally stabilized, cast-in-place rigid gravity wall, generally trapezoidal in shape. The construction of these walls requires a large quantity of materials so these are rarely used except for low height walls less than 8.0 feet. These walls mainly rely on self-weight to resist external pressures and their construction is staged as bottom up construction, mostly in fill or cut/fill situations.

14.2.1.2 Semi-Gravity Walls

Semi-gravity walls resist external forces by the combined action of self-weight, weight of soil above footing and the flexural resistance of the wall components. A cast-in-place (CIP) concrete cantilever wall is an example and consists of a reinforced concrete stem and a base footing. These walls are non-proprietary.

Cantilever walls are best suited for use in areas exhibiting good bearing material. When bearing or settlement is a problem, these walls can be founded on piles or foundation improvement may be necessary. The use of piles significantly increases the cost of these walls. Walls exceeding 28 feet in height are provided with counter-forts or buttress slabs. Construction of these walls is staged as bottom-up construction and mostly constructed in fill situations. Cantilever walls are more suited where MSE walls are not feasible, although these walls are generally costlier than MSE walls.

14.2.1.3 Modular Gravity Walls

Modular walls are also known as externally stabilized gravity walls as these walls resist external forces by utilizing self-weight. Modular walls have prefabricated modules/components which are considered proprietary. The construction is bottom-up construction mostly used in fill situations.

14.2.1.3.1 Modular Block Gravity Walls

Modular block concrete facings are used without soil reinforcement to function as an externally stabilized gravity wall. The modular blocks are prefabricated dry cast or wet cast concrete blocks and the blocks are stacked vertically or slightly battered to resist external forces. The concrete blocks are either solid concrete or hollow core concrete blocks. The hollow core concrete blocks are filled with crushed aggregates or sand. Modular block gravity walls are limited to a maximum design height of 8 feet under optimum site geometry and soils conditions, but site conditions generally dictate the need for MSE walls when design heights are greater than 5.5 feet. Walls with a maximum height of less than 4 feet are deemed as “minor retaining walls” and do not require an R number. Refer to FDM 11-55-5.2 for more information. The modular blocks are proprietary and vary in sizes.

14.2.1.3.2 Prefabricated Bin, Crib and Gabion Walls

<u>Bin Walls</u>: Concrete and metal bin walls are built of adjoining open or closed faced bins and then filled with soil/rocks. Each metal bin is comprised of individual members bolted together. The concrete bin wall is comprised of prefabricated interlocking concrete modules. These wall systems are proprietary wall systems.

<u>Crib Walls</u>: Crib walls are constructed of interlocking prefabricated units of reinforced or unreinforced concrete or timber elements. Each crib is comprised of longitudinal and transverse members. Each unit is filled with free draining material. These wall systems are proprietary wall systems.

<u>Gabion Walls</u>**:** Gabion walls are constructed of steel wire baskets filled with selected rock fragments and tied together. Gabions walls are flexible, free draining and easy to construct. These wall systems are proprietary wall systems. Maximum heights are normally less than 21 feet. These walls are desirable where equipment access is limited. The wires used for constructing gabions baskets must be designed with adequate corrosion protection.

14.2.1.4 Rock Walls

Rock walls are also known as 'Rockery Walls'. These types of gravity walls are built by stacking locally available large stones or boulders into a trapezoid shape. These walls are highly flexible and height of these walls is generally limited to approximately 8.0 feet. A layer of gravel and geotextile is commonly used between the stones and the retained soil. These walls can be designed using the *FHWA Rockery Design and Construction Guideline*.

14.2.1.5 Mechanically Stabilized Earth (MSE) Walls:

Mechanically Stabilized Earth (MSE) walls include a selected soil mass reinforced with metallic or geosynthetic reinforcement. The soil reinforcement is connected to a facing element to prevent the reinforced soil from sloughing. Construction of these walls is staged as bottom-up construction. These can be constructed in cut and fill situations, but are better suited to fill sites. MSE walls are normally used for wall heights between 10 to 40 feet. A brief description of various types of MSE walls is given below:

<u>Precast Concrete Panel MSE Walls</u>: These types of walls employ a metallic strip or wire grid reinforcement connected to precast concrete panels to reinforce a selected soil mass. The concrete panels are usually 5'x5' or 5'x10' size panels. These walls are proprietary wall systems.

<u>Modular Block Facing MSE Wall</u>: Prefabricated modular concrete block walls consist of almost vertically stacked concrete modular blocks and the soil reinforcement is secured between the blocks at predetermined levels. Metallic strips or geogrids are generally used as soil reinforcement to reinforce the selected soil mass. Concrete blocks are either solid or hollow core blocks, and must meet freeze/thaw requirements. The hollow core blocks are filled with aggregates or sand. These types of walls are proprietary wall systems.

<u>Geotextile/Geogrids/Welded Wire Faced MSE Walls</u>: These types of MSE walls consist of compacted soil layers reinforced with continuous or semi-continuous geotextile, geogrid or welded wire around the overlying reinforcement. The wall facing is formed by wrapping each layer of reinforcement around the overlying layer of backfill and re-embedding the free end into the backfill. These types of walls are used for temporary or permanent applications. Permanent facings include shotcrete, gunite, galvanized welded wire mesh, cast-in-place concrete or prefabricated concrete panels.

14.2.1.6 Soil Nail Walls

Soil nail walls are internally stabilized cut walls that use in-situ reinforcement for resisting earth pressures. The large diameter rebars (generally #10 or greater) are typically used for the reinforcement. The construction of soil nail walls is staged top-down and soil nails are installed after each stage of excavation. Shotcrete can be applied as a facing. The facing of a soil nail wall is typically covered with vertical drainage strips located over the nail then covered with shotcrete. Soil nail walls are used for temporary or permanent construction. Specialty contractors are required when constructing these walls. Soil nail walls have been installed to heights of 60.0 feet or more but there have only been a limited number of soil nail walls constructed on WisDOT projects.

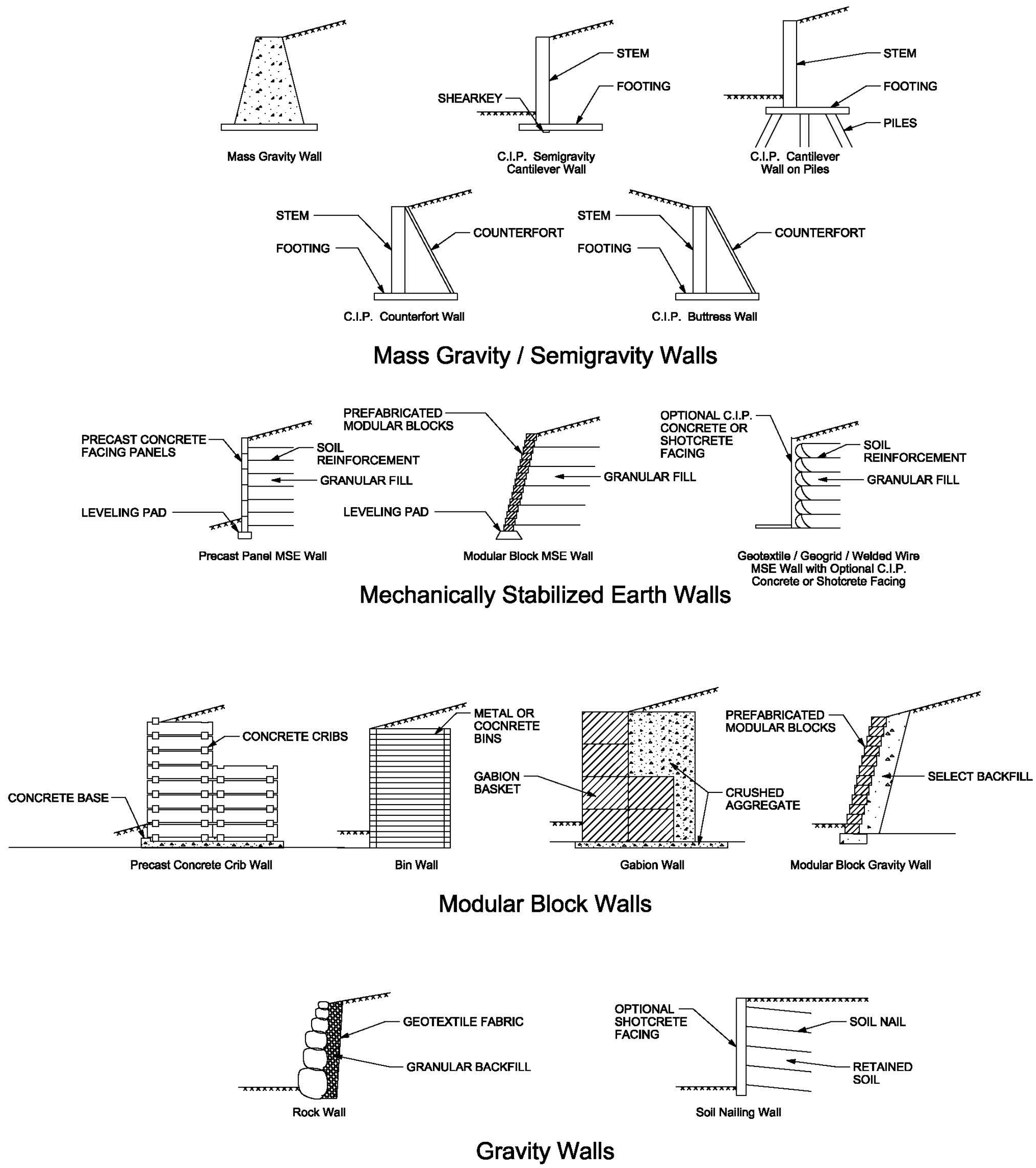


Figure 14.2-1
Gravity Walls

14.2.2 Non-Gravity Walls

Non-gravity walls are classified into cantilever and anchored wall categories. These walls are considered as externally stabilized walls and generally used in cut situations. The walls include sheet pile, soldier pile, tangent and secant pile type with or without anchors. Figure 14.2-2 shows common types of non-gravity walls.

14.2.2.1 Cantilever Walls

These types of walls derive lateral resistance through embedment of vertical elements into natural ground and the flexure resistance of the structural members. They are used where excavation support is needed in shallow cut situations.

Cantilever Sheet Pile Walls: Cantilever sheet pile walls consist of interlocking steel panels, driven into the ground to form a continuous sheet pile wall. The sheet piles resist the lateral earth pressure utilizing the passive resistance in front of the wall and the flexural resistance of the sheet pile. Most sheet pile walls are less than 15 feet in height.

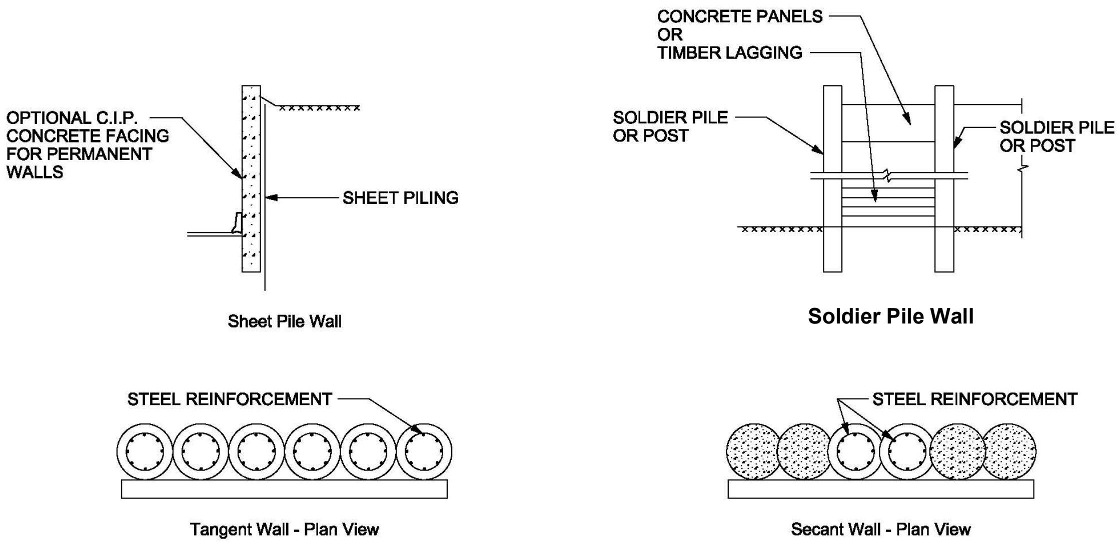
Soldier Pile Walls: A soldier pile wall derives lateral resistance and moment capacity through embedment of vertical members (soldier piles) into natural ground usually in cut situations. The vertical elements (usually H piles) may be drilled or driven steel or concrete members. The soil behind the wall is retained by lagging. The lagging may be steel, wood, or concrete. For permanent walls, wall facings are usually constructed of either cast-in-place concrete or precast concrete panels (prestressed, if needed) that extend between vertical elements. Solider pile walls that use precast panels and H piles are also known as post-and-panel walls. Soldier pile walls can also be constructed from the bottom-up. These walls should be considered when minimizing disturbance to the site is critical, such as environmental and/or construction procedures. Soldier pile walls are also suitable for sites where rock is encountered near the surface, since holes for the piles can be drilled/prebored into the rock.

Tangent and Secant Pile Walls: A tangent pile wall consists of a single row of drilled shafts (bored piles) installed in the ground. Each pile touches the adjacent pile tangentially. The concrete piles are reinforced using a single steel beam or a steel reinforcement cage. A secant wall, similar to a tangent pile wall, consists of overlapping adjacent piles. All piles generally contain reinforcement, although alternating reinforced piles may be necessary. Secant and tangent wall systems are used to hold earth and water where water tightness is important, and lowering of the water table is not desirable. To improve wall water tightness, additional details can used to minimize water seepage.

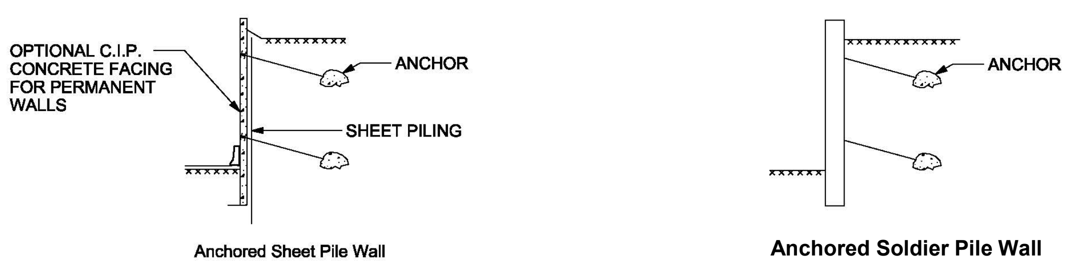
14.2.2.2 Anchored Walls

Anchored walls are externally stabilized non-gravity cut walls. Anchored walls are essentially the same as cantilever walls except that these walls utilize anchors (tiebacks) to extend the wall heights beyond the design limit of the cantilever walls. These walls require less toe embedment than cantilever walls.

These walls derive lateral resistance by embedment of vertical wall elements into firm ground and by anchorages. Most commonly used anchored walls are anchored sheet pile walls and soldier pile walls. Tangent and secant walls can also be anchored with tie backs and used as anchored walls. The anchors can be attached to the walls by tie rods, bars or wire tendons. The anchoring device is generally a deadman, screw-type, or grouted tieback anchor. Anchored walls can be built to significant heights using multiple rows of anchors.



Cantilever Walls



Anchored Walls

Figure 14.2-2
Non-Gravity Walls

14.2.3 Tiered and Hybrid Wall Systems

A tiered wall system is a series of two or more walls, with each wall set back from the underlying walls. The upper wall exerts an additional surcharge on the lower lying wall and requires special design attention. The design of these walls has not been discussed in this chapter. Hybrids wall systems combine wall components from two or more different wall systems and provide an alternative to a single type of wall used in cut or fill locations. These types of walls require special design attention as components of these walls require different magnitudes of deformation to develop loading resistance. The design of such walls will be on a case-by-case basis, and is not discussed in this chapter.

Some examples of tiered and hybrid walls systems are shown in Figure 14.2-3.

14.2.4 Temporary Shoring

Temporary shoring is used to 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. Shoring should not be required nor paid for when used primarily for the convenience of the contractor. Temporary shoring is designed by the contractor and may consist of a wall system, or some other type of support. MSE walls with flexible facings and sheet pile walls are commonly used for temporary shoring.

14.2.5 Wall Classification Chart

A wall classification chart has been developed and shown as Table 14.2-1.

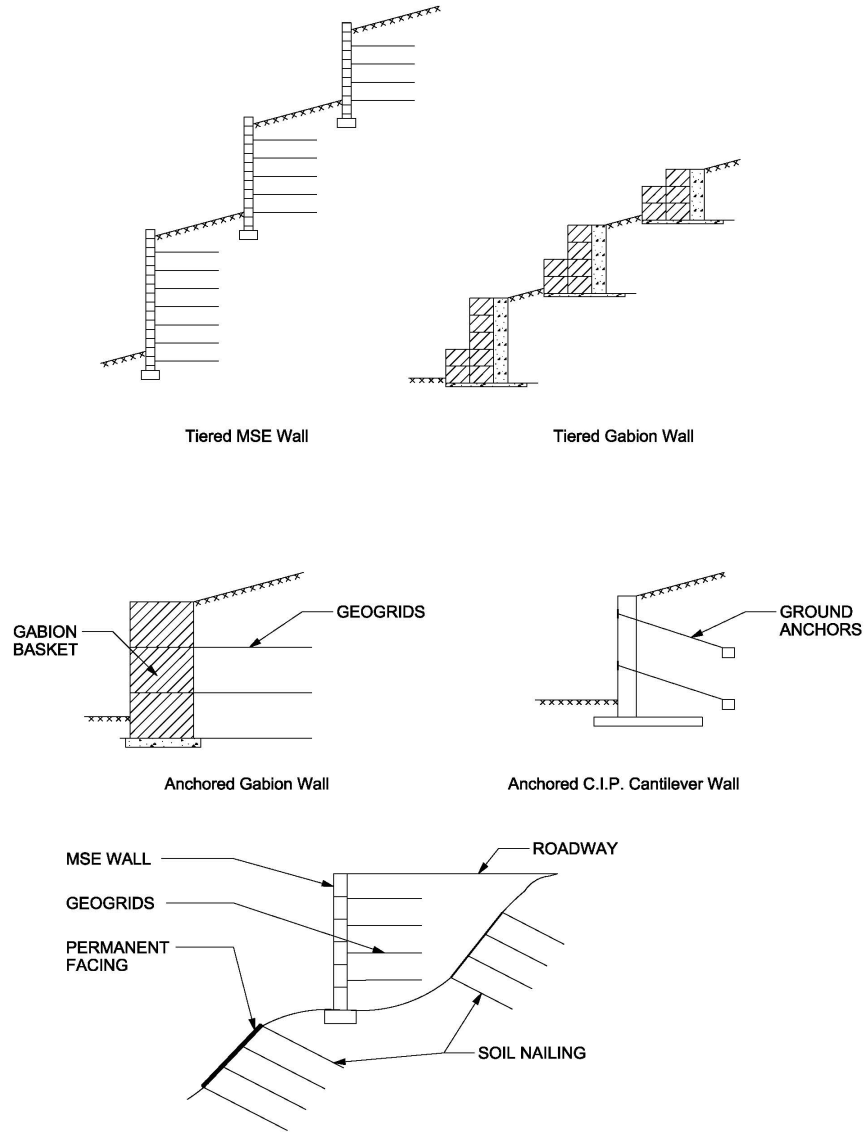


Figure 14.2-3
Tiered & Hybrid Wall Systems

Wall Category	Wall Sub-Category	Wall Type	Typical Construction Concept	Proprietary
Gravity	Mass Gravity	CIP Concrete Gravity	Bottom Up (Fill)	No
	Semi-Gravity	CIP Concrete Cantilever	Bottom Up (Fill)	No
	Reinforced Earth	MSE Walls: • Precast Panels • Modular Blocks • Geogrid/ Geo-textile/Wire- Faced	Bottom Up (Fill)	Yes
	Modular Gravity	Modular Blocks, Gabion, Bin, Crib	Bottom Up (Fill)	Yes
	In-situ Reinforced	Soil Nailing	Top Down (Cut)	No
Non-Gravity	Cantilever	Sheet Pile, Soldier Pile, Tangent/Secant	Top Down (Cut) /Bottom Up (Fill)	No
	Anchored	Anchored Sheet Pile, Soldier Pile, Tangent/Secant	Top Down (Cut)	No

Table 14.2-1
Wall Classification

14.3 Wall Selection Criteria

14.3.1 General

The objective of selecting a wall system is to determine an appropriate wall system that is practical to construct, structurally sound, economic, aesthetically pleasing, environmentally consistent with the surroundings, and has minimal maintenance problems.

With the development of many new wall systems, designers have the choice of selecting many feasible wall systems that can be constructed on a given highway project. Designers are encouraged to evaluate several feasible wall systems for a particular project where wall systems can be economically constructed. After consideration of various wall types, a single type should be selected for final analyses and design. Wall designers must consider the general design concepts described in section 14.4 and specific wall design requirements described in 14.5 thru 14.11 of this chapter, and key wall selection factors discussed in this section.

In general, selection of a wall system should include, but not limited to the key factors described in this section for consideration when generating a list of acceptable retaining wall systems for a given site.

14.3.1.1 Project Category

The designer must determine if the wall system is permanent or temporary.

14.3.1.2 Cut vs. Fill Application

Due to construction techniques and base width requirements for stability, some wall types are better suited for cut sections where as others are suited for fill or fill/cut situations. The key considerations are the amount of excavation or shoring, overall wall height, proximity of wall to other structures, and right-of-way width available. The site geometry should be evaluated to define site constraints. These constraints will generally dictate if fill, fill/cut or cut walls are required.

Cut Walls

Cut walls are generally constructed from the top down and used for both temporary and permanent applications. Cantilever sheet pile walls are suitable for shallower cuts. If a deeper cut is required to be retained, a key question is to determine the availability of right-of-way (ROW). Subsurface conditions such as shallow bedrock also enter into considerations of cut walls. Anchored walls, soil nail walls, and anchored soldier pile walls may be suitable for deeper cuts although these walls require either a larger permanent easement or permanent ROW.

Fill walls

Walls constructed in fill locations are typically used for permanent construction and may require large ROW to meet the base width requirements. The necessary fill material may be required to be granular in nature. These walls use bottom up construction and have typical cost effective ranges. Surface conditions must also be considered. For instance, if soft compressible soils are present, walls that can tolerate larger settlements and movements must be considered. MSE walls are generally more economical for fill locations than CIP cantilever walls.

Cut/fill Walls

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

14.3.1.3 Site Characteristics

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

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

14.3.1.4 Miscellaneous Design Considerations

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

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

14.3.1.5 Right of Way Considerations

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

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

14.3.1.6 Utilities and Other Conflicts

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

14.3.1.7 Aesthetics

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

14.3.1.8 Constructability Considerations

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

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

14.3.1.9 Environmental Considerations

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

14.3.1.10 Cost

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

14.3.1.11 Mandates by Other Agencies

In certain project locations, other agency mandates may limit the types of wall systems considered.

14.3.1.12 Requests made by the Public

A Public Interest Finding could dictate the wall system to be used on a specific project.

14.3.1.13 Railing

For safety reasons most walls will require a protective railing. The railing will usually be located behind the wall. The roadway designer will generally determine whether a pedestrian or non-pedestrian railing is required and what aesthetic considerations are needed.

14.3.1.14 Traffic barrier

A traffic barrier should be installed if vehicles, bicycles, or pedestrians are likely to be present on top of the wall. The roadway designer generally determines the need for a traffic barrier.

14.3.2 Wall Selection Guide Charts

Table 14.3-1 and Table 14.3-2 summarize the characteristics for the various wall types that are normally considered during the wall selection process. The tables also present some of the advantages, disadvantages, cost effective height range and other key selection factors. A wall designer can use these tables and the general wall selection criteria discussed in 14.3.1 as a guide. Designers are encouraged to contact the Structures Design Section if they have any questions relating to wall selection for their project.

Wall Type	Temp.	Perm.	Cost Effective Height (ft)	Req'd. ROW	Advantages	Disadvantages
CIP Concrete Gravity		√	3 - 10	0.5H - 0.7H	• Durable • Meets aesthetic requirement • Requires small quantity of select backfill	• High cost • May need deep foundation • Longer const. time
CIP Concrete Cantilever		√	6 - 28	0.4H - 0.7H	• Durable meets aesthetic requirement • Requires small quantity of select backfill	• High cost • May need deep foundation • Longer const. time & deeper embedment
Reinforced CIP Counterfort		√	26 - 40	0.4H - 0.7H	• Durable • Meets aesthetic requirement • Requires small back fill quantity	• High cost • May need deep foundation • Longer const. time & deeper embedment
Modular Block Gravity		√	3 - 8	0.4H - 0.7H	• Does not require skilled labor or specialized equipment	• Height limitations
Metal Bin		√	6 - 20	0.4H - 0.7H	• Does not require skilled labor or special equipment	• Difficult to make height adjustment in the field
Concrete Crib		√	6 - 20	0.4H - 0.7H	• Does not require skilled labor or specialized equipment	• Difficult to make height adjustment in the field
Gabion		√	6 - 20	0.4H - 0.7H	• Does not require skilled labor or specialized equipment	• Need large stone quantities • Significant labor
MSE Wall (precast concrete panel with steel reinforcement)		√	10 – 30*	0.7H - 1.0H	• Does not require skilled labor or specialized equipment	• Requires use of select backfill
MSE Wall (modular block and geo-synthetic reinforcement)		√	6 – 22*	0.7H - 1.0H	• Does not require skilled labor or specialized equipment	• Requires use of select backfill
MSE Wall (geotextile/geogrid/ welded wire facing)	√	√	6 – 35*	0.7H - 1.0H	• Does not require skilled labor or specialized equipment	• Requires use of select backfill

*WisDOT maximum wall height

Table 14.3-1
Wall Selection Chart for Gravity Walls

Wall Type	Temp.	Perm.	Cost Effective Height (ft)	Req'd. ROW	Water Tightness	Advantages	Disadvantages
Sheet Pile	√	√	6 - 15	Minimal	Fair	• Rapid construction • Readily available	• Deep foundation may be needed • Longer construction time
Soldier Pile	√	√	6 - 28	0.2H - 0.5H	Poor	• Easy construction • Readily available	• High cost • Deep foundation may be needed • Longer construction time
Tangent Pile		√	20 - 60	0.4H - 0.7H	Fair/Poor	• Adaptable to irregular layout • Can control wall stiffness	• High cost • Deep foundation may be needed • Longer construction
Secant Pile		√	14 - 60	0.4H - 0.7H	Fair	• Adaptable to irregular layout • Can control wall stiffness	• Difficult to make height adjustment in the field • High cost
Anchored	√	√	15 - 35	0.4H - 0.7H	Fair/Poor	• Rapid construction	• Difficult to make height adjustment in the field
Soil Nail	√	√	6 - 20	0.4H - 0.7H	Fair	• Option for top-down	• Cannot be used in all soil types • Cannot be used below water table • Significant labor

Table 14.3-2
Wall Selection Chart for Non-Gravity Walls

14.4 General Design Concepts

This section covers the general design standards and criteria to be used for the design of temporary and permanent gravity and non-gravity walls including proprietary and non-proprietary wall systems.

The design criteria for tiered walls that retain other walls or hybrid walls systems requiring special design are not covered specifically in this section.

14.4.1 General Design Steps

The design of wall systems should follow a systematic process applicable for all wall systems and summarized below:

1. Basic Project Requirement: This includes determination of wall alignment, wall geometry, wall function, aesthetic, and project constraints (e.g. right of way, easement during construction, environment, utilities, etc.) as part of the wall development process described in 14.1.

2. Wall Selection: Select wall type based on step 1 and the wall section criteria discussed in 14.3.

3. Geotechnical Investigation: Subsurface investigation and analyses should be performed in accordance with 14.4.4 and Chapter 10 - Geotechnical Investigation to develop foundation and fill material design strength parameters and foundation bearing capacity. Note: this work generally requires preliminary checks performed in step 7, based on steps 4 thru 6.

4. Wall Loading: Determine all applicable loads likely to act on the wall as discussed in 14.4.5.3.

5. Initial Wall Sizing: This step requires initial sizing of various wall components and establishing wall batter which is wall specific and described under each specific wall designs discussed in 14.5 thru 14.13.

6. Wall Design Requirements: Design wall systems using design standards and service life criteria and the *AASHTO Load and Resistance Factor Design* (*AASHTO LRFD*) requirements discussed in 14.4.1 and 14.4.2.

7. Perform external stability, overall stability, and wall movement checks discussed in 14.4.7. These checks will be wall specific and generally performed by the Geotechnical Engineer of record. The stability checks should be performed using the performance limits, load combinations, and the load/resistance factors per *AASHTO LRFD* requirements described in 14.4.5.5 and 14.4.5.6 respectively.

8. Perform internal stability and structural design of the individual wall components and miscellaneous components. These computations are performed by the Designer for non-proprietary walls. For proprietary walls, internal stability is the responsibility of the contractor/supplier after letting.

9. Repeat design steps 4 thru 8 if the required checks are not met.

14.4.2 Design Standards

Retaining wall systems shall be designed in conformance with the current *AASHTO Load and Resistance Factor Design Specifications* (AASHTO LRFD) and in accordance with the WisDOT Bridge Manual. Walls shall be designed to address all limit states.

Wall systems including rock walls and soil nail systems which are not specifically covered by the *AASHTO LRFD* specifications shall be designed using the hierarchy of guidelines presented in this chapter, Allowable Stress Design (ASD) or *AASHTO Load Factor Design* (LFD) methods or the design procedures developed based on standard engineering and/or industry practices. The guidelines presented in this chapter will prevail where interpretation differs. WisDOT's decision shall be final in those cases. The new specifications for the wall designs were implemented October 1st, 2010.

14.4.3 Design Life

All permanent retaining walls and components shall be designed for a minimum service life of 75 years. All temporary walls shall be designed for a period of 36 months or for the project specific duration, whichever is greater. The design of temporary wall systems is the responsibility of the contractor. The temporary walls shall meet all the safety requirements as that of a permanent wall except for corrosion and aesthetics.

14.4.4 Subsurface Exploration

Geotechnical exploration may be needed to explore the soil/rock properties for foundation, retained fill, and backfill soils for all retaining walls regardless of wall height. It is the designer's responsibility to ensure that pertinent soils information, loading conditions, foundation considerations, consolidation potential, settlement and external stability is provided for the wall design.

Before planning a subsurface investigation, it is recommended that any other available subsurface information such as geological or other maps or data available from previous subsurface investigations be studied. Subsurface investigation and analyses should be performed where necessary, in accordance with Chapter 10 - Geotechnical Investigation.

The investigations and analyses may be required to determine or establish the following:

- Nominal bearing pressure, consolidation properties, unit weight and shear strength (drained or undrained strength for fine grained soils) for foundation soils/rocks.

- Shear strength, and unit weight of selected backfill.

- Shear strength and unit weight of random fill or in-situ soil behind selected backfill or wall

- Location of water table

14.4.5 Load and Resistance Factor Design Requirements

14.4.5.1 General

In the LRFD process, wall stability is checked as part of the design process for anticipated failure modes for various types of walls at specified limit states, and the wall components are sized accordingly.

To evaluate the limit states, all applicable design loads are computed as nominal or un-factored loads, than factored using a load factor and grouped to consider the force effect of all loads and load combinations in accordance with **LRFD [3.4.1]**. The factored loads are compared with the factored resistance as part of the stability check in accordance with **LRFD [11.5]** such that the factored resistance is not less than factored loads as presented in **LRFD [1.3.2.1]**

$Q = \sum \eta_i \gamma_I Q_i \leq \phi R_n = R_r$ **LRFD [1.3.2.1-1]**

Where:

η_I	=	Load modifier (a function of η_D, η_R, assumed 1.0 for retaining walls)
γ_I	=	Load factor
Q_i	=	Force effect
Q	=	Total factored force effect
ϕ	=	Resistance factor
R_n	=	Nominal resistance
R_r	=	Factored resistance = ϕR_n

14.4.5.2 Limit States

The limit states (as defined in **LRFD [3.4.1]**) that must be evaluated as part of the wall design requirements mainly include (1) Strength limit states; (2) Service limit states; and (3) Extreme Event limit states. The fatigue limit state is not used for retaining walls.

Strength limit state is applied to ensure that walls have adequate strength to resist external stability failure due to sliding, bearing resistance failure, etc. and internal stability failure such as pullout of reinforcement, etc. Evaluation of Strength limit states is accomplished by grouping factored loads and comparing to the reduced or factored soil strengths using resistance factors discussed in 14.4.5.6.

Service limit state is evaluated for overall stability and total or differential settlement checks. Evaluation of the Service limit states is usually performed by using expected service loads assuming a factor of 1.0 for nominal loads, a resistance factor of 1.0 for nominal strengths and elastic analyses.

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

14.4.5.3 Design Loads

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

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

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

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

14.4.5.4 Earth Pressure

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

Active Earth Pressure

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

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

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

For walls using Coulomb earth pressure theory:

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

Where:

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

δ = Friction angle between fill and wall (degrees)

B = Angle of fill to the horizontal (degrees)

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

ϕ'_f = Effective angle of internal friction (degrees)

Note: refer to Figure 14.4-1 for details.

For walls using Rankine earth pressure theory:

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

At-Rest Earth Pressure

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

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

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

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

Passive Earth Pressure

The development of passive earth pressure (K_p) requires a retaining wall to move into or toward the soil. As with the active earth pressure, Rankine earth pressure is the preferred method to be used to develop passive earth pressure coefficient. The use of Rankine theory will cause an under estimation of K_p, therefore resulting in a more conservative design. Coulomb earth pressure theory may be used if the appropriate conditions exist at a site; however, the designer is required to understand the limitations on the use of Coulomb earth pressure theory as applied to passive earth pressures.

Neglect any contribution from passive earth pressure in stability calculations unless the base of the wall extends below the depth to which foundation soil or rock could be weakened or removed by freeze-thaw, shrink-swell, scour, erosion, construction excavation, or any other means. In wall stability calculations, only the embedment below this depth, known as the effective embedment depth, shall be considered when calculating the passive earth pressure resistance. This is in accordance with **LRFD [11.6.3.5].**

14.4.5.4.1 Earth Load Surcharge

The effect of earth load surcharge including uniform, strip, and point loads shall be computed in accordance with **LRFD [3.11.6.1]** and **LRFD [3.11.6.2].**

14.4.5.4.2 Live Load Surcharge

Increased earth pressure on a wall occurs due to vehicular loading on top of the retained earth including operation of large or heavily-loaded cranes, staged equipment, soil stockpile or material storage, or any surcharge loads behind the walls. Earth pressure from live load surcharge shall be applied when a vehicular load is within one half of the wall height behind the back face of the wall or reinforced soil mass for MSE walls, in accordance with **LRFD [3.11.6.4].** In most cases, surcharge load can be modeled by assuming 2 ft of fill.

WisDOT policy item:

The equivalent height of soils for vehicular loading on retaining walls parallel to the traffic shall be 2.0 feet, regardless of the wall height. For standard unit weight of soil equal to 120 pcf, the resulting live load surcharge is 240 psf. Walls without traffic shall be designed for a live load surcharge of 100 psf to account for construction live loads.

14.4.5.4.3 Compaction Loads

Pressure induced by the compaction load can extend to variable depths due to the total static and dynamic forces exerted by compaction equipment. The effect of increased lateral earth pressure due to compaction loads during construction should be considered when compaction equipment is operated behind the wall. The compaction load surcharge effect is minimized by WISDOT standard specifications that require small walk behind compactors within 3 ft of the wall.

14.4.5.4.4 Wall Slopes

The slopes above and below the wall can significantly affect the earth pressures and wall stability. Slopes above the wall will influence the active earth pressure; slopes at the toe of the wall influences the passive earth pressures. In general, the back slope behind the wall should be no steeper than 2:1 (H:V). Where possible, a 4.0 ft wide horizontal bench should be provided at the front face of the wall.

14.4.5.4.5 Loading and Earth Pressure Diagrams

Loading and earth pressure diagrams are developed to compute nominal (unfactored) loads and moments. All applicable loads described in 14.4.5.3 and 14.4.5 shall be considered for computing nominal loads. For a typical wall, the force diagram for the earth pressure should be developed using a triangular distribution plus additional pressures resulting from earth or live load surcharge, water pressure, compaction etc. as discussed in 14.4.5.4.

The engineering properties for selected fill, concrete and steel are given in 14.4.6. The foundation and retained earth properties are selected as per discussions in 14.4.4 . One of the three cases is generally applicable for the development of loading diagrams and earth pressures:

1. Horizontal backslope with traffic surcharge

2. Sloping backslope

3. Broken backslope

Loading diagrams for CIP cantilever, MSE, modular block gravity, and prefabricated modular walls are shown for illustration. The designer shall develop loading diagrams as applicable.

CIP cantilever wall with sloping surcharge

For CIP cantilever walls, lateral active earth pressure shall be computed using Coulomb's theory for short heels or using Rankine theory for very long heels in accordance with the criteria presented in **LRFD [3.11.5.3]** and **LRFD [C3.11.5.3]**.

Walls resting on rock or batter piles can be designed for active earth pressure, based on WisDOT policy and in accordance with **LRFD [3.11.5.2].** Effect of the passive earth pressure on the front face of the wall shall be neglected in stability computation, unless the base of the wall extends below depth of maximum scour, freeze thaw or other disturbances in accordance with **LRFD [11.6.3.5]**.

Effect of surcharge loads ES present at the surface of the backfill of the wall shall be included in the analysis in accordance with 14.4.5.4.1. Walls with horizontal backfill shall be designed for live load surcharge in accordance with 14.4.5.4.2.

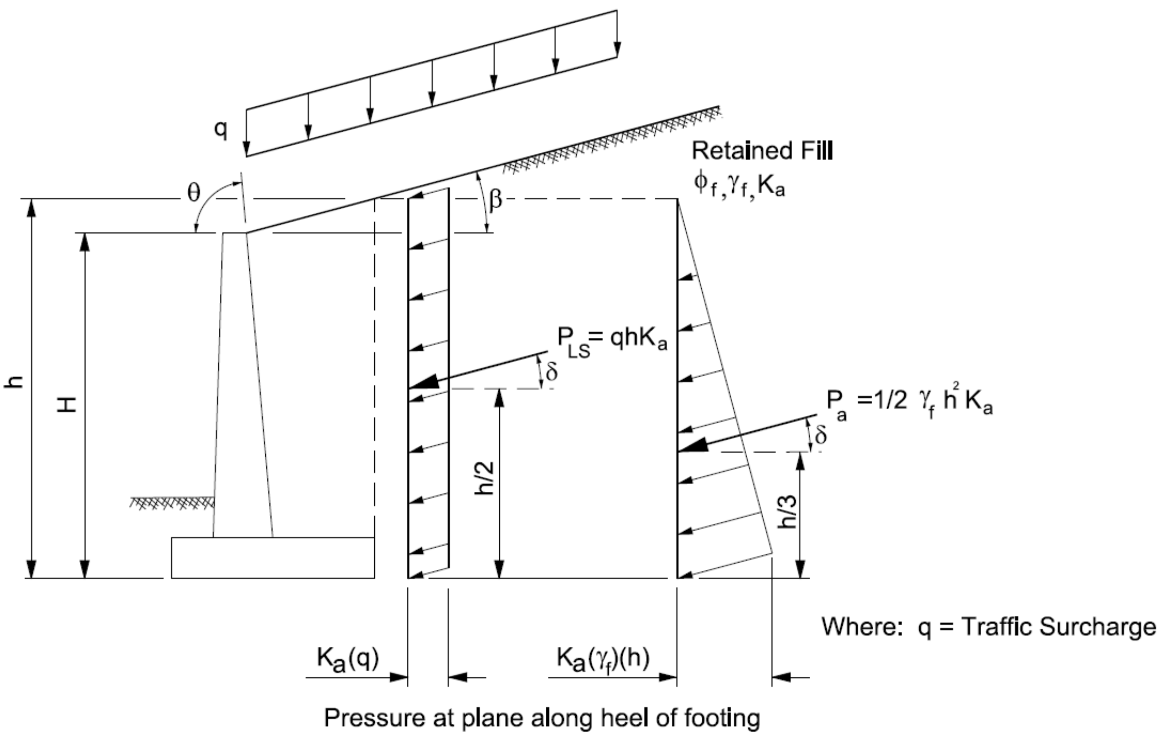


Figure 14.4-1
Loading Diagram for a Cantilever Retaining Wall with Surcharge Loading

MSE Walls

The loading and earth pressure diagram for an MSE wall shall be developed in accordance with **LRFD [11.10.5.2]** and described below for the three conditions defined earlier in this section.

MSE Wall with Horizontal Backslope and Traffic Surcharge

Figure 14.4-2 shows a procedure to estimate the earth pressure. The active earth pressure for horizontal backslope is computed using Rankine's theory as discussed in 14.4.5.4.

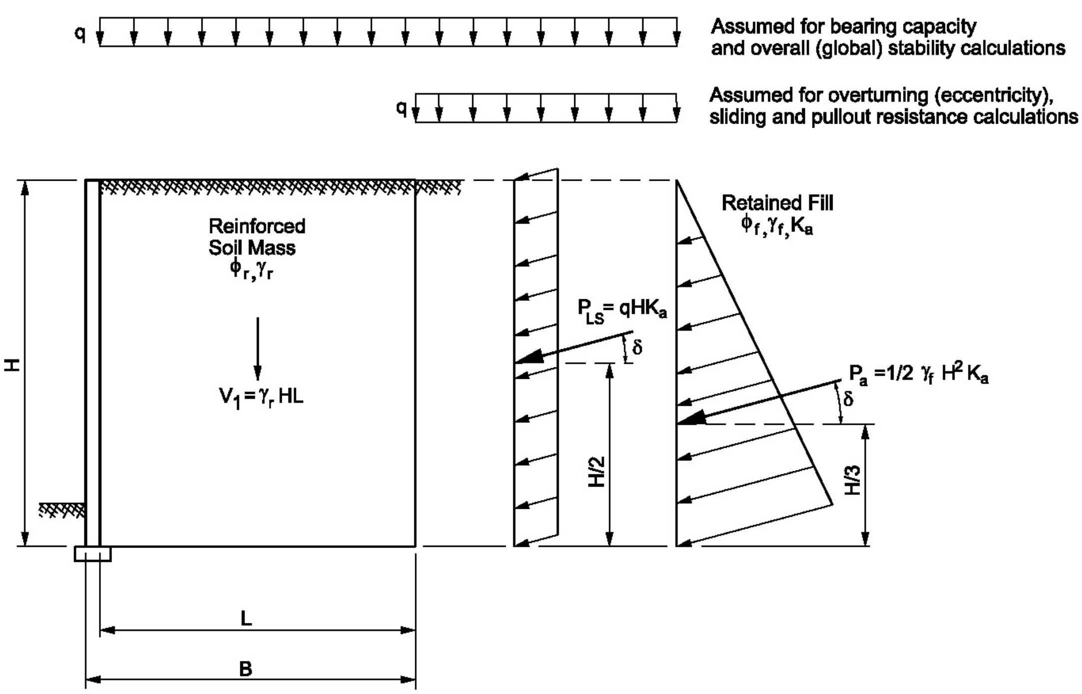


Figure 14.4-2
MSE Walls Earth Pressure for Horizontal Backslope with Traffic Surcharge
(Source **LRFD [Figure 11.10.5.2-1]**)

MSE Wall with Sloping Surcharge

Figure 14.4-3 shows a procedure to estimate the earth pressure. The active earth pressure for sloping backfill is computed using Coulomb's theory as discussed in 14.4.5.4.

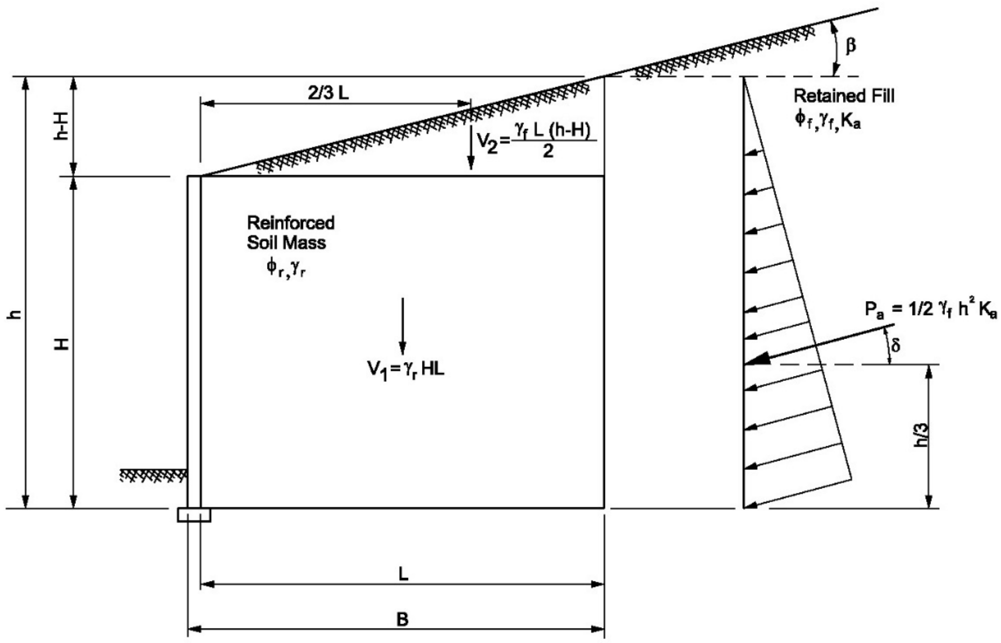


Figure 14.4-3
MSE Walls Earth Pressure for Sloping Backfill
(Source **LRFD [Figure 3.11.5.8.1-2]**)

MSE Wall with Broken Backslope

For broken backslopes, the active earth pressure coefficient is determined using Coulomb's equation except that surcharge angle β is substituted with slope angle β'.

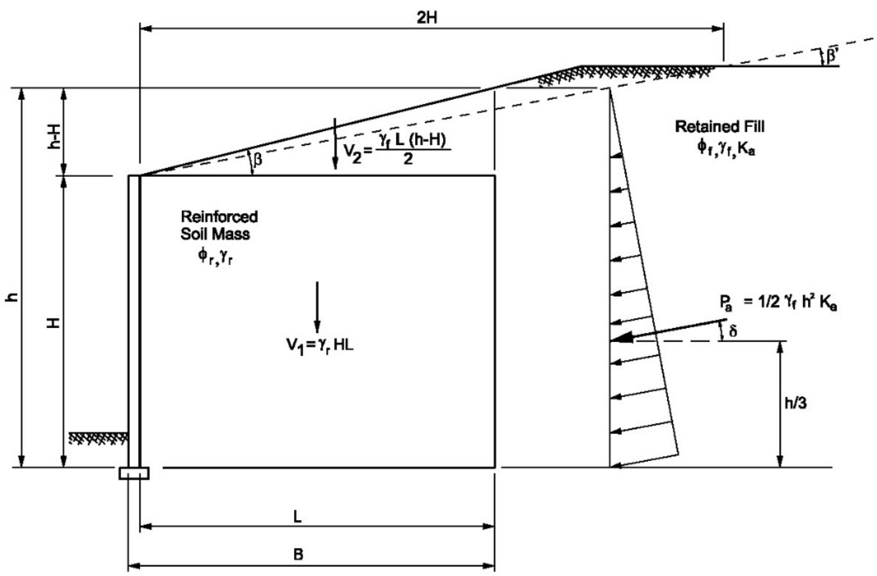


Figure 14.4-4
MSE Walls Earth Pressure for Broken Backfill
(Source **LRFD [Figure C3.11.5.8.1-1]**)

Modular Block Gravity Wall with Sloping Surcharge

When designing a “Modular Block Gravity Wall” without setback and with level backfill, the active earth pressure coefficient may be determined using Rankine theory as discussed in 14.4.5.4.

When designing a "Modular Block Gravity Wall" with setback, the active earth pressure coefficient K_a shall be determined using Coulomb theory as discussed in 14.4.5.4. The interface friction angle between the blocks and soil behind the blocks is assumed to be zero.

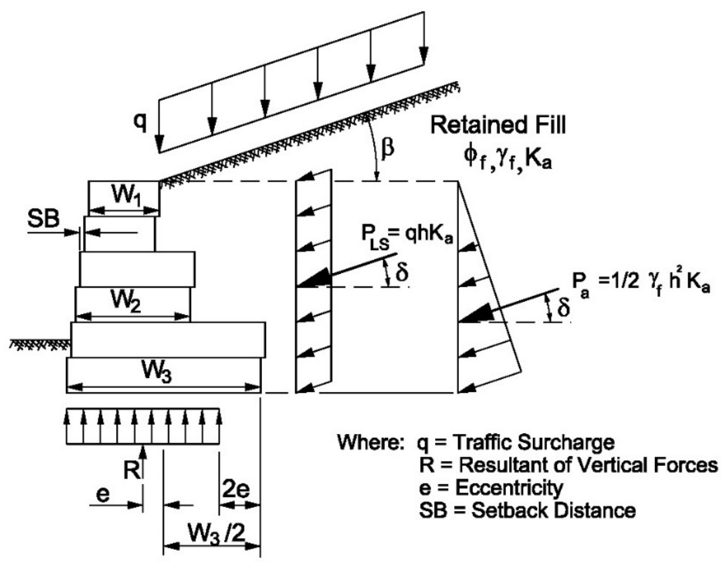


Figure 14.4-5
Modular Block Gravity Wall Analysis

No live load traffic and live load surcharge shall be allowed on modular block gravity walls although they are designed for a minimum live load of 100psf. The density of the blocks is assumed to be 135 pcf and the drainage aggregate inside or between the blocks 120 pcf. The forces acting on a modular block gravity wall are shown in Figure 14.4-5.

Prefabricated Modular Walls

Active earth pressure shall be determined by multiplying vertical loads by the coefficient of active earth pressure (K_a) and using Coulomb earth pressure theory in accordance with **LRFD [3.11.5.3] and LRFD [3.11.5.9]**. See Figure 14.4-6 for earth pressure diagram.

When the rear of the modules form an irregular surface (stepped surface), pressures shall be computed on an average plane surface drawn from the lower back heel of the lowest module as shown in Figure 14.4-7

Effect of the backslope soil surcharge and any other surcharge load imposed by existing structure should be accounted as discussed in 14.4.5.4. Trial wedge or Culmann method may also be used to compute the lateral earth pressure as presented in the *Foundation Analysis and Design*, 5th Edition (J. Bowles, 1996).

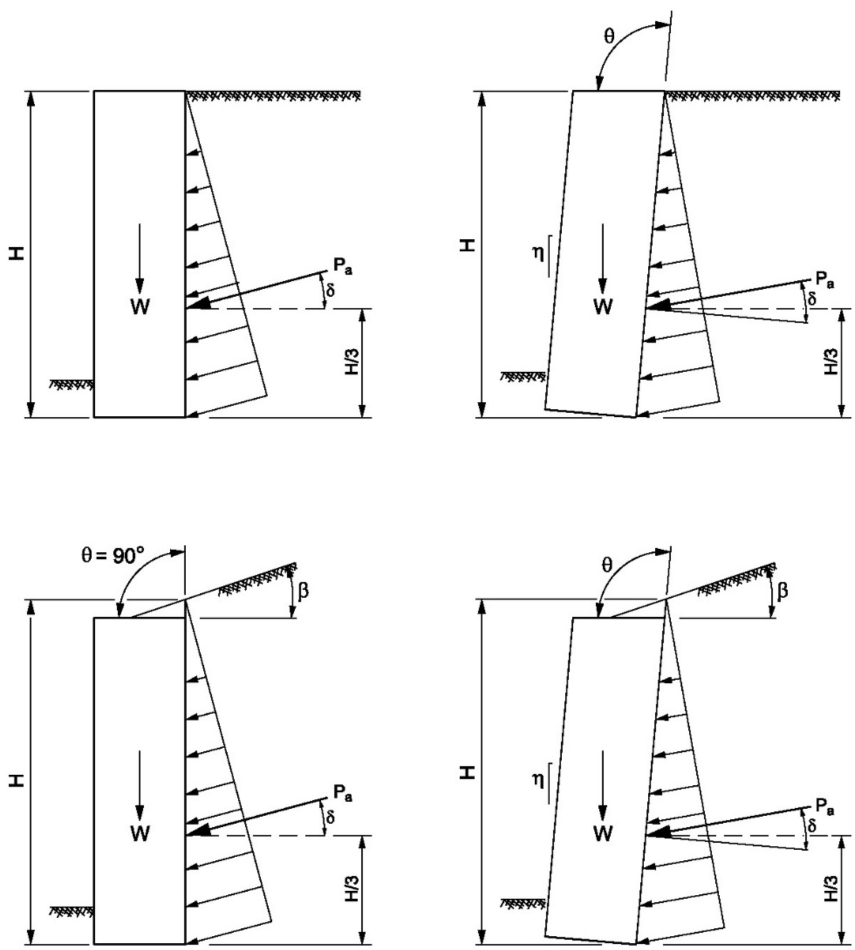


Figure 14.4-6
Lateral Earth Pressure on Concrete Modular Systems of Constant Width
(Source **LRFD [Figure 3.11.5.9-1]**)

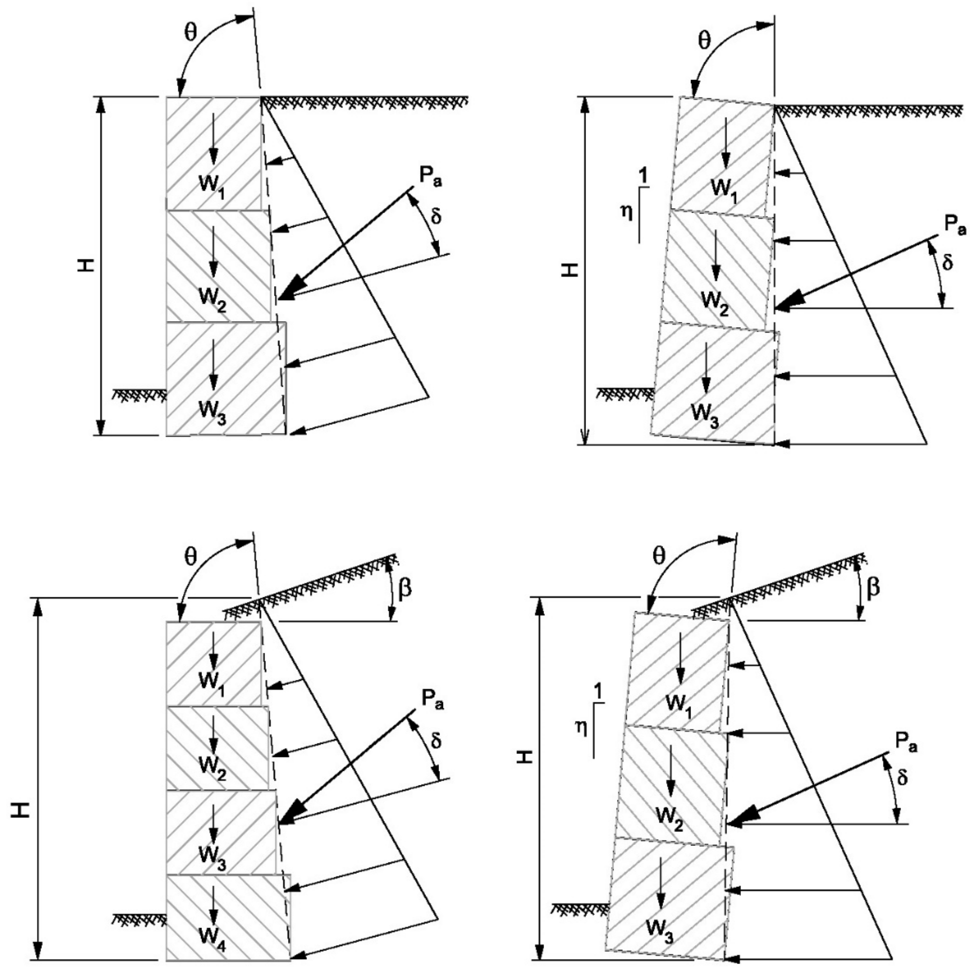


Figure 14.4-7
Lateral Earth Pressure on Concrete Modular Systems of Variable Width
(Source **LRFD [Figure 3.11.5.9-2]**)

14.4.5.5 Load factors and Load Combinations

The nominal loads and moments as described in 14.4.5.4.5 are factored using load factors found in **LRFD [Tables 3.4.1-1 and 3.4.1-2]**. The load factors applicable for most wall types considered in this chapter are given in Table 14.4-1. Load factors are selected to produce a total extreme factored force effect, and for each loading combination, both maximum and minimum extremes are investigated as part of the stability check, depending upon the expected wall failure mechanism.

Direction of Load	Load Type	Load Factor, γ_i		
		Strength I Limit		Service I Limit
		Maximum	Minimum	
Load Factors for Vertical Loads	Dead Load of Structural Components and Non-structural attachments DC	1.25	0.90	1.00
	Earth Surcharge Load ES	1.50	0.75	1.00
	Vertical Earth Load EV	1.35	1.00	1.00
	Water Load WA	1.00	1.00	1.00
	Live Load Surcharge LS	1.75	0.0	1.00
	Dead Load of Wearing Surfaces and Utilities DW	1.50	0.65	1.00
Load Factors for Horizontal Loads	Horizontal Earth Pressure EH Active At-Rest Passive	 1.50 1.35 1.35	 0.90 0.90 NA	 1.00 1.00 1.00
	Earth Surcharge ES	1.50	0.75	1.00
	Live Load Surcharge LS	1.75	1.75	1.00

Table 14.4-1
Load Factors

The factored loads are grouped to consider the force effect of all loads and load combinations for the specified load limit state in accordance with **LRFD [3.4.1]** and **LRFD [11.5.6].** Figure 14.4-8 illustrates the load factors and load combinations applicable for checking sliding stability and eccentricity for a cantilever wall at the Strength I limit state. This figure shows that structure weight DC is factored by using a load factor of 0.9 and the vertical earth load EV is factored by using a factor of 1.0. This causes contributing stabilizing forces against sliding to have a minimum force effect. At the same time, the horizontal earth load is factored by 1.5 resulting in maximum force effect for computing sliding at the base.

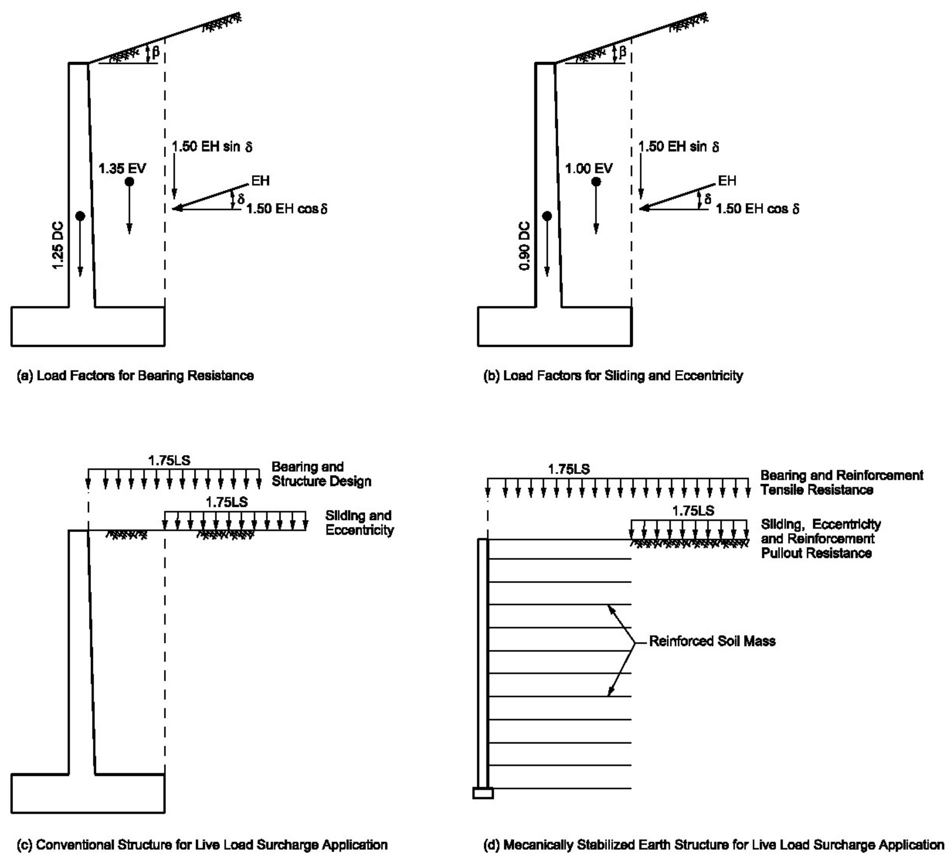


Figure 14.4-8
Application of Load Factors
(Source **LRFD [11.5.6]**)

14.4.5.6 Resistance Requirements and Resistance Factors

The wall components shall be proportioned by the appropriate methods so that the factored resistance as shown in **LRFD [1.3.2.1-1]** is no less than the factored loads, and satisfy criteria in accordance with **LRFD [11.5.4]** and **LRFD [11.6] thru [11.11]**. The factored resistance R_r is computed as follows: $R_r = \phi R_n$

Where

R_r	=	Factored resistance
R_n	=	Nominal resistance recommended in the Geotechnical Report
ϕ	=	Resistance factor

The resistance factors shall be selected in accordance with **LRFD [Tables 10.5.5.2.2-1, 10.5.5.2.3-1, 10.5.5.2.4-1, 11.5.7-1]**. Commonly used resistance factors for retaining walls are presented in Table 14.4-2.

14.4.6 Material Properties

The unit weight and strength properties of retained earth and foundation soil/rock (γ_f) are supplied in the geotechnical report and should be used for design purposes. Unless otherwise noted or recommended by the Designer or Geotechnical Engineer of record, the following material properties shall be assumed for the design and analysis if the selected backfill, concrete, and steel conforms to the WisDOT's *Standard Construction Specifications*:

<u>Granular Backfill Soil Properties</u>:

Internal Friction angle of backfill ϕ_f = 30 degrees

Backfill cohesion c = 0 psf

Unit Weight γ_f = 120 pcf

<u>Concrete</u>:

Compressive strength, f'_c at 28 days = 3500 psi

Unit Weight = 150 pcf

<u>Steel reinforcement</u>:

Yield strength f_y = 60,000 psi

Modulus of elasticity E_s = 29,000 ksi

Wall-Type and Condition		Resistance Factors
Mechanically Stabilized Earth Walls, Gravity Walls, and Semi-Gravity		
Bearing resistance	• Gravity & Semi-gravity • MSE	0.55 0.65
Sliding		1.00
Tensile resistance of metallic reinforcement and connectors	Strip reinforcement • Static loading Grid reinforcement • Static loading	0.75 0.65
Tensile resistance of geo-synthetic reinforcements and connectors	• Static loading	0.90
Pullout resistance of tensile reinforcement	• Static loading	0.90
Prefabricated Modular Walls		
Bearing		**LRFD [10.5]**
Sliding		**LRFD [10.5]**
Passive resistance		**LRFD [10.5]**
Non-Gravity Cantilevered and Anchored Walls		
Axial compressive resistance of vertical elements		**LRFD [10.5]**
Passive resistance of vertical elements		0.75
Pullout resistance of anchors	• Cohesionless soils • Cohesive soils • Rock	0.65 0.70 0.50
Pullout resistance of anchors	• Where proof tests are conducted	1.00
Tensile resistance of anchor tendons	• Mild steel • High strength steel	0.90 0.80
Flexural capacity of vertical elements		0.90

Table 14.4-2
Resistance Factors
(Source **LRFD [Table 11.5.7-1]**)

14.4.7 Wall Stability Checks

During the design process, walls shall be checked for anticipated failure mechanisms relating to external stability, internal stability (where applicable), movement and overall stability. In general, external and internal stability of the walls should be investigated at Strength limit states, in accordance with **LRFD [11.5.1]**. In addition, investigate the wall stability for excessive vertical and lateral displacement and overall stability at the Service limit states in accordance with **LRFD [11.5.2]**. Figure 14.4-2 thru Figure 14.4-14 show anticipated failure mechanisms for various types of walls.

14.4.7.1 External Stability

The external stability should be satisfied (generally performed by the Geotechnical Engineer) for all walls. The external stability check should include failure against lateral sliding, overturning (eccentricity), and bearing pressure failure as applicable for gravity or non-gravity wall systems in accordance with **LRFD [11.5.3]**. External stability checks should be performed at the Strength I limit state.

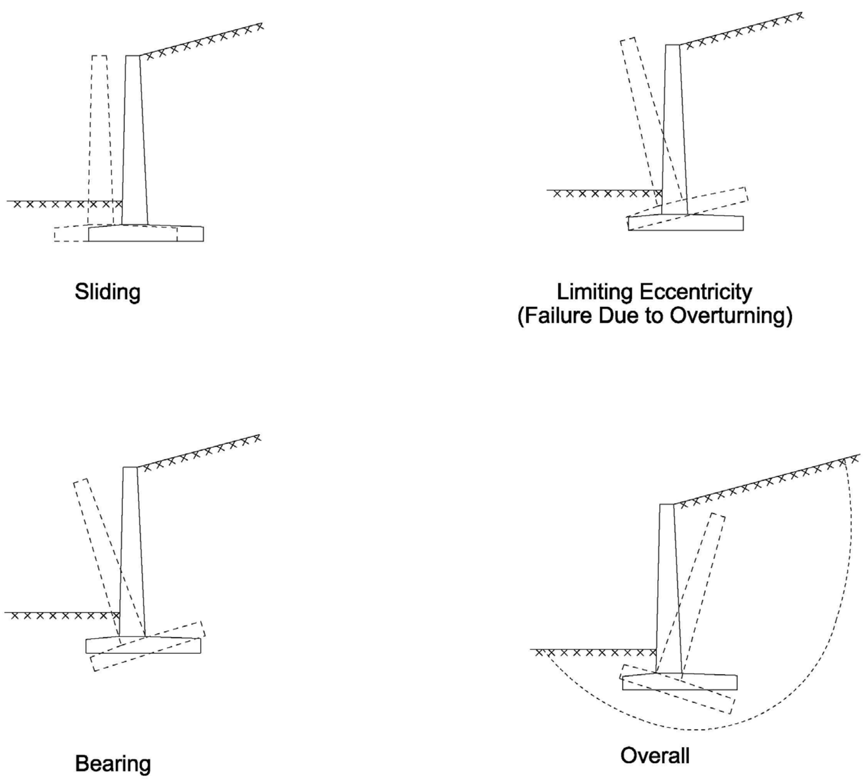


Figure14.4-9
External Stability Failure of CIP Semi-Gravity Walls

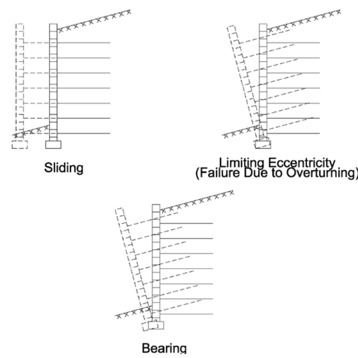


Figure 14.4-10
External Stability Failure of MSE Walls

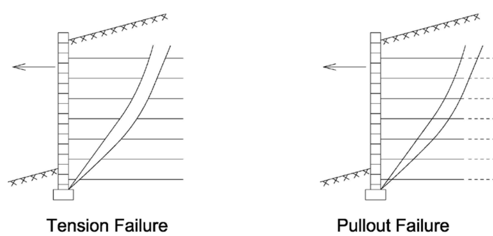


Figure 14.4-11
Internal Stability Failure of MSE Walls

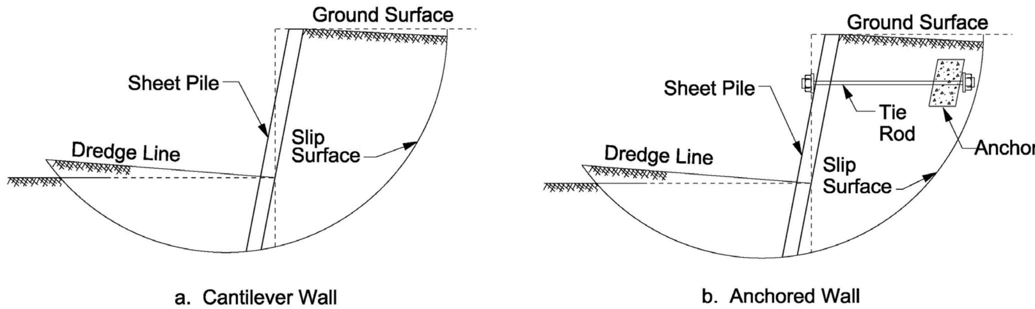


Figure 14.4-12
Deep Seated Failure of Non-Gravity Walls

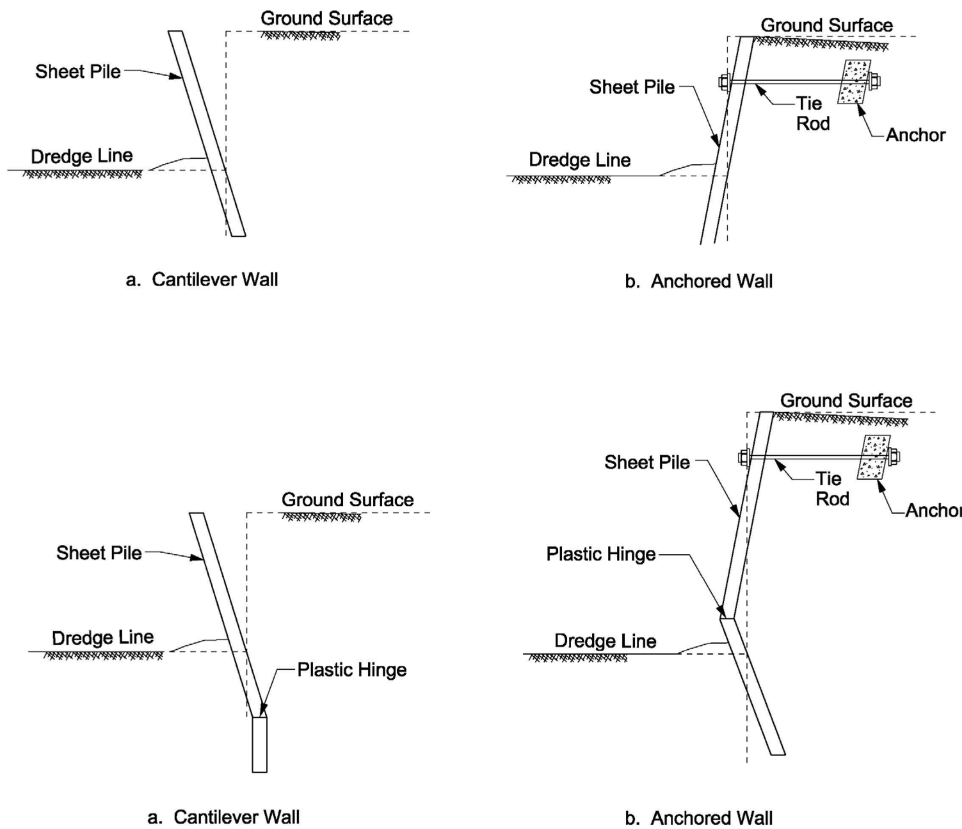


Figure 14.4-13
Flexural Failure of Non-Gravity Walls

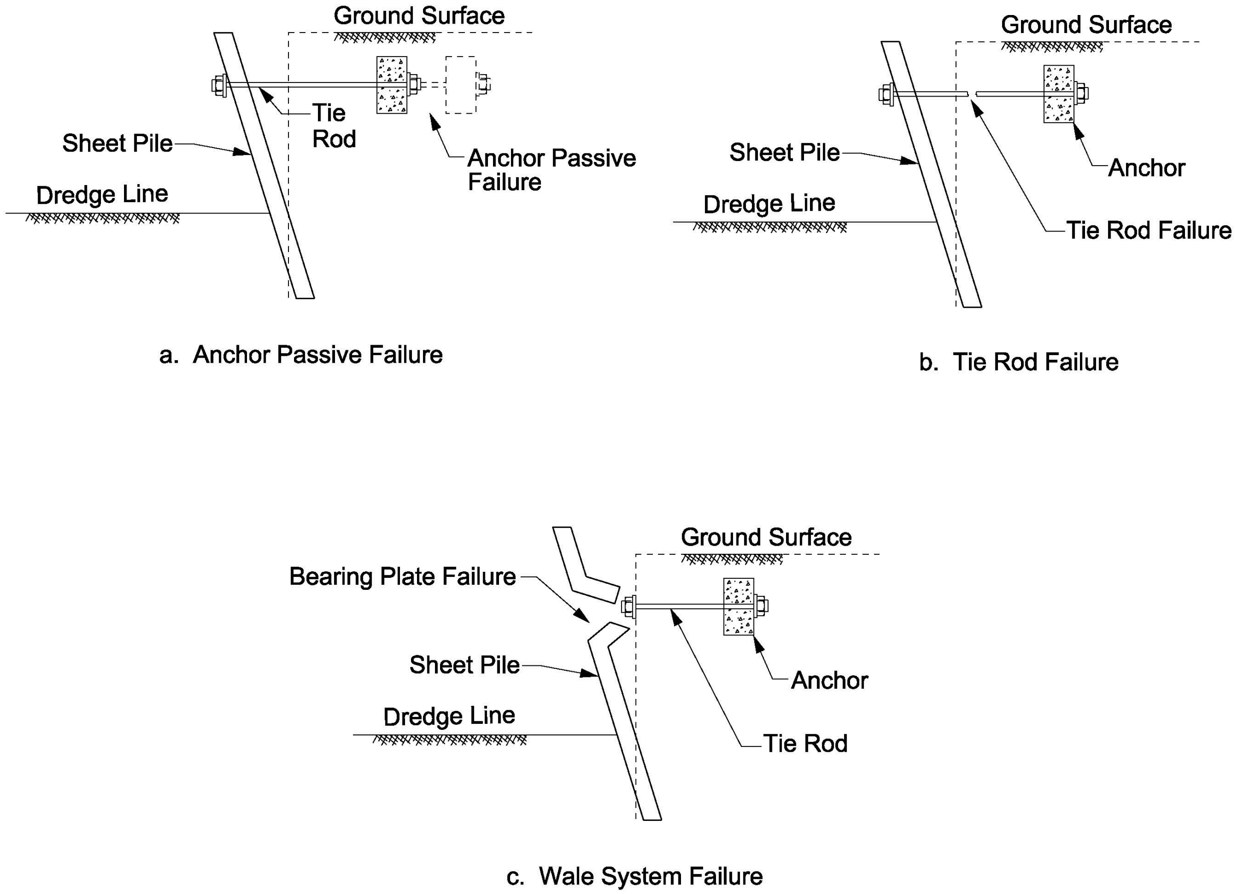


Figure 14.4-14
Flexural Failure of Non-Gravity Walls

14.4.7.2 Wall Settlement

Retaining walls shall be designed for the effects of total and differential foundation settlement at the Service I limit state, in accordance with **LRFD [11.5.2]** and 11.2. Maximum tolerable retaining wall total and differential foundation settlements are controlled largely by the potential for cosmetic and/or structural damage to facing elements, copings, barrier, guardrail, signs, pavements, utilities, structure foundations, and other highway appurtenances supported on or near the retaining wall.

14.4.7.2.1 Settlement Guidelines

The following table provides guidance for maximum tolerable vertical and total differential Settlement for various retaining wall types where Δh is the total settlement in inches and

Wall Type	Total Settlement Δh in inches	Total Differential Settlement $\Delta h1$:L (in/in)
CIP semi-gravity cantilever walls	1-2	1:500
MSE walls with large pre-cast panel facing (panel front face area >30ft^2)	1-2	1:500
MSE walls with small pre-cast panel facing (panel front face area <30ft^2)	1-2	1:300
MSE walls with full-height cast-in-panel facing	1-2	1:500
MSE walls with modular block facing	2-4	1:200
MSE walls with geotextile /welded-wire facing	4-8	1:50-1:60
Modular block gravity walls	1-2	1:300
Concrete Crib walls	1-2	1:500
Bin walls	2-4	1:200
Gabion walls	4-6	1:50
Non-gravity cantilever and anchored walls	1-2.5	----

Table 14.4-3
Maximum Tolerable Settlement Guidelines for Retaining Walls

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

14.4.7.3 Overall Stability

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

14.4.7.4 Internal Stability

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

14.4.7.5 Wall Embedment

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

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

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

14.4.7.6 Wall Subsurface Drainage

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

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

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

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

14.4.7.7 Scour

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

14.4.7.8 Corrosion

All metallic components of WISDOT retaining wall systems subjected to corrosion, should be designed to last through the designed life of the walls. Corrosion protection should be designed in accordance with the criteria given in **LRFD [11.10.6].** In addition, **LRFD [11.8.7]**, **[11.9.7]** and **[11.10]** also include design guidance for corrosion protection on non-gravity cantilever walls, anchored walls and MSE walls respectively.

14.4.7.9 Utilities

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

14.4.7.10 Guardrail and Barrier

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

14.5 Cast-In-Place Concrete Cantilever Walls

14.5.1 General

A cast-in-place, reinforced concrete cantilever wall is a semi-gravity wall that consists of a base slab or footing from which a vertical wall or stem extends upward. Reinforcement is provided in both members to supply resistance to bending. These walls are generally founded on good bearing material. Cantilever walls shall not be used without pile support if the foundation stratum is prone to excessive vertical or differential settlement, unless subgrade improvements are made. Cantilever walls are typically designed to a height of 28 feet. For heights exceeding 28 feet, consideration should be given to providing a counterfort. Design of counterfort CIP walls is not covered in this chapter.

CIP cantilever walls shall be designed in accordance with *AASHTO LRFD,* design concepts presented in 14.4 and the *WisDOT Standard Specifications* including the special provisions.

14.5.2 Design Procedure for Cast-in-Place Concrete Cantilever Walls

The CIP wall shall be designed to resist lateral pressure caused by supported earth, surcharge loads and water in accordance with **LRFD [11.6].** The external stability, settlement, and overall stability shall be evaluated at the appropriate load limit states in accordance with **LRFD [11.5.5],** to resist anticipated failure mechanism. The structural components mainly stem and footing should be designed to resist flexural resistance in accordance with **LRFD [11.6.3].**

Figure 14.5-1 shows possible external stability failure and deep seated rotational failure mechanisms of CIP cantilever walls that must be investigated as part of the stability check.

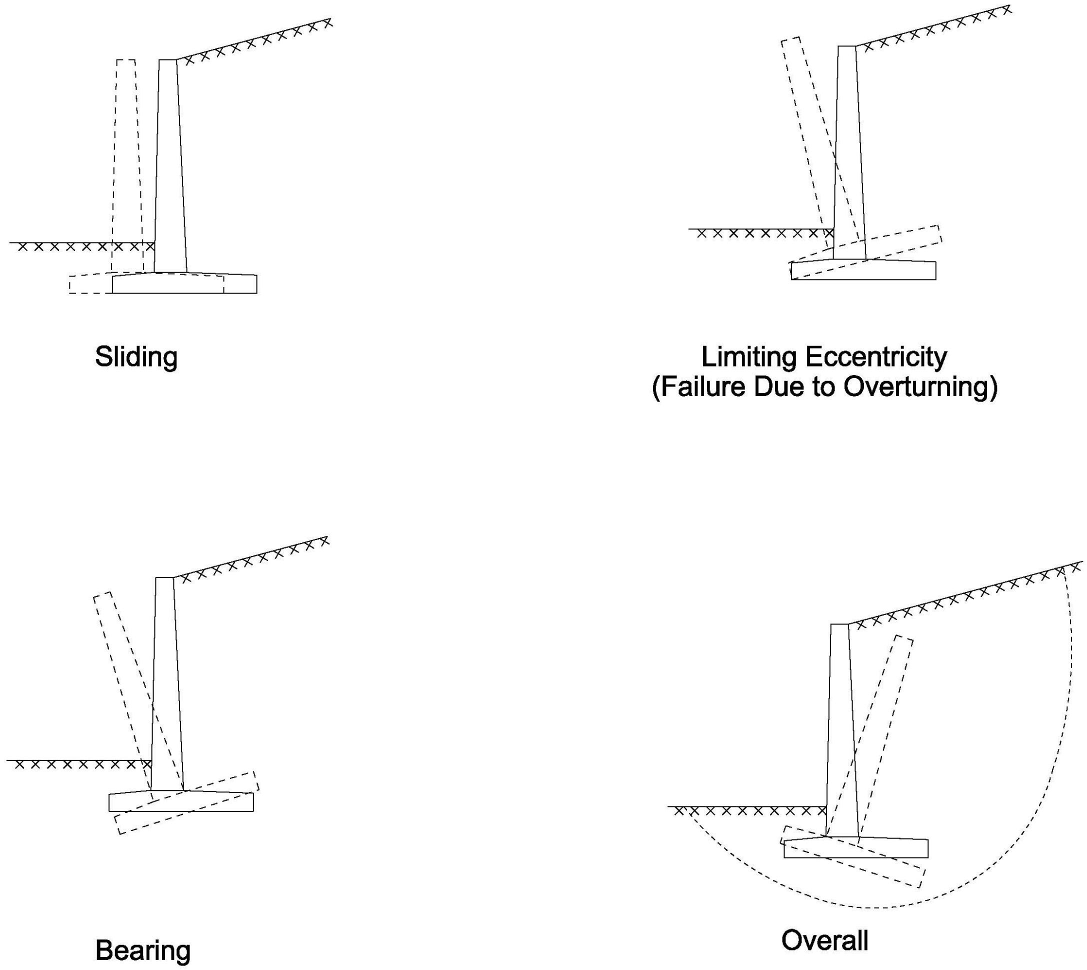


Figure 14.5-1
CIP Semi-Gravity Wall Failure Mechanism

14.5.2.1 Design Steps

The general design steps discussed in 14.4.1 shall be followed for the wall design. These steps as applicable for CIP cantilever walls are summarized below.

1. Establish project requirements including wall height, geometry and wall location as discussed in 14.1 of this chapter.

2. Perform Geotechnical investigation

3. Develop soil strength parameters

4. Determine preliminary sizing for external stability evaluation

5. Determine applicable unfactored or nominal loads

6. Evaluate factored loads for all appropriate limit states

7. Perform stability check to evaluate bearing resistance, eccentricity, and sliding as part of external stability

8. Estimate wall settlement and lateral wall movement to meet guidelines stated in Table 14.4-3.

9. Check overall stability and revise design, if necessary, by repeating steps 4 to 8.

It is assumed that steps 1, 2 and 3 have been performed prior to starting the design process.

14.5.3 Preliminary Sizing

A preliminary design can be performed using the following guideline.

1. The wall height and alignment shall be selected in accordance with the preliminary plan preparation process discussed in 14.1.

2. Preliminary CIP wall design may assume a stem top width of 12 inches. Stem thickness at the bottom is based on load requirements and/or batter. The front batter of the stem should be set at ¼ inch per foot for stem heights up to 28 feet. For stem heights from 16 feet to 26 feet inclusive, the back face batter shall be a minimum of ½ inch per foot, and for stem heights of 28 ft maximum and greater, the back face shall be ¾ inch per foot per stability requirements.

3. Minimum Footing thickness for stem heights equal to or less than 10 ft shall be 1.5 ft and 2.0 ft when the stem height exceeds 10 ft or when piles are used.

4. The base of the footing shall be placed below the frost line, or 4 feet below the finished ground line. Selection of shallow footing or deep foundation shall be based on the geotechnical investigation, which should be performed in accordance with guidelines presented in Chapter 11 - Foundation Support.

5. The final footing embedment shall be based on wall stability requirements including bearing resistance, wall settlement limitations, external stability, internal stability and overall stability requirements.

6. If the finished ground line is on a grade, the bottom of footings may be sloped to a maximum grade of 12 percent. If the grade exceeds 12 percent, place the footings level and use steps.

The designer has the option to vary the values of each wall component discussed in steps 2 to 6 above, depending on site requirements and to achieve economy. See Figure 14.5-2 for initial wall sizing guidance.

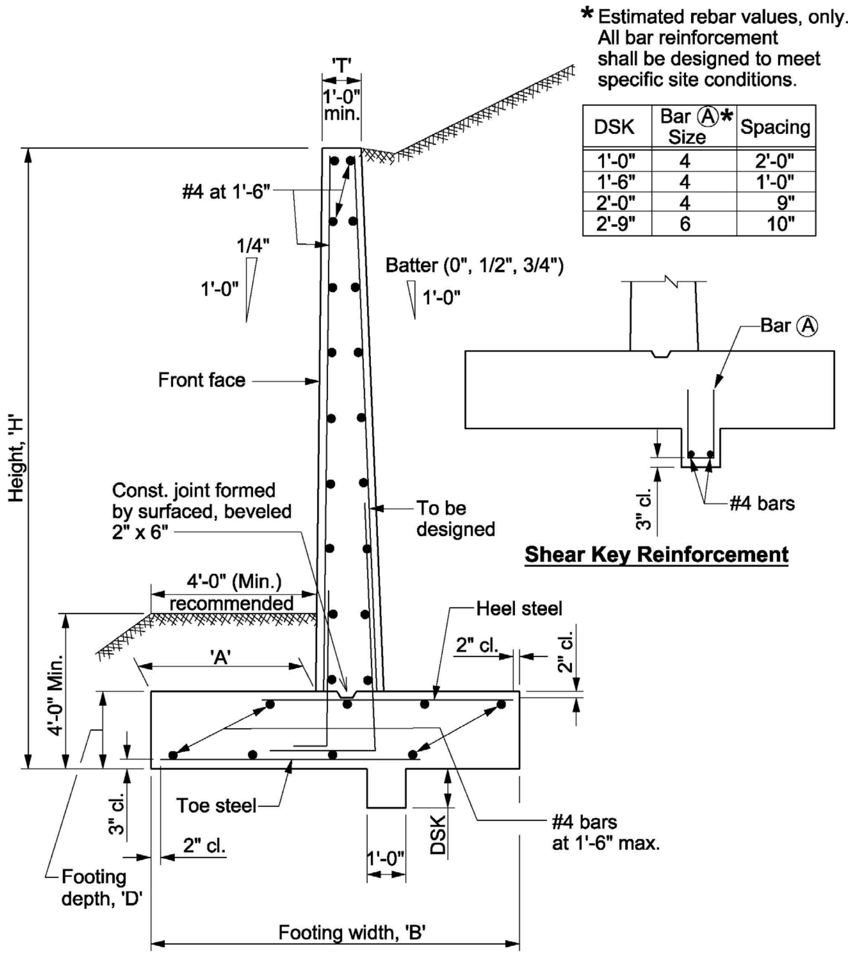


Figure 14.5-2
CIP Walls General Details

14.5.3.1 Wall Back and Front Slopes

CIP walls shall not be designed for backfill slope steeper than 2:1(H:V). Where practical, walls shall have a horizontal bench of 4.0 feet wide at the front face.

14.5.4 Unfactored and Factored Loads

Unfactored loads and moments are computed after establishing the initial wall geometry and using procedures defined in 14.4.5.4.5. A load diagram as shown in Figure **14.4-1** for the earth pressure is developed assuming a triangular distribution plus additional pressures resulting from earth surcharge, water pressure, compaction or any other loads, etc. The material properties for backfill soil, concrete and steel are given in 14.4.6. The foundation and retained earth properties as recommended in the Geotechnical Report shall be used for computing nominal loads.

The computed nominal loads discussed in 14.5.4 are multiplied by applicable load factors given in Table 14.4-1. Figure 14.4-8 shows load factor and load combinations along with their application for the load limit state evaluation. A summary of load factors and load combinations as applicable for a typical CIP cantilever wall is presented in Table 14.4-1 and **LRFD [3.4.1]**, respectively. Computed factored loads and moments are used for performing stability checks.

14.5.5 External Stability Checks

The external stability check includes checks for limiting eccentricity (overturning), bearing stress, and sliding at Strength I and Extreme Event II due to vehicle impact in cases where live load traffic is carried.

14.5.5.1 Eccentricity Check

The eccentricity of the retaining wall shall be evaluated in accordance with **LRFD [11.6.3.3]**. The location of the resultant force should be within 1/3 of base width of the foundation centroid (e<B/3) for foundations on soil, and within 0.45 of the base width of the foundation centroid (e<0.45B) for foundations on rock. If there is inadequate resistance to overturning (eccentricity value greater than limits given above), consideration should be given to either increasing the width of the wall base, or providing a deep foundation.

14.5.5.2 Bearing Resistance

The bearing resistance shall be evaluated at the strength limit state using factored loads and resistances. Bearing resistance of the walls founded directly on soil or rock shall be computed in accordance with 11.2 and **LRFD [10.6]**. The bearing resistance for walls on piles shall be computed in accordance with 11.3 and **LRFD [10.6]**. Figure 14.5-3 shows bearing stress criteria for a typical CIP wall on soil and rock respectively.

The vertical stress for footings on soil shall be calculated using:

$$\sigma_v = \frac{\sum V}{(B-2e)}$$

For walls founded on rock, the vertical stress is calculated assuming a linearly distributed pressure over an effective base area. The vertical stress for footings on rock shall be computed using:

$$\sigma_v = \frac{\Sigma V}{B}\left(1 \pm \frac{6e}{B}\right)$$

Where

ΣV = Summation of vertical forces

B = Base width

e = Eccentricity as shown in Figure 14.5-3 and Figure 14.5-4

If the resultant is outside the middle one-third of the wall base, then the vertical stress shall be computed using:

$$\sigma_{v\,max} = \left(\frac{2\sum V}{3\left(\frac{B}{2} - e\right)} \right)$$

$\sigma_{vmin} = 0$

The computed vertical stress shall be compared with factored bearing resistance in accordance with the **LRFD [10.6.3.1]** using following equation:

$q_r = \phi_b q_n > \sigma_v$

Where:

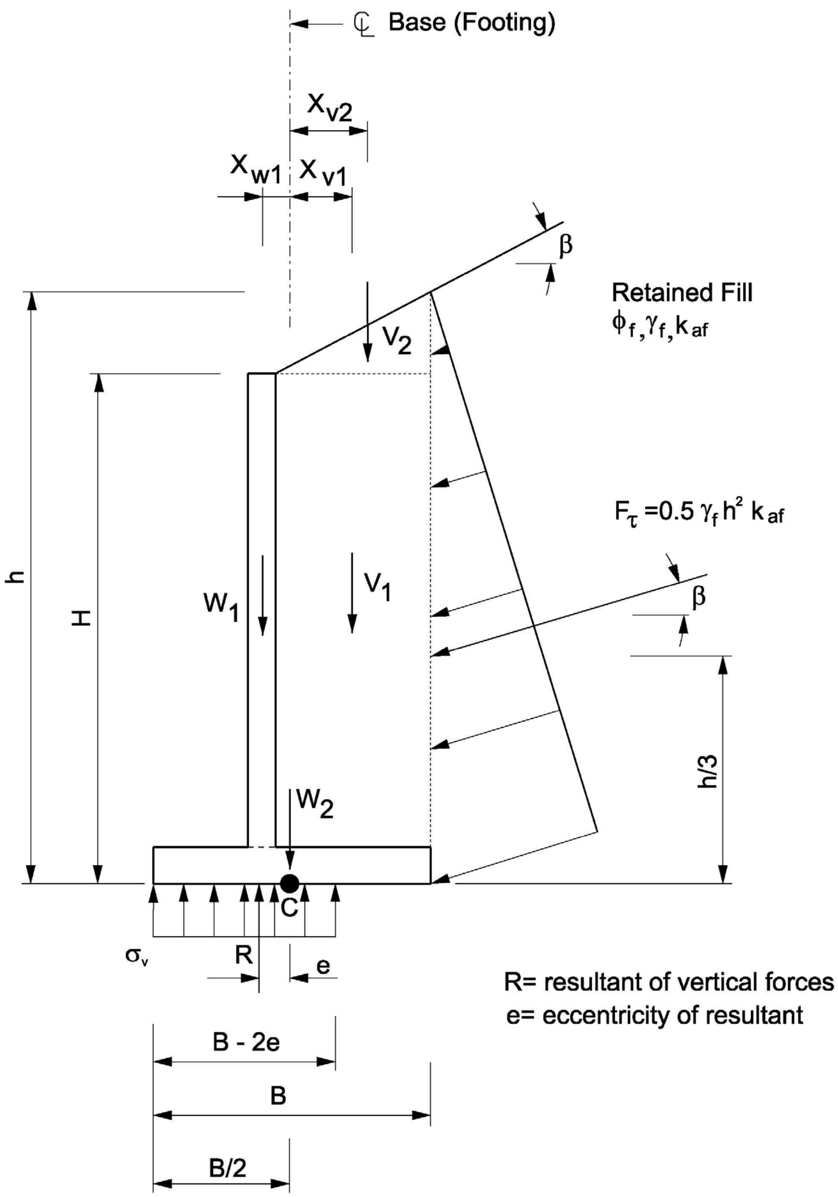
q_r = Factored bearing resistance

q_n = Nominal bearing resistance computed using **LRFD [10.6.3.1.2-a]**

σ_v = Vertical stress

B = Base width

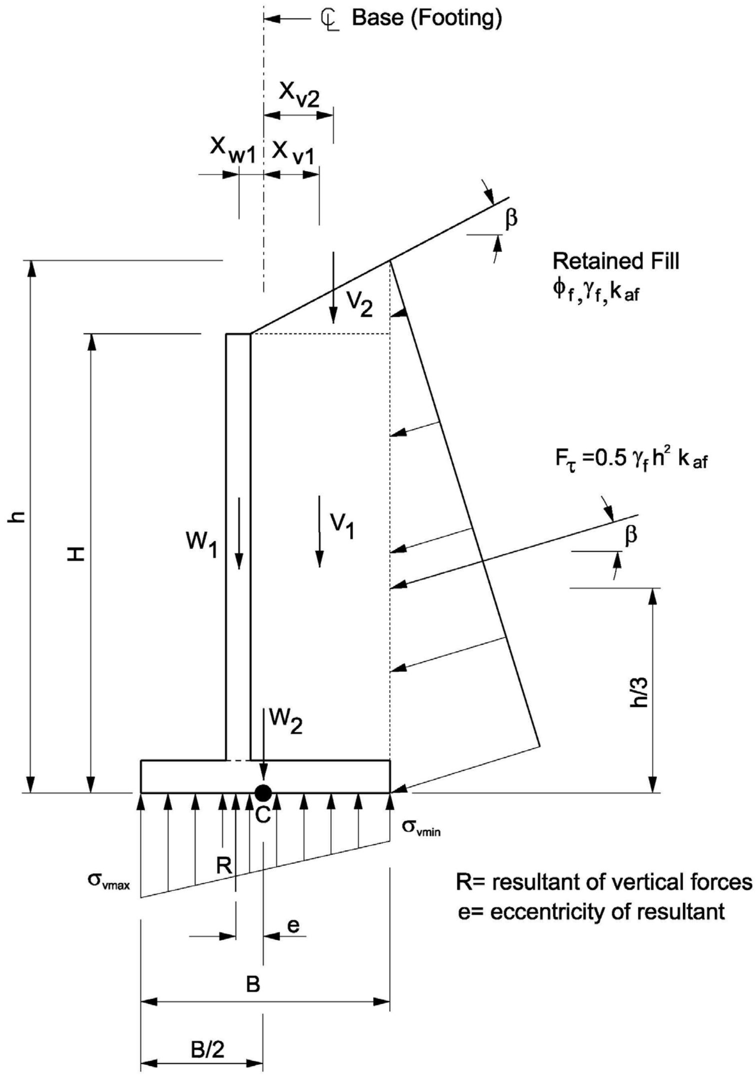
e = Eccentricity as shown in Figure 14.5-3 and Figure 14.5-4



Summing Moments about Point C:

$$e = \frac{(F_T \cos\beta)h/3 - (F_T \sin\beta)B/2 - V_1 X_{V1} - V_2 X_{V2} + W_1 X_{W1}}{V_1 + V_2 + W_1 + W_2 + F_T \sin\beta}$$

Figure 14.5-3
Loading Diagram and Bearing Stress Criteria for CIP Cantilever Walls on Soil
(source *AASHTO LRFD*)



If e > B/6, σ_{vmin} will drop to zero, and as "e" increases, the portion of the heel of the footing which has zero vertical stress increases.

Summing Moments about Point C:

$$e = \frac{(F_T \cos\beta)h/3 - (F_T \sin\beta)B/2 - V_1 X_{V1} - V_2 X_{V2} + W_1 X_{W1}}{V_1 + V_2 + W_1 + W_2 + F_T \sin\beta}$$

Figure 14.5-4

Loading Diagram and Bearing Stress Criteria for CIP Cantilever Walls on Rock
(source *AASHTO LRFD*)

14.5.5.3 Sliding

The sliding resistance of CIP cantilever walls is computed by considering the wall as a shallow footing resting on soil/rock or footing resting on piles in accordance with **LRFD [10.5]**. Sliding resistance of a footing resting on soil/rock foundation is computed in accordance with the **LRFD [10.6.3.4]** using the equation given below:

$$R_R = \phi\, R_n = \phi_\tau\, R_\tau + \phi_{ep} R_{ep}$$

Where:

R_R	=	Factored resistance against failure by sliding
R_n	=	Nominal sliding resistance against failure by sliding
ϕ_τ	=	Resistance factor for shear between soil and foundation per **LRFD [Table 10.5.5.2.2.1]**
R_τ	=	Nominal sliding resistance between soil and foundation
ϕ_{ep}	=	Resistance factor for passive resistance per **LRFD Table [10.5.5.2.2.1]**
R_{ep}	=	Nominal passive resistance of soil throughout the life of the structure

Contribution from passive earth pressure resistance against the embedded portion of the wall is neglected if the soil in front of the wall can be removed or weakened by scouring, erosion or any other means. Also, the live load surcharge is not considered as a stabilizing force over the heel of the wall when checking sliding.

If adequate sliding resistance cannot be achieved, footing design may be modified as follows:

- Increase the base width of the footing
- Construct a shear key
- Increase wall embedment to a sufficient depth, where passive resistance can be relied upon
- Incorporate a deep foundation, including battered piles (Usually a costly measure)

Guideline for selecting the shear key design is presented in 14.5.7.3. The design of wall footings resting on piles is performed in accordance with **LRFD [10.5]** and Chapter 11 - Foundation Support. Footings on piles resist sliding by the following:

1. Passive earth pressure in front of wall. Same as spread footing.
2. Lateral resistance of vertical piles as well as the horizontal components of battered piles. Maximum batter is 3 inches per foot. Refer to Chapter 11 - Foundation Support for lateral load capacity of piles.

3. Lateral resistance of battered or vertical piles in addition to horizontal component of battered piles. Refer to Chapter 11- Foundation Support for allowable lateral load capacity.

4. Do not use soil friction under the footing as consolidation of the soil may eliminate contact between the soil and footing.

14.5.5.4 Settlement

The settlement of CIP cantilever walls can be computed in accordance with guidelines and performance criteria presented in 14.4.7.2. The guideline for total and differential settlement is presented in Table 14.4-3. The actual performance limit can be changed for specific project requirements. For additional guidance contact the Geotechnical Engineering Unit.

14.5.6 Overall Stability

Investigate Service 1 load combination using an appropriate resistance factor and procedures discussed in **LRFD [11.6]** and 14.4.7.3. In general, the resistance factor, φ, may be taken as;

- 0.75 - where the geotechnical parameters are well defined, and slope does not support or contain a structural element.

- 0.65 – where the geotechnical parameters are based on limited information or the slope contains or supports a structural element.

14.5.7 Structural Resistance

The structural design of the stem and footing shall be performed in accordance with *AASHTO LRFD* and the design guidelines discussed below.

14.5.7.1 Stem Design

The initial sizing of the stem should be selected in accordance with criteria presented in 14.5.3. The stems of cantilever walls shall be designed as cantilevers supported at the footing. Axial loads (including the weight of the wall stem and frictional forces due to backfill acting on the wall stem) shall be considered in addition to the bending due to eccentric vertical loads, surcharge loads and lateral earth pressure if they control the design of the wall stems. The flexural design of the cantilever wall should be performed in accordance with *AASHTO LRFD*.

Loads from railings or parapets on top of the wall need not be applied simultaneously with live loads. These are dynamic loads which are resisted by the mass of the wall.

14.5.7.2 Footing Design

The footing of a cantilever wall shall be designed as a cantilever beam. The heel section must support the weight of the backfill soil and the shear component of the lateral earth pressure. All loads and moments must be factored using the criteria load factors discussed in 14.5.4. Use the following criteria when designing the footing.

1. Minimum footing thickness shall be selected in accordance with criteria presented in 14.5.3. The final footing thickness shall be based on shear at a vertical plane behind the stem.

2. For toe, design for shear at a distance from the face of the stem equal to the effective "d" distance of the footing. For heel, design for shear at the face of stem.

3. Where the footing is resting on piles, the piles shall be designed in accordance with criteria for pile design presented in Chapter 11 – Foundation Support. Embed piles six inches into footing. Place bar steel on top of the piles.

4. For spread footings, use a minimum of 3 inches clear cover at the bottom of footing. Use 2 inches clear cover for edge distance.

5. The critical sections for bending moments in footings shall be taken at the front and back faces of the wall stem. Bearing pressure along the bottom of the heel extension may conservatively be ignored. No bar steel is provided if the required area per foot is less than 0.05 square inches.

6. Design for heel moment, without considering the upward soil or pile reaction, is not required unless such a condition actually exists.

14.5.7.3 Shear Key Design

A shear key shall be provided to increase the sliding resistance when the factored sliding resistance determined using procedure discussed in 14.5.5.3 is inadequate. Use the following criteria when designing the shear key:

1. Place shear key in line with stem except under severe loading conditions.

2. The key width is 1'-0" in most cases. The minimum key depth is 1'-0".

3. Place shear key in unformed excavation against undisturbed material.

4. Analyze shear key in accordance with **LRFD [10.6.3.4]** and 14.5.5.3 .

5. The shape of shear key in rock is governed by the quality of the rock, but in general a 1 ft. by 1 ft key is appropriate.

14.5.7.4 Miscellaneous Design Information

1. Contraction joints shall be provided at intervals not exceeding 30 feet and expansion joints at intervals not exceeding 90 feet for reinforced concrete walls. Typical details of expansion and contraction joints are given in Figure 14.5-5. Expansion joints shall be constructed with a joint, filling material of the appropriate thickness to ensure the functioning of the joint and shall be provided with a waterstop capable of functioning over the anticipated range of joint movements.

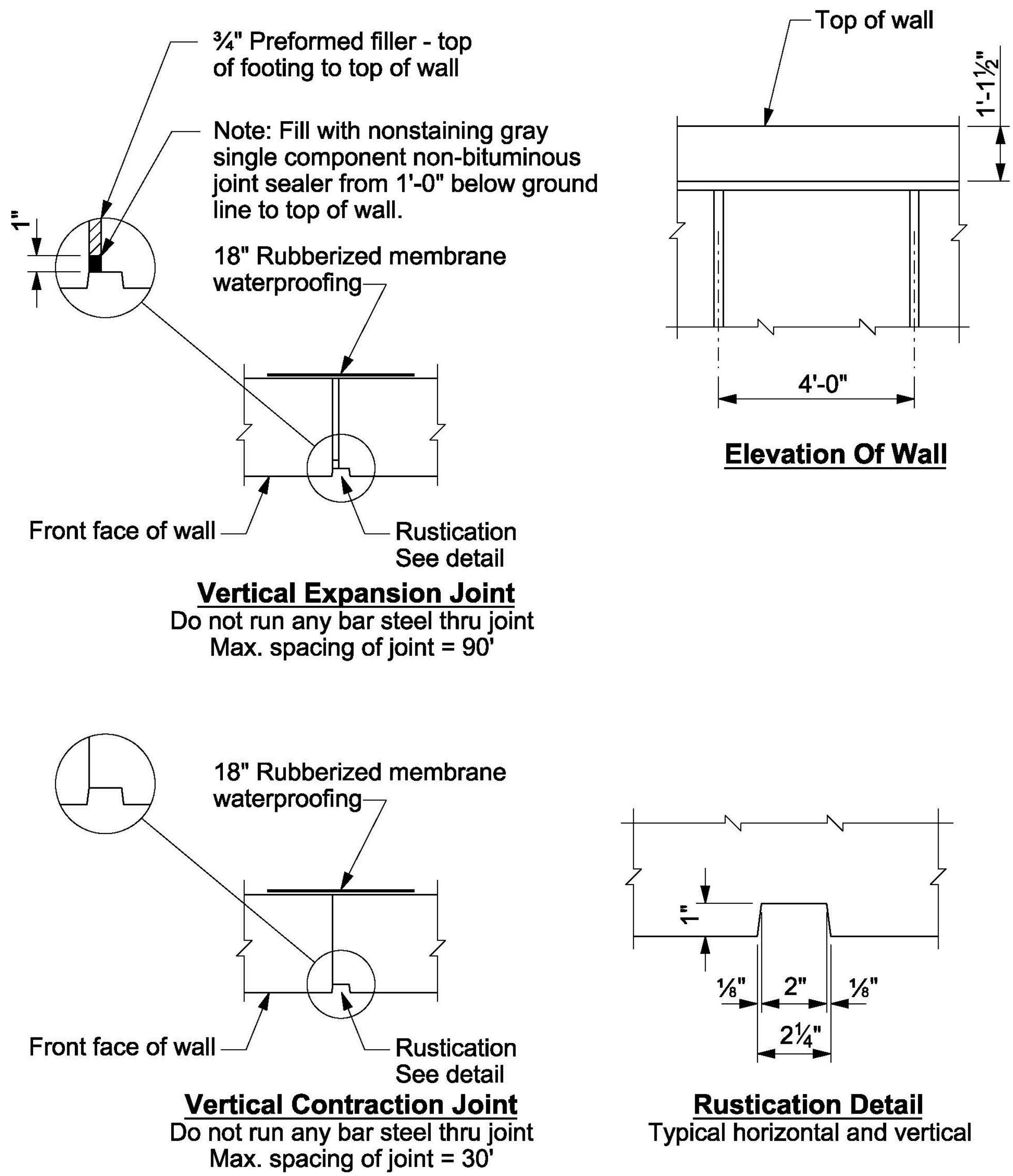


Figure 14.5-5
Retaining Wall Joint Details

2. Optional transverse construction joints are permitted in the footing, with a minimum spacing of three panel lengths. Footing joints should be offset a minimum of 1'-0 from wall joints. Run reinforcing bar steel thru footing joints.

3. The backfill material behind all cantilever walls shall be granular, free draining, non-expansive, non-corrosive material and shall be drained by weep holes with permeable material or other positive drainage systems, placed at suitable intervals and elevations. Structure backfill is placed behind the wall only to a vertical plane 18 inches beyond the face of footing. Lower limit is to the bottom of the footing.

4. If a wall is adjacent to a traveled roadway or sidewalk, use pipe underdrains in back of the wall instead of weep holes. Use a six-inch pipe wrapped underdrain located as detailed in this chapter. Provide a minimum slope of 0.5% and discharge to suitable drainage (i.e. a storm sewer system or ditch).

14.5.8 Design Tables for Cast-in-Place Concrete Cantilever Walls

Design tables suitable for use in preliminary design have been assembled and presented in this sub-section. These design tables are based on WisDOT design criteria and the material properties summarized in Table 14.5-1. Active earth pressure for the design tables was computed using the Rankine's equation for horizontal slopes and Coulomb's equation for surcharged slopes with the resultant perpendicular to the wall backface plus the wall friction angle. It was assumed that no water pressure exists. Service limit states were ignored in the analyses. The requirement of concrete is in accordance with **LRFD [5.4.2]** and 9.2. The requirement for bar steel is based on **LRFD [5.4.3]** and 9.3. The aforementioned assumptions were used in creating Table 14.5-2 thru Table 14.5-7. Refer to Figure 14.5-2 for details.

These tables should not be used if any of the assumptions or strength properties of the retained or foundation earth or the materials used for construction are different than those used in these design tables. The designer should also determine if the long-term or short-term soil strength parameters govern external stability analyses.

14.5.9 Design Examples

Refer to 14.18 for the design examples.

Design Criteria/Assumptions	Value
Concrete strength	3.5 ksi
Reinforcement yield strength	60 ksi
Concrete unit weight	150 pcf
Soil unit weight	120 pcf
Friction angle between fill and wall	21 degrees
Angle of Internal Friction (Soil - Backfill)	30 degrees

Angle of Internal Friction (Soil - Foundation)	34 degrees
Angle of Internal friction (Rock)	25 degrees
Cohesion (Soil)	0 psi
Cohesion (Rock)	20 psi
Soil Cover over Footing	4 feet
Stem Front Batter	0.25"/ft
Stem Back Batter	See Tables
Factored bearing resistance (On Soil)	**LRFD [10.6.3.1.2]**
Factored bearing resistance (On Rock)	20 ksf
Live Load Surcharge (Traffic)	240 psf
Live Load Surcharge (No Traffic)	100 psf
Lateral Earth Pressure (Horizontal Backfill)	Rankine
Lateral Earth Pressure (2:1 Backfill)	Coulomb

Table 14.5-1
Assumptions Summary for Preliminary Design of CIP Walls

HORIZONTAL BACKFILL – NO TRAFFIC – ON SOIL

H	B	A	D	Batter	Toe Steel			Heel Steel			Stem Steel		Shear	
(ft)	(ft)	(ft)	(ft)	(in/ft)	Size	Spa	L	Size	Spa	L	Size	Spa	Key	DSK
6	3'- 6"	0'- 9"	1'- 6"	0	---	---	---	---	---	---	---	---	NO	---
8	4'- 6"	1'- 0"	1'- 6"	0	---	---	---	4	12	3' - 5"	4	12	NO	---
10	5'- 3"	1'- 3"	1'- 6"	0	---	---	---	4	12	3' - 10"	4	12	NO	---
12	6'- 3"	1'- 6"	2'- 0"	0	---	---	---	4	10	4' - 7"	5	12	NO	---
14	7'- 3"	1'- 9"	2'- 0"	0	4	12	2' - 7"	5	9	5' - 6"	6	10	NO	---
16	8'- 0"	2'- 0"	2'- 0"	0.50	4	12	2' - 10"	5	8	5' - 5"	6	10	NO	---
18	8'- 9"	2'- 3"	2'- 0"	0.50	4	12	3' - 1"	7	11	6' - 7"	6	8	NO	---
20	9'- 9"	2'- 6"	2'- 0"	0.50	4	10	3' - 4"	7	8	7' - 3"	7	8	NO	---
22	10'- 6"	2'- 9"	2'- 3"	0.50	4	9	3' - 7"	9	12	9' - 2"	9	12	NO	---
24	11'- 6"	3'- 0"	2'- 9"	0.50	4	9	3' - 10"	9	11	9' - 10"	8	9	NO	---
26	12'- 0"	4'- 0"	2'- 9"	0.50	5	8	4' - 10"	8	8	8' - 5"	8	8	YES	1'- 6"
28	13'- 0"	5'- 0"	3'- 0"	0.75	7	11	6' - 6"	8	8	7' - 9"	8	7	YES	1'- 6"

Table 14.5-2
Reinforcement for Cantilever Retaining Walls

HORIZONTAL BACKFILL – TRAFFIC – ON SOIL

H	B	A	D	Batter	Toe Steel			Heel Steel			Stem Steel		Shear	
(ft)	(ft)	(ft)	(ft)	(in/ft)	Size	Spa	L	Size	Spa	L	Size	Spa	Key	DSK
6	4'- 6"	0'- 6"	1'- 6"	0	---	---	---	4	12	3' - 11"	---	---	NO	---
8	5'- 3"	0'- 9"	1'- 6"	0	---	---	---	4	11	4' - 5"	4	12	NO	---
10	6'- 6"	1'- 0"	1'- 6"	0	---	---	---	6	12	5' - 11"	4	8	NO	---
12	7'- 3"	1'- 3"	2'- 0"	0	---	---	---	6	11	6' - 5"	5	9	NO	---
14	8'- 3"	1'- 6"	2'- 0"	0	---	---	---	7	10	7' - 7"	6	9	NO	---
16	9'- 0"	2'- 3"	2'- 0"	0.50	4	12	3'- 1"	7	10	7'- 0 "	6	9	NO	---
18	9'- 3"	2'- 9"	2'- 0"	0.50	4	10	3'- 7"	7	10	6' - 7"	8	12	YES	1'- 0"
20	10'- 0"	3'- 6"	2'- 0"	0.50	5	9	4'- 4"	6	7	6'- 0 "	8	10	YES	1'- 0"
22	11'- 0"	4'- 3"	2'- 3"	0.50	5	7	5'- 1"	6	7	6' - 2"	7	7	YES	1'- 0"
24	11'- 9"	5'- 0"	2'- 6"	0.50	7	10	6'- 6"	6	7	6'- 0 "	9	11	YES	1'- 6"
26	12'- 9"	5'- 9"	2'- 9"	0.50	8	11	7'- 9"	6	7	6' - 2"	9	9	YES	1'- 6"
28	14'- 3"	7'- 0"	3'- 0"	0.75	9	11	9'- 7"	6	7	5' - 9"	9	9	YES	2'- 0"

Table 14.5-3
Reinforcement for Cantilever Retaining Walls

2:1 BACKFILL – NO TRAFFIC – ON SOIL

H	B	A	D	Batter	Toe Steel			Heel Steel			Stem Steel		Shear	
(ft)	(ft)	(ft)	(ft)	(in/ft)	Size	Spa	L	Size	Spa	L	Size	Spa	Key	DSK
6	4'- 6"	2'- 0"	1'- 6"	0	---	---	---	---	---	---	4	12	YES	1'- 0"
8	6'- 0"	2'- 6"	1'- 6"	0	4	12	3'- 4"	4	12	3' - 5"	4	9	YES	1'- 0"
10	7'- 6"	2'- 0"	1'- 6"	0	4	12	2'- 10"	6	11	5' - 11"	6	9	YES	1'- 0"
12	9'- 0"	1'- 9"	2'- 0"	0	4	12	2'- 7"	7	9	8' - 2"	8	11	YES	1'- 0"
14	10'- 6"	2'- 6"	2'- 6"	0	4	12	3'- 4"	8	10	9' - 8"	9	10	YES	1'- 6"
16	12'- 3"	3'- 9"	2'- 9"	0.50	5	12	4'- 7"	7	7	8' - 10"	9	10	YES	2'- 0"
18	14'- 0"	4'- 6"	3'- 0"	0.50	6	12	5'- 7"	9	9	11' - 2"	10	10	YES	2'- 0"
20	15'- 6"	5'- 6"	3'- 3"	0.50	7	11	7'- 0"	10	11	12' - 8"	10	8	YES	2'- 9"

Table 14.5-4
Reinforcement for Cantilever Retaining Walls

HORIZONTAL BACKFILL – NO TRAFFIC – ON ROCK

H	B	A	D	Batter	Toe Steel			Heel Steel			Stem Steel	
(ft)	(ft)	(ft)	(ft)	(in/ft)	Size	Spa	L	Size	Spa	L	Size	Spa
6	2'- 9"	0'- 9"	1'- 6"	0	---	---	---	---	---	---	4	12
8	3'- 6"	1'- 0"	1'- 6"	0	---	---	---	---	---	---	4	12
10	4'- 3"	1'- 3"	1'- 6"	0	---	---	---	4	12	2' - 10"	4	12
12	5'- 0"	1'- 6"	2'- 0"	0	4	12	2'- 4"	4	12	3' - 4"	5	12
14	5'- 9"	1'- 9"	2'- 0"	0	4	12	2'- 7"	4	12	3' - 10"	6	10
16	6'- 6"	2'- 0"	2'- 0"	0.50	4	12	2'- 10"	4	11	3' - 8"	6	10
18	7'- 3"	2'- 3"	2'- 0"	0.50	4	11	3'- 1"	5	12	4' - 3"	6	8
20	7'- 9"	2'- 6"	2'- 0"	0.50	5	11	3'- 4"	5	9	4' - 5"	8	11
22	8'- 6"	2'- 9"	2'- 0"	0.50	5	9	3'- 7"	6	10	5' - 1"	7	7
24	9'- 3"	3'- 0"	2'- 0"	0.50	6	10	4'- 1"	7	10	6'- 0 "	9	11
26	10'- 0"	3'- 3"	2'- 3"	0.50	6	9	4'- 4"	8	11	7' - 2"	10	12
28	10'- 6"	3'- 6"	2'- 6"	0.75	6	8	4'- 7"	8	11	6' - 9"	9	9

Table 14.5-5
Reinforcement for Cantilever Retaining Walls

HORIZONTAL BACKFILL – TRAFFIC – ON ROCK

H (ft)	B (ft)	A (ft)	D (ft)	Batter (in/ft)	Toe Steel Size	Toe Steel Spa	Toe Steel L	Heel Steel Size	Heel Steel Spa	Heel Steel L	Stem Steel Size	Stem Steel Spa
6	3'- 6"	0'- 9"	1'- 6"	0	---	---	---	---	---	---	4	12
8	4'- 3"	1'- 0"	1'- 6"	0	---	---	---	4	12	3' - 2"	4	12
10	5'- 0"	1'- 3"	1'- 6"	0	---	---	---	4	12	3' - 7"	4	8
12	5'- 9"	1'- 6"	2'- 0"	0	---	---	---	4	12	4' - 1"	5	9
14	6'- 6"	1'- 9"	2'- 0"	0	4	12	2'- 7"	4	8	4' - 6"	6	9
16	7'- 3"	2'- 0"	2'- 0"	0.50	4	12	2'- 10"	4	7	4' - 5"	7	12
18	8'- 0"	2'- 3"	2'- 0"	0.50	4	11	3'- 1"	6	11	5' - 4"	8	12
20	8'- 9"	2'- 6"	2'- 3"	0.50	4	9	3'- 4"	6	9	5' - 9"	8	10
22	9'- 6"	2'- 9"	2'- 6"	0.50	5	12	3'- 7"	7	11	6' - 8"	9	12
24	10'- 3"	3'- 0"	2'- 9"	0.50	5	10	3'- 10"	7	9	7' - 1"	9	11
26	11'- 0"	4'- 0"	2'- 6"	0.50	7	10	5'- 6"	8	11	7' - 5"	8	7
28	11'- 9"	4'- 3"	2'- 9"	0.75	6	7	5'- 4"	8	11	7' - 3"	8	7

Table 14.5-6
Reinforcement for Cantilever Retaining Walls

2:1 BACKFILL – NO TRAFFIC – ON ROCK

H (ft)	B (ft)	A (ft)	D (ft)	Batter (in/ft)	Toe Steel Size	Toe Steel Spa	Toe Steel L	Heel Steel Size	Heel Steel Spa	Heel Steel L	Stem Steel Size	Stem Steel Spa
6	3'- 9"	2'- 0"	1'- 6"	0	---	---	---	---	---	---	4	12
8	5'- 0"	2'- 9"	1'- 6"	0	4	12	3'- 7"	4	12	2' - 2"	4	12
10	6'- 0"	3'- 3"	1'- 6"	0	4	9	4'- 1"	4	12	2' - 7"	6	12
12	7'- 0"	4'- 0"	2'- 0"	0	5	11	4'- 10"	4	12	2' - 10"	6	9
14	8'- 3"	4'- 6"	2'- 0"	0	6	10	5'- 7"	4	12	3' - 7"	8	11
16	9'- 0"	5'- 3"	2'- 0"	0.50	8	11	7'- 3"	4	12	2' - 11"	8	11
18	10'- 0"	4'- 9"	2'- 0"	0.50	8	10	6'- 9"	6	11	4' - 10"	9	10
20	11'- 3"	4'- 0"	2'- 6"	0.50	7	10	5'- 6"	8	10	8'- 0 "	11	11
22	12'- 3"	4'- 6"	3'- 0"	0.50	7	9	6'- 0"	9	12	9' - 2"	11	9

Table 14.5-7
Reinforcement for Cantilever Retaining Walls

14.5.10 Summary of Design Requirements

1. Stability Check

    a. Strength I and Extreme Event II limit states

    - Eccentricity
    - Bearing Stress
    - Sliding

    b. Service I limit states

    - Overall Stability
    - Settlement

2. Foundation Design Parameters

    Use values provided by Geotechnical analysis

3. Concrete Design Data

    - f'c = 3500 psi
    - fy = 60,000 psi

4. Retained Soil

    - Unit weight = 120 lb/ft^3
    - Angle of internal friction - use value provided by Geotechnical analysis

5. Soil Pressure Theory

    - Coulomb theory for short heels or Rankine theory for long heels at the discretion of the designer.

6. Surcharge Load

    - Traffic live load surcharge = 2 feet = 240 lb/ft^2
    - If no traffic surcharge, use 100 lb/ft^2

7. Load Factors

Group	γ_{DC}	γ_{EV}	γ_{LSv}	γ_{LSh}	γ_{EH}	γ_{CT}	Probable use
Strength I-a	0.90	1.00	1.75	1.75	1.50		Sliding, eccentricity
Strength I-b	1.25	1.35	1.75	1.75	1.50		Bearing /wall strength
Extreme II-a	0.90	1.00	-	-	-	1.00	Sliding, eccentricity
Extreme II-b	1.25	1.35	-	-	-	1.00	Bearing
Service I	1.00	1.00	1.00	1.00	1.00		Global/settlement/wall crack control

Table 14.5-8
Load Factor Summary for CIP Walls

8. Bearing Resistance Factors

- ϕ_b = 0.55 **LRFD [Table 11.5.7-1]**

9. Sliding Resistance Factors

- ϕ_τ = 1.0 **LRFD [Table 11.5.7-1]**
- ϕ_{ep} = 0.5 **LRFD Table [10.5.5.2.2-1]**

14.6 Mechanically Stabilized Earth Retaining Walls

14.6.1 General Considerations

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

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

The MSE walls shall be designed for external stability of the wall system and internal stability of the reinforced soil mass. The global stability shall also be considered as part of design evaluation. MSE walls are proprietary wall systems and the design responsibilities with respect to global, external, and internal stability as well as settlement are shared between the designer (WisDOT or Consultant) and contractor. The designer is responsible for the overall stability, preliminary external stability and settlement whereas the contractor is responsible for the internal stability, compound stability and structural design of the wall. The responsibilities of the designer and contractor are outlined in 14.6.3.2. The design and drawings of MSE walls provided by the contractor must also be in compliance with the WisDOT special provisions as stated in 14.15.2 and 14.16

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

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

14.6.1.1 Usage Restrictions for MSE Walls

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

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

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

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

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

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

14.6.2 Structural Components

The main structural elements or components of an MSE wall are discussed below. General elements of a typical MSE wall are shown in Figure 14.6-1. These include:

- Selected Earthfill in the Reinforced Earth Zone
- Reinforcement
- Wall Facing Element
- Leveling Pad
- Wall Drainage

A combination of different wall facings and reinforcement provide a choice of selecting an MSE wall which can be used for several different functions.

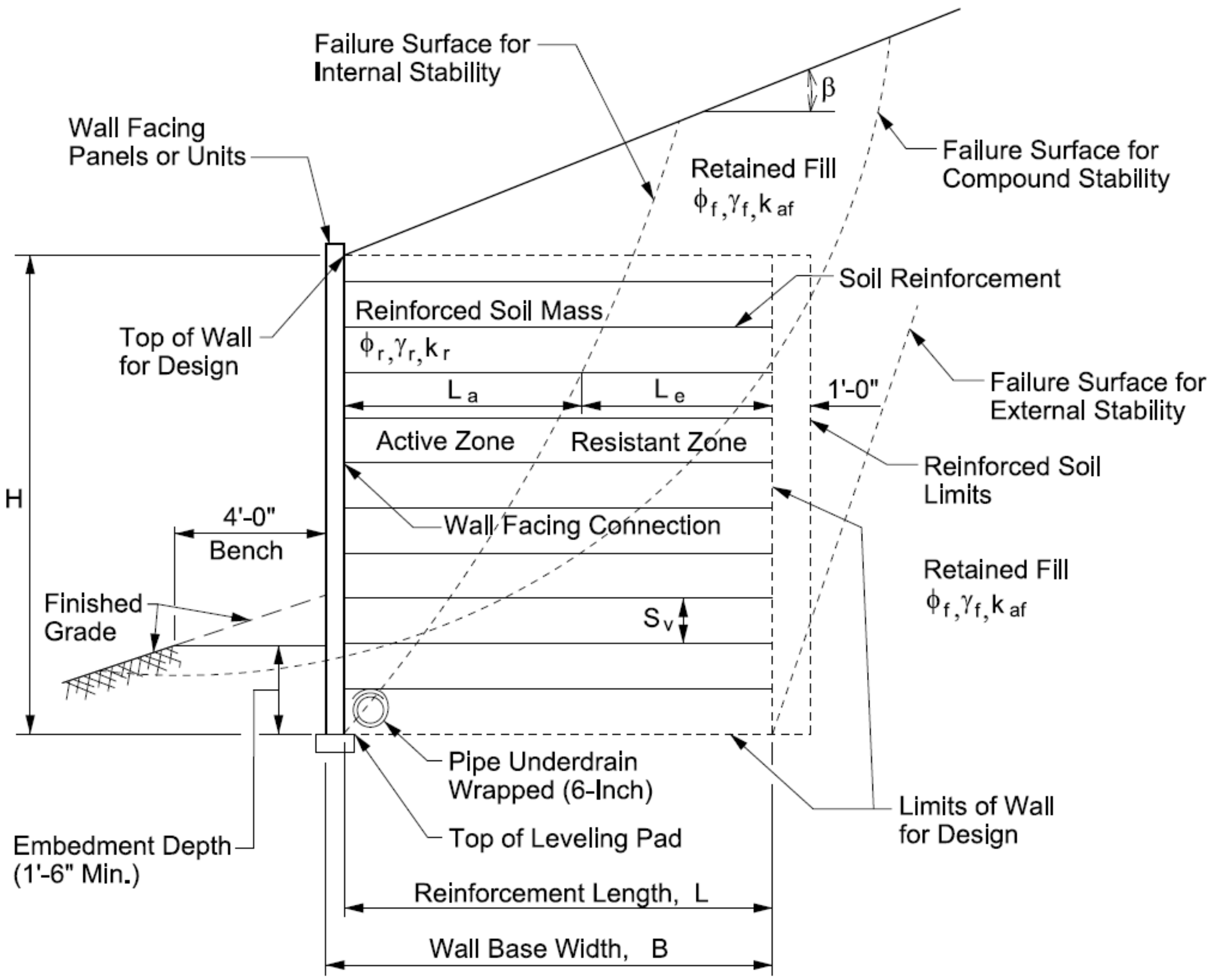


Figure 14.6-1
Structural Components of MSE Walls

14.6.2.1 Reinforced Earthfill Zone

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

Reinforcement Material	Property	Criteria
Metallic	Resistivity	> 3000 ohm-cm
Metallic	Chlorides	< 100 ppm
Metallic	Sulfates	< 200 ppm
Metallic	pH	5.0 < pH < 10.0
Geosynthetic	pH	4.5 < pH < 9.0
Metallic/Geosynthetic	Organic Content	< 1.0 %

Table 14.6-1

Electrochemical Properties of Reinforced Fill MSE Walls

An angle of internal friction of 30 degrees and unit weight of 120 pcf shall be used for the stability analyses as stated in 14.4.6. If it is desired to use an angle of internal friction greater than 30 degrees, it shall be determined by the most current wall specifications.

14.6.2.2 Reinforcement:

Soil reinforcement can be either metallic (strips or bar grids like welded wire fabric) or non-metallic including geotextile and geogrids made from polyester, polypropylene, or high density polyethylene. Metallic reinforcements are also known as inextensible reinforcement and the non-metallic as extensible. Inextensible reinforcement deforms less than the compacted soil infill used in MSE walls, whereas extensible reinforcement deforms more than compacted soil infill

The metallic or inextensible reinforcement is mild steel, and usually galvanized or epoxy coated. Three types of steel reinforcement are typically used:

<u>Steel Strips</u>: The steel strip type reinforcement is mostly used with segmental concrete facings. Commercially available strips are ribbed top and bottom, 2 to 4 inch wide and 1/8 to 5/32 inch thick.

<u>Steel grids</u>: Welded wire steel grids using two to six W7.5 to W24 longitudinal wires spaced either at 6 or 8 inches. The transverse wire may vary from W11 to W20 and are spaced from 9 to 24 inches apart.

<u>Welded wire mesh</u>: Welded wire meshes spaced at 2 by 2 inch of thinner steel wire can also be used.

The galvanized steel reinforcement that is used for soil reinforcement is oversized in cross sectional areas to account for the corrosion that occurs during the life of the structure and the resulting loss of section. The net section remaining after corrosion at the end of the design service life is used to check design requirements

The non-metallic or extensible reinforcement includes the following:

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

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

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

14.6.2.3 Facing Elements

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

Major facing types are:

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

Segmental Precast Concrete Panels

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

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

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

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

Concrete Modular Blocks Facings

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

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

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

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

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

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

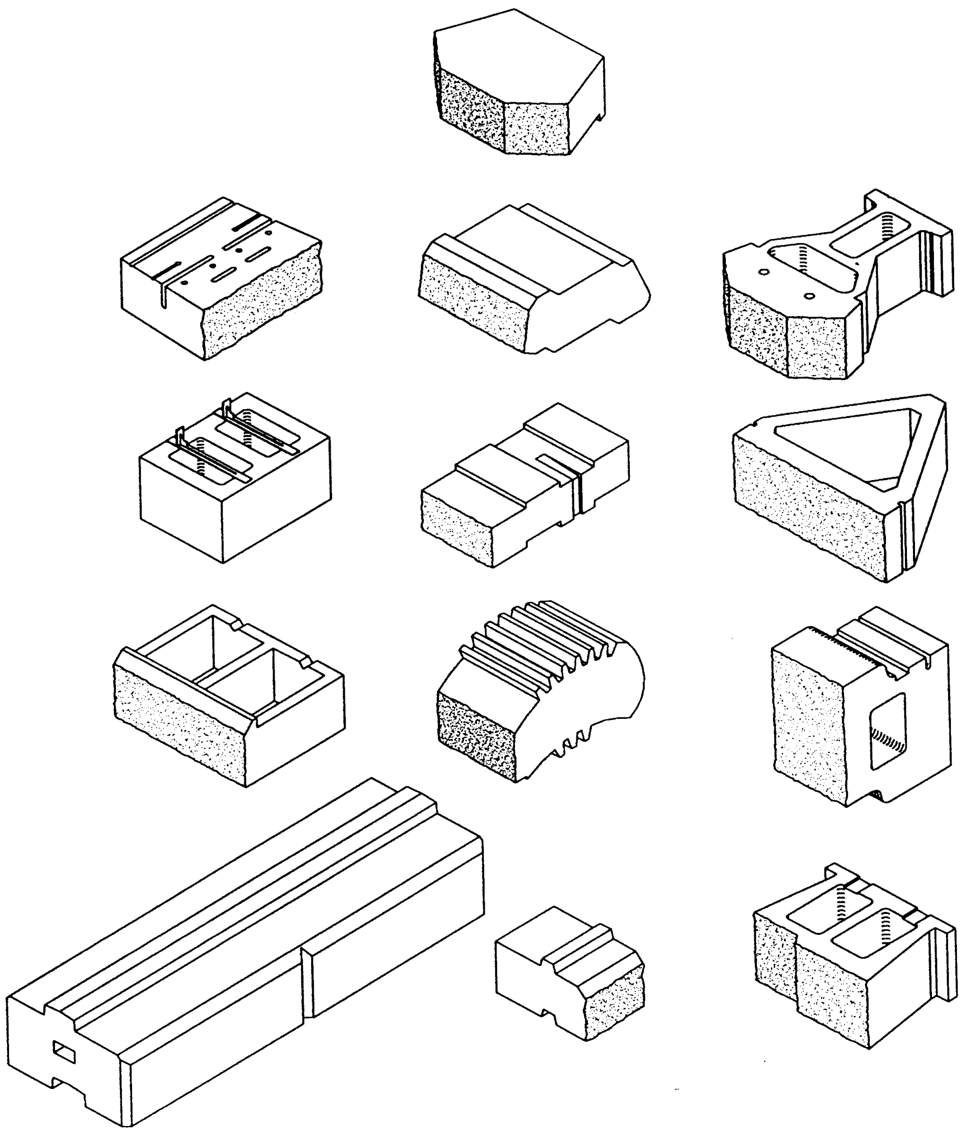
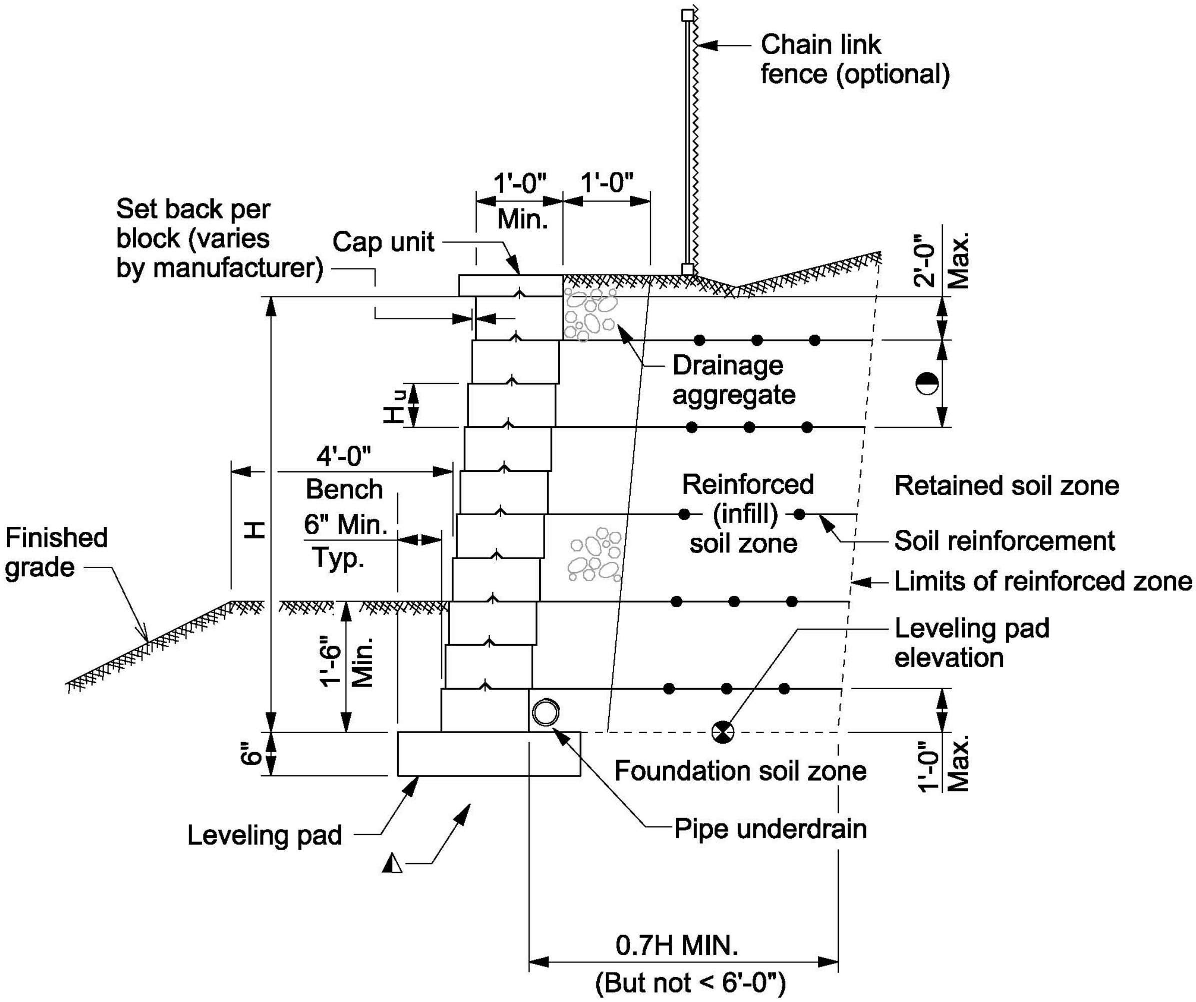


Figure 14.6-2
Modular Blocks
(Source FHWA-NHI-10-025)



Modular Block MSE Wall

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

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

Figure 14.6-3

Typical Modular Block MSE Walls

MSE Wire-Faced Facing

Welded wire fabric facing is used to build MSE wire-faced walls. These are essentially MSE walls with a welded wire fabric facing instead of a precast concrete facing. The wire size, spacing and patterns used in the facing are developed from performance data of full size wall tests and from applications in actual walls. A test to determine the connection strength between the soil reinforcement and the facing panels is required. Some systems do not use a connection because the ground reinforcement and facing panel are of one piece construction.

MSE wire-faced wall systems usually incorporate a backing mat behind the front facing. A fine metallic screen and geotextile fabric is placed behind the backing mat (or behind the facing if a backing mat is not used) to prevent the backfill from passing thru the front face.

MSE wire-faced walls can tolerate considerable differential settlement because of the flexibility of the wire facing. The limiting differential settlement is 1/50. The flexibility of the wire facing results in face bulging between ground reinforcement. The actual amount varies per system but normally is less than one inch. Recommended limits on bulging are 2" for permanent walls and 3" for temporary walls. This type of wall works well when a permanent wall facing can be placed after settlement/movement has occurred.

When MSE wire-faced walls are used for permanent wall applications, all steel components must be galvanized. When used for temporary wall applications black steel (non-galvanized) may be used since the walls are usually left in place and buried.

Temporary MSE wire-faced walls can be used as temporary shoring if site conditions permit. This wall type can also be used when staged construction is required to maintain traffic when an existing roadway is being raised and/or widened in conjunction with bridge approaches, railroad crossings or road reconstruction.

Cast-In- Place Concrete Facing

MSE walls with cast in place concrete facings are identical to MSE wire faced walls except a cast-in-place concrete facing is added after the wire face wall is erected. Modifications are made to the standard wire face wall detail to anchor the concrete facing to the wire facing and soil reinforcement. They are usually used when a special aesthetic facial treatment is required without the numerous joints that are common to precast panels. They can also be used where differential or total settlement is above tolerable limits for other wall types. A MSE wire faced wall can be constructed and allowed to settle with the concrete facing added after consolidation of the foundation soils has occurred.

The cast-in-place concrete facing shall be a minimum of 8-inches thick and contain coated or galvanized reinforcing steel. This is required because the panels and/or anchor that extend into the cast-in-place concrete are galvanized and a corrosion cell would be created if black steel contacts galvanized steel. All wire ties and bar chairs used in the cast–in-place concrete must also be coated or galvanized. Note that the 8-inch minimum wall thickness will occur at the points of maximum panel bulging and that the wall will be thicker at other locations. Also note that the 8-inch minimum is measured from the trough of any form liner or rustication.

Vertical construction joints are required in the cast-in-place concrete facing to allow for expansion and contraction and to allow for some differential settlement. Closer spacing of vertical construction joints is required when differential settlement may occur, but by delaying the placement of the cast-in-place concrete, the effects of differential settlement is minimized. Higher walls also require closer spacing of vertical construction joints if differential settlement is anticipated. Horizontal construction joints may disrupt the flow of a special aesthetic facial treatment and are sometimes not allowed for that reason. The designer should specify if optional horizontal construction joints are allowed. Cork filler is placed at vertical construction joints because cork is compressible and will allow some expansion and rotation to occur at the joint. An expandable polyvinyl chloride waterstop (PCW) is used on the back side of a vertical construction joint. Since forms are only used at the front face of the wall the PCW can be attached to a 10-inch board which is supported by the wire facing. The 8-inch minimum wall thickness may be decreased at the location of the vertical construction joint to accommodate the PCW and its support board.

Geosynthetic Facing

Geosynthetic reinforcements are looped around at the facing to form the exposed face of the MSE Wall. These faces are susceptible to ultraviolet light degradation, vandalism, and damage due to fire. Geogrid used for soil reinforcement can be looped around to form the face of the completed retaining structure in a similar manner to welded wire mesh and fabric facing. This facing is generally used in temporary applications. Similar to wire faced walls, these walls typically have a geotextile behind the geogrids, to prevent material from passing through the face.

14.6.3 Design Procedure

14.6.3.1 General Design Requirements

The procedure for design of an MSE wall requires evaluation of external stability and internal stability (structural design) at Strength Limit States and overall stability and vertical/lateral movement at Service Limit State. The Extreme Event II load combination is used to design and analyze for vehicle impact where traffic barriers are provided to protect MSE walls. The design and stability is performed in accordance with *AASHTO LRFD* and design guidance discussed in 14.4.

14.6.3.2 Design Responsibilities

MSE walls are proprietary wall systems and the structural design of the wall system is provided by the contractor. The structural design of the MSE wall system must include an analysis of internal stability (soil reinforcement pullout and stress) and local stability (facing connection forces and internal panel stresses). Additionally, the contractor should also provide internal drainage. Design drawings and calculations must be submitted to the Bureau of Structures for acceptance.

External stability, overall stability and settlement calculations are the responsibility of the WISDOT/Consultant designer. Compound stability is the responsibility of the Contractor. Soil borings and soil design parameters are provided by Geotechnical Engineer.

Although abutment loads can be supported on spread footings within the reinforced soil zone, it is WisDOT policy to support the abutment loads for multiple span structures on piles or shafts that pass through the reinforced soil zone to the in-situ soil below. Piles shall be driven prior to the placement of the reinforced earth. Strip type reinforcement can be skewed around the piles but must be connected to the wall panels and must extend to the rear of the reinforced soil zone.

For continuous welded wire fabric reinforcement, the contractor should provide details on the plans showing how to place the reinforcement around piles or any other obstacle. Abutments for single span structures may be supported by spread footings placed within the soil reinforcing zone, with WISDOT's approval. Loads from such footings must be considered for both internal wall design and external stability considerations.

14.6.3.3 Design Steps

Design steps specific to MSE walls are described in FHWA publication No*. FHWA-NHI-10-24* and modified shown below:

1. Establish project requirements including all geometry, loading conditions (transient and/or permanent), performance criteria, and construction constraints.
2. Evaluate existing topography, site subsurface conditions, in-situ soil/rock properties, and wall backfill parameters.
3. Select MSE wall using project requirement per step 1 and wall selection criteria discussed in 14.3.1.
4. Based on initial wall geometry, estimate wall embedment depth and length of reinforcement.
5. Estimate unfactored loads including earth pressure for traffic surcharge or sloping back slope and /or front slope.
6. Summarize load factors, load combinations, and resistance factors
7. Calculate factored loads for all appropriate limit states and evaluate (external stability) at Strength I Limit State
    a. sliding
    b. eccentricity
    c. bearing
8. Compute settlement at Service limit states
9. Compute overall stability at Service limit states
10. Compute vertical and lateral movement
11. Design wall surface drainage systems
12. Compute internal stability
    a. Select reinforcement
    b. Estimate critical failure surface
    c. Define unfactored loads
    d. Calculate factored horizontal stress and maximum tension at each reinforcement level
    e. Calculate factored tensile stress in each reinforcement
    f. Check factored reinforcement pullout resistance
    g. Check connection resistance requirements at facing
13. Design facing element
14. Design subsurface drainage

Steps 1-11 are completed by the designer and steps 12-14 are completed by the contractor after letting.

14.6.3.4 Initial Geometry

Figure 14.6-1 provides MSE wall elements and dimensions that should be established before making stability computations for the design of an MSE wall. The height (H) of an MSE wall is measured vertically from the top of the MSE wall to the top of the leveling pad. The length of reinforcement (L) is measured from the back of MSE wall panels. Alternately, the length of reinforcement (L1) is measured from the front face for modular block type MSE walls.

The MSE walls, with panel type facings, generally do not exceed heights of 35 feet, and with modular block type facings, should not exceed heights of 22 feet. Wall heights in excess of these limits will require approval on a case by case basis from WisDOT.

In general, a minimum reinforcement length of 0.7H or 8 feet whichever is greater shall be provided. MSE wall structures with sloping surcharge fills or other concentrated loads will generally require longer reinforcement lengths of 0.8H to 1.1H. As an exception, a minimum reinforcement length of 6.0 feet or 0.7H may be provided in accordance with **LRFD [C11.10.2.1]** provided all conditions for external and internal stability are met and smaller compaction equipment is used on a case by case basis as approved by WisDOT. MSE walls may be built to heights mentioned above; however, the external stability requirements may limit MSE wall height due to bearing capacity, settlement, or stability problems.

14.6.3.4.1 Wall Embedment

The minimum wall embedment depth to the bottom of the MSE wall reinforced backfill zone (top of the leveling pad shown in **LRFD [Figure 11.10.2-1]** and Figure 14.6-1 shall be based on external stability analysis (sliding, bearing resistance, overturning, and settlement) and the global (overall) stability requirements.

Minimum MSE wall leveling pad (and front face) embedment depths below lowest adjacent grade in front of the wall shall be in accordance with **LRFD [11.10.2.2]**, including the minimum embedment depths indicated in **LRFD [Table C11.10.2.2-1]** or 1.5 ft. whichever is greater. The embedment depth of MSE walls along streams and rivers shall be at least 2.0 ft below the potential scour elevation in accordance with **LRFD [11.10.2.2]** and the *Bridge Manual*.

WisDOT policy item:

The minimum depth of embedment of MSE walls shall be 1.5 feet

14.6.3.4.2 Wall Backslopes and Foreslopes

The wall backslopes and foreslopes shall be designed in accordance with 14.4.5.4.4. A minimum horizontal bench width of 4 ft (measured from bottom of wall horizontally to the slope face) shall be provided, whenever possible, in front of walls founded on slopes. This minimum bench width is required to protect against local instability near the toe of the wall.

14.6.3.5 External Stability

The external stability of the MSE walls shall be evaluated for sliding, limiting eccentricity, and bearing resistance at the Strength I limit state. The settlement shall be calculated at Service I limit state.

Unfactored loads and factored load shall be developed in accordance with 14.6.3.5.1. It is assumed that the reinforced mass zone acts as a rigid body and that wall facing, the reinforced soil and reinforcement act as a rigid body.

For adequate stability, the goal is to have the factored resistance greater than the factored loads. According to publication FHWA-NHI-10-024, a capacity to demand ratio (CDR) can be used to quantify the factored resistance and factored load. CDR has been used to express the safety of the wall against sliding, limiting eccentricity, and bearing resistance.

14.6.3.5.1 Unfactored and Factored Loads

Unfactored loads and moments are computed based on initial wall geometry and using procedures defined in 14.4.5.4.5. The loading diagrams for one of the 3 possible earth pressure conditions are developed. These include 1) horizontal backslope with traffic surcharge shown in Figure 14.4-2; 2) sloping backslope shown in Figure 14.4-3; and, 3) broken backslope condition as shown in Figure 14.4-4.

The computed nominal loads discussed in 14.5.4 are multiplied by applicable load factors given in Table 14.4-1. A summary of load factors and load combinations as applicable for typical MSE wall stability check is presented in Table 14.6-4. Computed factored load and moments are used for performing stability checks.

14.6.3.5.2 Sliding Stability

The stability should be computed in accordance with **LRFD [11.10.5.3]** and **LRFD [10.6.3.4]**. The sliding stability analysis shall also determine the minimum resistance along the following potential surfaces in the zones shown in **LRFD [Figure 11.10.2.1]**.

- Sliding within the reinforced backfill (performed by contractor)
- Sliding along the reinforced back-fill/base soil interface (performed by designer)

The coefficient of friction angle shall be determined as:

- For discontinuous reinforcements, such as strips – the lesser of friction angle of either reinforced backfill, ϕ_r, the foundation soil, ϕ_{fd}.
- For continuous reinforcements, such as grids and sheets – the lesser of ϕ_r or ϕ_{fd} and ρ.

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 wall could occur during the service life of the structure and lead to partial or complete loss of passive resistance. The shear strength of the facing system is also ignored.

For adequate stability, the factored resistance should be greater than the factored load for sliding,

The following equation shall be used for computing sliding:

$R_\tau = \phi\ R_n = \phi_\tau\ (V)\ (\tan\delta)$

Where:

R_R	=	Factored resistance against failure by sliding
R_n	=	Nominal sliding resistance against failure by sliding
R_τ	=	Nominal sliding resistance between soil and foundation
ϕ_τ	=	Resistance factor for shear between the soil and foundation per **LRFD [Table 11.5.7-1]**; 1.0
V	=	Factored vertical dead load
δ	=	Friction angle between foundation and soil
ρ	=	Maximum soil reinforcement interface angle **LRFD [11.10.5.3]**
$\tan\delta$	=	$\tan\phi_{fd}$ where ϕ is lesser of (ϕ_τ , ϕ_{fd} , ρ)
H_{tot}	=	Factored total horizontal load for Strength Ia
CDR	=	$R_\tau/H_{tot} \geq 1$

14.6.3.5.3 Eccentricity Check

The eccentricity check is performed in accordance with **LRFD [11.6.3.3]** and using procedure given in publication, *FHWA-NHI-10-025*

The eccentricity is computed using:

$e = B/2 - X_0$

Where:

$$X_0 = \frac{\sum M_V - M_H}{\sum V}$$

Where:

ΣM_V = Summation of Resisting moment due to vertical earth pressure

ΣM_H = Summation of Moments due to Horizontal Loads

ΣV = Summation of Vertical Loads

For eccentricity to be considered acceptable, the calculated location of the resultant vertical force (based on factored loads) should be within the middle two-thirds of the base width for soil foundations (i.e., e_{max} = B/3) and middle nine-tenths of the base width for rock foundations (i.e., e_{max} = 0.45B). Therefore, for each load group, e must be less than e_{max}. If e is greater than e_{max}, a longer length of reinforcement is required. The CDR for eccentricity should be greater than 1.

CDR = $e_{max}/e > 1$

14.6.3.5.4 Bearing Resistance

The bearing resistance check shall be performed in accordance with **LRFD [11.10.5.4]**. Provisions of **LRFD [10.6.3.1]** and **LRFD [10.6.3.2]** shall apply**.** Because of the flexibility of MSE walls, an equivalent uniform base pressure shall be assumed. Effect of live load surcharge shall be added, where applicable, because it increases the load on the foundation. Vertical stress, $\sigma_{v,}$ shall be computed using following equation.

The bearing resistance computation requires:

Base Pressure $(\sigma_v) = \dfrac{\sum V}{B - 2e}$

σ_v = Vertical pressure

ΣV = Sum of all vertical forces

B = Reinforcement length

e = Eccentricity = B/2 – X_0

X_0 = $(\Sigma M_R - \Sigma M_H)/\Sigma V$

ΣM_V = Total resisting moments

ΣM_H = Total driving moments

The nominal bearing resistance, q_n, shall be computed using methods for spread footings. The appropriate value for the resistance factor shall be selected from **LRFD [Table 11.5.7-1]**.

The computed vertical stress, σ_v, shall be compared with factored bearing resistance, q_r in accordance with the **LRFD [11.10.5.4]** and a Capacity Demand Ratio, CDR, shall be calculated using the following equation:

$$q_r = \phi_b \, q_n \geq \sigma_v$$

Where:

q_r = Factored bearing resistance

q_n = Nominal bearing resistance computed using **LRFD [10.6.3.1.2a-1]**

ϕ_b = 0.65 using **LRFD [Table 11.5.7-1]**

CDR = $q_r/\sigma_v > 1.0$

14.6.3.6 Vertical and Lateral Movement

Excessive MSE wall foundation settlement can result in damage to the wall facing, coping, traffic barrier, bridge superstructure, bridge end panel, pavement, and/or other settlement-sensitive elements supported on or near the wall.

Techniques to reduce damage from post-construction settlements and deformations may include full-height vertical sliding joints through the rigid wall facing elements and appurtenances, and/or ground improvement or reinforcement techniques. Staged preload/surcharge construction using onsite materials or imported fills may also be used.

Settlement shall be computed using the procedures outlined in 14.4.7.2 and the allowable limit settlement guidelines in 14.4.7.2.1 and in accordance with **LRFD [11.10.4]** and **LRFD [10.6.2.4].** Differential settlement from the front face to the back of the wall shall be evaluated, as appropriate.

For MSE walls with rigid facing concrete panels, slip joints of 0.75 inch width can be provided to control differential settlement as per **LRFD [Table C11.10.4-1]**.

14.6.3.7 Overall Stability

Overall Stability shall be performed in accordance with **LRFD [11.10.4.3]**. Provision of **LRFD [11.6.2.3]** shall also apply. Overall and compound stability of complex MSE wall system shall also be investigated, especially where the wall is located on sloping or soft ground where overall stability may be inadequate. Compound external stability is the responsibility of the contractor/wall supplier. The long term strength of each backfill reinforcement layer intersected by the failure surface should be considered as restoring forces in the limit equilibrium slope stability analysis. Figure 14.6-4 shows failure surfaces generated during overall or compound stability evaluation.

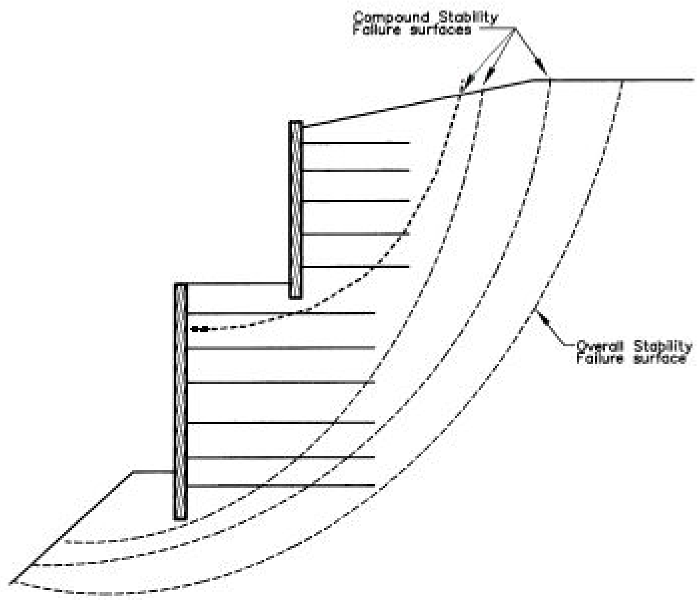


Figure 11.10.4.3-1 Overall and Compound Stability of Complex MSE Wall Systems.

Figure 14.6-4
MSE Walls Overall and Compound Stability
(Source *AASHTO LRFD*)

14.6.3.8 Internal Stability

Internal stability of MSE walls shall be performed by the wall contractor/supplier. The internal stability (safety against structural failure) shall be performed in accordance with **LRFD [11.10.6]** and shall be evaluated with respect to following at the Strength Limit:

- Tensile resistance of reinforcement to prevent breakage of reinforcement
- Pullout resistance of reinforcement to prevent failure by pullout
- Structural resistance of face elements and face elements connections

14.6.3.8.1 Loading

Figure 14.4-11 shows internal failure mechanism of MSE walls due to tensile and pullout failure of the soil reinforcement. The maximum factored tension load (T_{max}) due to tensile and pullout reinforcement shall be computed at each reinforcement level using the *Simplified Method* approach in accordance with **LRFD [11.10.6.2]**. Factored load applied to the reinforcement-facing connection (T_0) shall be equal to maximum factored tension reinforcement load (T_{max}) in accordance with **LRFD [11.10.6.2.2].**

14.6.3.8.2 Reinforcement Selection Criteria

At each reinforcement level, the reinforcement must be sized and spaced to preclude rupture under the stress it is required to carry and to prevent pullout for the soil mass. The process of sizing and designing the reinforcement consists of determining the maximum developed tension loads, their location, along a locus of maximum stress and the resistance provided by reinforcement in pullout capacity and tensile strength.

Soil reinforcements are either extensible or inextensible as discussed in 14.6.2.2.

When inextensible reinforcements are used, the soil deforms more than the reinforcement. The critical failure surface for this reinforcement type is determined by dividing the zone into active and resistant zones with a bilinear failure surface as shown in part (a) of Figure 14.6-5.

When extensible reinforcements are used, the reinforcement deforms more than soil and it is assumed that shear strength is fully mobilized and active earth pressure developed. The critical failure surface for both horizontal and sloping backfill conditions are represented as shown in lower part (b) of Figure 14.6-5.

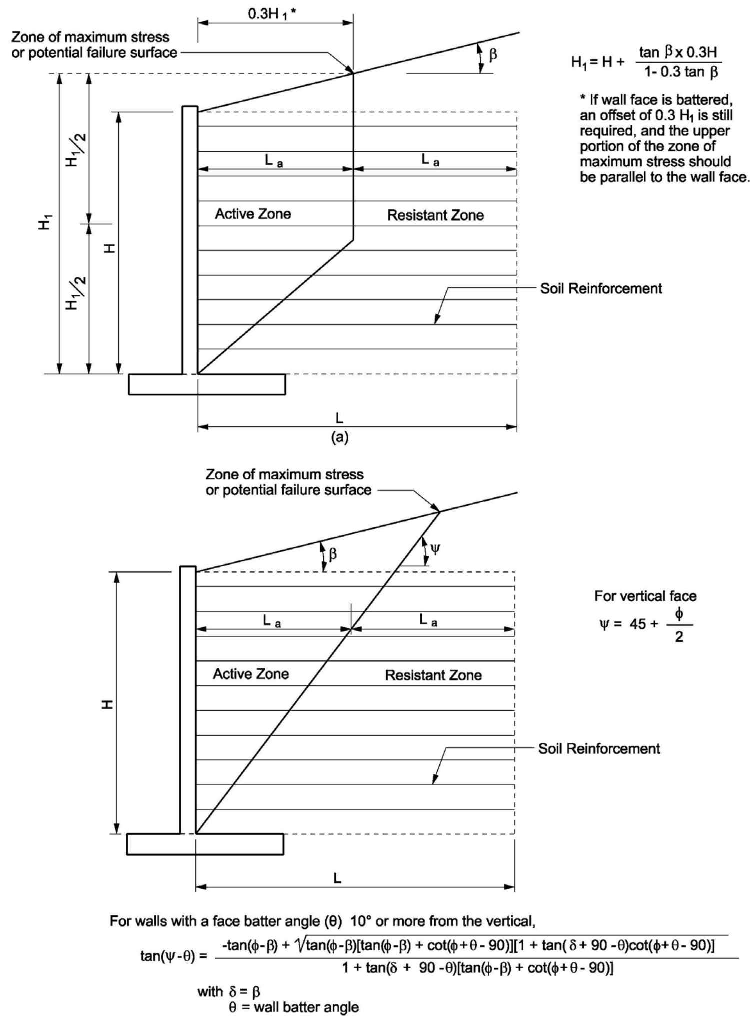


Figure 14.6-5
Location of Potential Failure Surface for Internal Stability of MSE Walls
(Source *AASHTO LRFD*)

14.6.3.8.3 Factored Horizontal Stress

The *Simplified Method* is used to compute maximum horizontal stress and is computed using the equation

$$\sigma_H = \gamma_P \left(\sigma_v k_r + \Delta\sigma_H\right)$$

Where:

γ_P = Maximum load factor for vertical stress (EV)

k_r = Lateral earth pressure coefficient computed using k_r/k_a

σ_V = Pressure due to reinforce soil mass and any surcharge loads above it

$\Delta\sigma_H$ = Horizontal stress at reinforcement level resulting in a concentrated horizontal surcharge load

Research studies have indicated that the maximum tensile force is primarily related to the type of reinforcement in the MSE mass, which, in turn, is a function of the modulus extensibility, and density of reinforcement. Based on this research, a relationship between the type of reinforcement and the overburden stress has been developed and is shown in Figure 14.6-6.

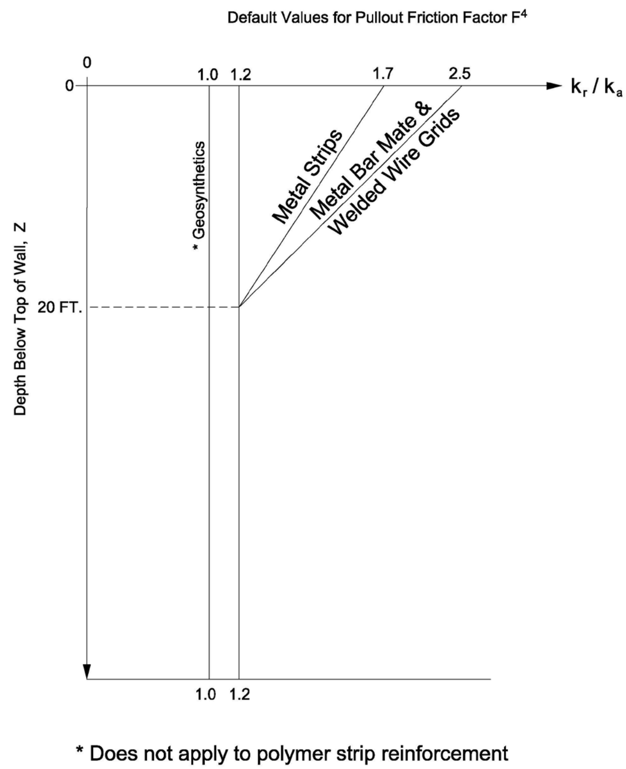


Figure 14.6-6
Variation of the Coefficient of Lateral Stress Ratio with Depth
(Source *AASHTO LRFD*)

Lateral stress ratio k_r/k_a, can be used to compute k_r at each reinforcement level. For vertical face batter <10^0, K_a is obtained using Rankine theory. For wall face with batter greater than 10^0 degrees, Coulomb's formula is used. If present, surcharge load should be added into the estimation of σ_V. For the simplified method, vertical stress for the maximum reinforcement load calculations are shown in Figure 14.6-7 .

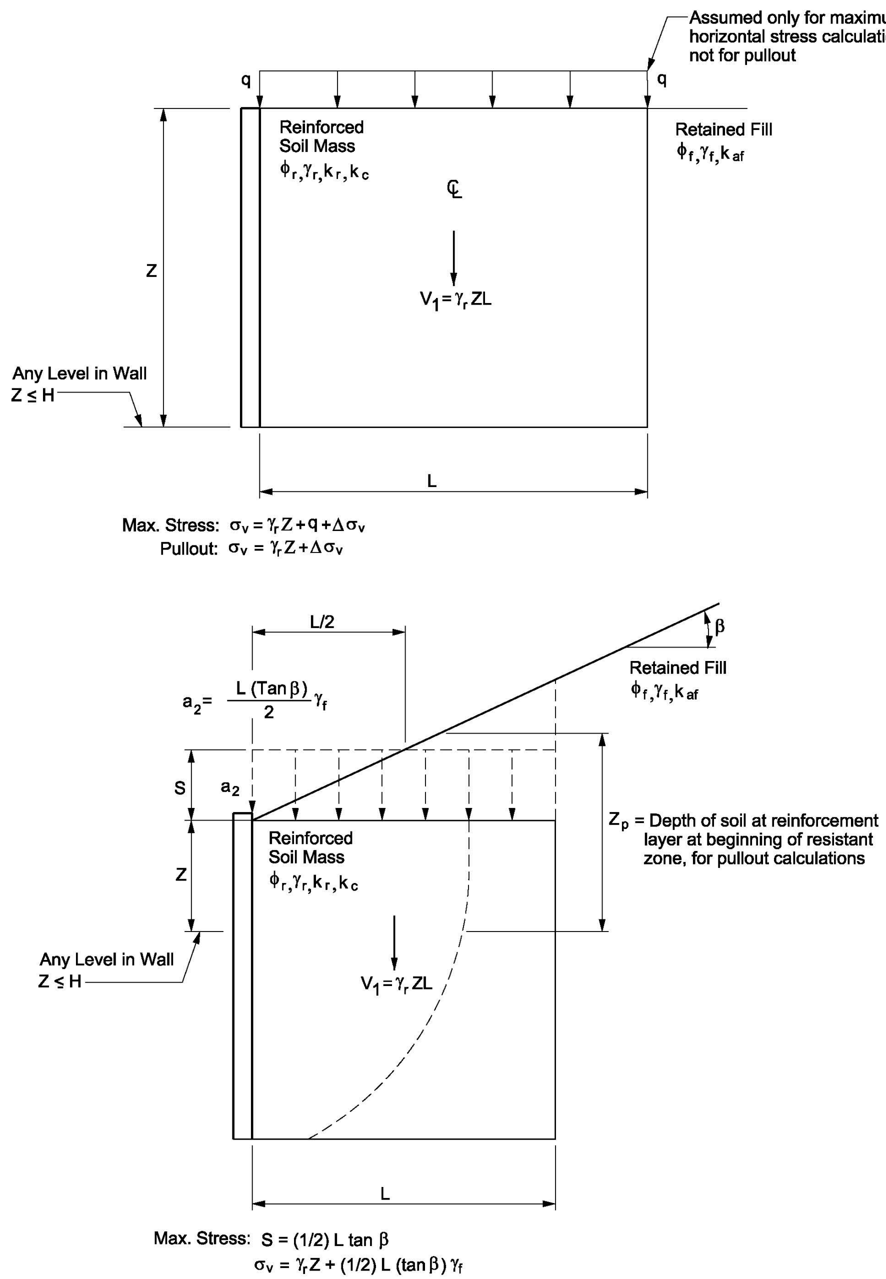


Figure 14.6-7
Calculation of Vertical Stress for Horizontal and Sloping Backslope for Internal Stability
(Source *AASHTO LRFD*)

14.6.3.8.4 Maximum Factored Tension Force

The maximum tension load also referred as maximum factored tension force is applied to the reinforcements layer per unit width of wall (T_{max}) will be based on the reinforcement vertical spacing (S_V) as under:

$T_{max} = \sigma_H S_V$

Where:

T_{max}	=	Maximum tension load
σ_H	=	Factored horizontal load defined in 14.6.3.8.3

$T_{max\text{-}UWR}$ may also be computed at each level for discrete reinforcements (metal strips, bar mats, grids, etc) per a defined unit width of reinforcement

$T_{max\text{-}UWR}$	=	$(\sigma_H S_V)/R_C$
R_C	=	Reinforcement coverage ratio **LRFD [11.10.6.4.1]**

14.6.3.8.5 Reinforcement Pullout Resistance

MSE wall reinforcement pullout capacity is calculated in accordance with **LRFD [11.10.6.3]**. The potential failure surface for inextensible and extensible wall system and the active and resistant zones are shown in Figure 14.6-5. The pullout resistance length, L_e, shall be determined using the following equation

$$\phi L_e = \frac{T_{max}}{\left(F^* \cdot \alpha \cdot \sigma'_v \cdot C \cdot R_c\right)}$$

Where:

L_e	=	Length of reinforcement in the resistance zone
T_{max}	=	Maximum tension load
ϕ	=	Resistance factor for reinforcement pullout
F*	=	Pullout friction factor, Figure 14.6-8
α	=	Scale correction factor
σ'_V	=	Unfactored effective vertical stress at the reinforcement level in the resistance zone
C	=	2 for strip, grid, and sheet type reinforcement

R_c = Reinforcement coverage ratio **LRFD [11.10.6.4.1]**

The correction factor, α, depends primarily upon the strain softening of compacted granular material, and the extensibility, and the length of the reinforcement. Typical value is given in Table 14.6-2.

Reinforcement Type	α
All steel reinforcement	1.0
Geogrids	0.8
Geotextiles	0.6

Table 14.6-2
Typical values of α
(Source **LRFD [Table 11.10.6.3.2-1]**)

The pullout friction factor, F*, can be obtained accurately from laboratory pullout tests performed with specific material to be used on the project. Alternating, lower bound default values can be used from the laboratory or field pull out test performed in the specific back fill to be used on the project.

As shown in Figure 14.6-5, the total length of reinforcement (L) required for the internal stability is computed as below

$$L = L_e + L_a$$

Where:

L_e = Length of reinforcement in the resistance zone

L_a = Remainder length of reinforcement

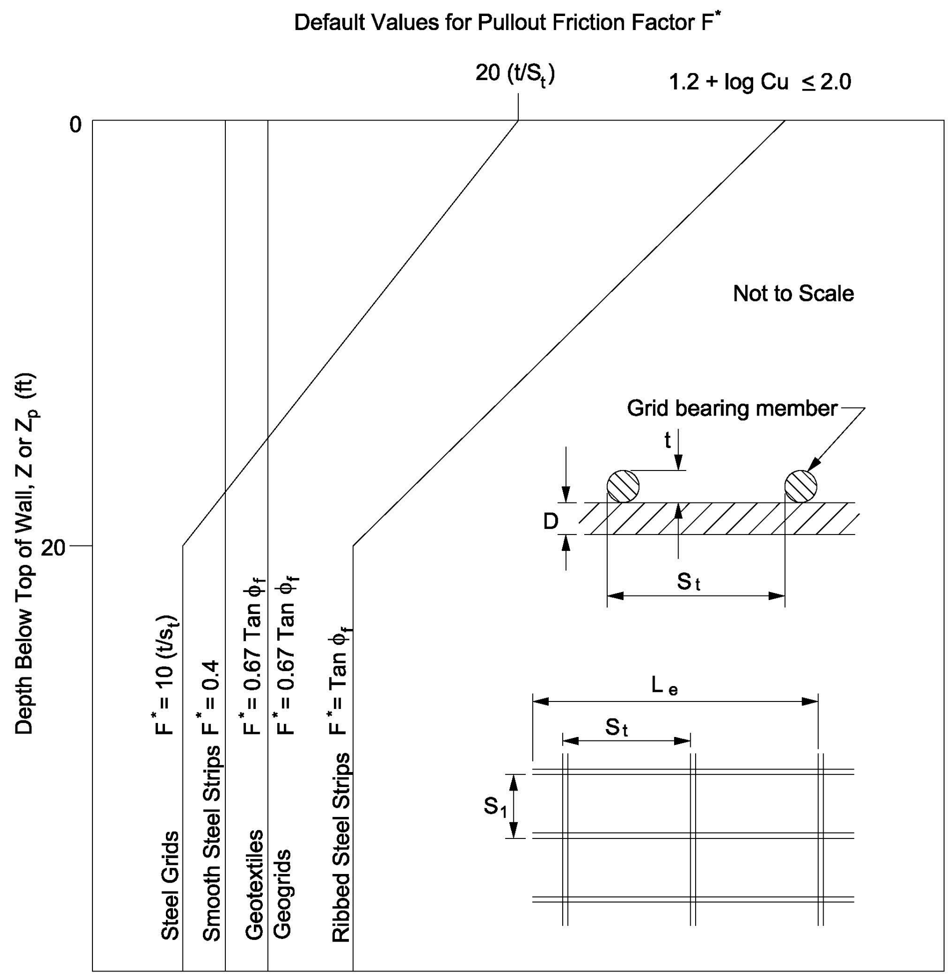


Figure 14.6-8
Default Values of F*
(Source: **LRFD [Figure 11.10.6.3.2-2]**)

14.6.3.8.6 Reinforced Design Strength

The maximum factored tensile stress (T_{MAX}) in each reinforcement layer as determined in 14.6.3.8.4 is compared to the long term reinforcement design strength computed in accordance with **LRFD [11.10.6.4.1]** as:

$$T_{MAX} \leq \phi \, T_{al} \, R_C$$

Where

ϕ = Resistance factor for tensile resistance

R_c = Reinforcement coverage ratio

T_{al} = Nominal tensile resistance (reinforcement design strength) at each reinforcement level

The value for T_{MAX} is calculated with a load factor of 1.35 for vertical earth pressure, EV. The tensile resistance factor for metallic and geosynthetic reinforcement is based on the following:

Metallic Reinforcement	Strip Reinforcement • Static Loading Grid Reinforcement • Static Loading	 0.75 0.65
Geosynthetic reinforcement	• Static Loading	0.90

Table 14.6-3
Resistance Factor for Tensile and Pullout Resistance
(Source **LRFD [Table 11.5.7-1]**)

14.6.3.8.7 Calculate T_{al} for Inextensible Reinforcements

T_{al} for inextensible reinforcements is computed as below:

$T_{al} = (A_c F_y)/b$

Where:

F_y = Minimum yield strength of steel

b = Unit width of sheet grid, bar, or mat

A_c = Design cross sectional area corrected for corrosion loss

14.6.3.8.8 Calculate T_{al} for Extensible Reinforcements

The available long-term strength, T_{al}, for extensible reinforcements is computed as:

$$T_{al} = \frac{T_{ult}}{RF} = \frac{Tult}{RF_{ID} * RF_{CR} * RF_D}$$

Where:

T_{ult}	=	Minimum average roll value ultimate tensile strength
RF	=	Combined strength reduction factor to account for potential long term degradation due to installation, damage, creep, and chemical aging
RF_{ID}	=	Strength Reduction Factor related to installation damage
RF_{CR}	=	Strength Reduction Factor caused by creep due to long term tensile load
RF_D	=	Strength Reduction Factor due to chemical and biological degradation

RF shall be determined from product specific results as specified in **LRFD [11.10.6.4.3b]**.

14.6.3.8.9 Design Life of Reinforcements

Long term durability of the steel and geosynthetic reinforcement shall be considered in MSE wall design to ensure suitable performance throughout the design life of the structure.

The steel reinforcement shall be designed to achieve a minimum designed life in accordance with **LRFD [11.5.1]** and shall follow the provision of **LRFD [11.10.6.4.2]**. The provision for corrosion loss shall be considered in accordance with the guidance presented in **LRFD [11.10.6.4.2a]**.

The durability of polymeric reinforcement is influenced by time, temperature, mechanical damage, stress levels, and changes in molecular structure. The strength reduction for geosynthetic reinforcement shall be considered in accordance with **LRFD [11.10.6.4.2b]**.

14.6.3.8.10 Reinforcement /Facing Connection Design Strength

Connections shall be designed to resist stresses resulting from active forces as well as from differential movement between the reinforced backfill and the wall facing elements in accordance with **LRFD [11.10.6.4.4]**.

<u>Steel Reinforcement</u>

Capacity of the connection shall be tested per **LRFD [5.10.8.3].** Elements of the connection which are embedded in facing elements shall be designed with adequate bond length and bearing area in the concrete, to resist the connection forces. The steel reinforcement connection strength requirement shall be designed in accordance with **LRFD [11.10.6.4.4a]**.

Connections between steel reinforcement and the wall facing units (e.g. bolts and pins) shall be designed in accordance with **LRFD [6.13]**. Connection material shall also be designed to accommodate loss due to corrosion.

<u>Geosynthetic Reinforcement</u>

The portion of the connection embedded in the concrete facing shall be designed in accordance with **LRFD [5.10.8.3]**. The nominal geosynthetic connection strength requirement shall be designed in accordance with **LRFD [11.10.6.4.4b]**.

14.6.3.8.11 Design of Facing Elements

Precast Concrete Panel facing elements are designed to resist the horizontal forces developed internally within the wall. Reinforcement is provided to resist the average loading conditions at each depth in accordance with structural design requirements in *AASHTO LRFD*. The embedment of the reinforcement to panel connector must be developed by test, to ensure that it can resist the maximum tension. The concrete panel must meet temperature and shrinkage steel requirements. Epoxy protection of panel reinforcement is required.

Modular Block Facing elements must be designed to have sufficient inter-unit shear capacity. The maximum spacing between unit reinforcement should be limited to twice the front block width or 2.7 feet, whichever is less. The maximum depth of facing below the bottom reinforcement layer should be limited to the block width of modular facing unit. The top row of reinforcement should be limited to 1.5 times the block width. The factored inter-unit shear capacity as obtained by testing at the appropriate normal load should exceed the factored horizontal earth pressure.

14.6.3.8.12 Corrosion

Corrosion protection is required for all permanent and temporary walls in aggressive environments as defined in **LRFD [11.10.2.3.3].** Aggressive environments in Wisconsin are typically associated with salt spray and areas near storm water pipes in urban areas. MSE walls with steel reinforcement should be protected with a properly designed impervious membrane layer below the pavement and above the first level of the backfill reinforcement. The details of the impervious layer drainage collector pipe can be found in *FHWA-NHI-0043* (FHWA 2001).

14.6.3.9 Wall Internal Drainage

The wall internal drainage should be designed using the guidelines provided in 14.4.7.6. Pipe underdrain must be provided to properly drain MSE walls. Chimney or blanket drains with collector-pipe drains are installed as part of the MSE walls sub-drainage system. Collector pipes with solid pipes are required to carry the discharge away from the wall. All collector pipes and solid pipes should be 6-inch diameter.

14.6.3.10 Traffic Barrier

Design concrete traffic barriers on MSE walls to distribute applied traffic loads in accordance with **LRFD [11.10.10.2]** and WisDOT standard details. Traffic impact loads shall not be transmitted to the MSE wall facing. Additionally, MSE walls shall be isolated from the traffic barrier load. Traffic barrier shall be self-supporting and not rely on the wall facing.

14.6.3.11 Design Example

Example E-2 shows a segmental precast panel MSE wall with steel reinforcement. Example E-3 shows a segmental precast panel MSE wall with geogrid reinforcement. Both design examples include external and internal stability of the walls. The design examples are included in 14.18.

14.6.3.12 Summary of Design Requirements

1. Strength Limit Checks

   a. External Stability

      - Sliding

$$CDR = \left(\frac{R_{\tau}}{H_{tot}} \right) > 1.0$$

      - Eccentricity Check

$$CDR = \left(\frac{e_{\max}}{e} \right) > 1.0$$

      - Bearing Resistance

$$CDR = \left(\frac{q_r}{\sigma_v} \right) > 1.0$$

   b. Internal stability

      - Tensile Resistance of Reinforcement
      - Pullout Resistance of Reinforcement
      - Structural resistance of face elements and face elements connections

   c. Service Limit Checks

      - Overall Stability
      - Wall Settlement and Lateral Deformation

2. Concrete Panel Facings

   - f'_c = 4000 psi (wet cast concrete)
   - Min. thickness = 5.5 inches
   - Min. reinforcement = 1/8 square inch per foot in each direction (uncoated)

- Min. concrete cover = 1.5 inches
- fy = 60,000 psi

3. Traffic/ Surcharge
   - Traffic live load surcharge = 240 lb/ft^2 or
   - Non traffic live load surcharge =100 lb/ft^2
4. Reinforced Earthfill
   - Unit weight = 120 lb/ft^3
   - Angle of internal friction = 30°, or as determined from Geotechnical analyses (maximum allowed is 36°)
5. Retained Soil
   - Unit weight = 120 lb/ft^3
   - Angle of internal friction = 30°, or as determined from Geotechnical analyses
6. Design Life
   - 75 year minimum for permanent walls
7. Soil Pressure Theory
   - Coulomb's Theory
8. Soil Reinforcement

   For steel or geogrid systems, the minimum soil reinforcement length shall be 70 percent of the wall height and not less than 8 feet. The length of soil reinforcement shall be equal from top to bottom. Soil reinforcement must extend a minimum of 3 feet beyond the failure plane.

9. Summary of Load Combinations and Load Factors

Group	γ_{DC}	γ_{EV}	γ_{LSv}	γ_{LSh}	γ_{EH}	γ_{CT}	Probable use
Strength Ia	0.90	1.00	0.0	1.75	1.50		Sliding, eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50		Bearing, wall strength
Extreme IIa	0.90	1.00	-	-	1.00	1.00	Sliding, eccentricity
Extreme IIb	1.25	1.35	-	-	1.00	1.00	Bearing
Service I	1.00	1.00	1.00	1.00	1.00	-	Global, settlement, wall crack control

<u>Table 14.6-4</u>
Load Factor Summary for MSE-External Stability

10. Resistance Factors for External Stability

Stability mode	Condition	Resistance Factor
Sliding		1.00
Bearing		0.65
Overall stability	Geotechnical parameters are well defined and slope does not support a structural element	0.75
	Geotechnical parameters are based on limited information, or the slope supports a structural element	0.65

<u>Table 14.6-5</u>
Resistance Factor Summary for MSE-External Stability
(Source **LRFD [Table 11.5.7-1]**)

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. These walls can be formed to a tight radius of curvature of 50 ft. or greater. 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

ϕ_τ = Resistance factor for shear between soil and foundation per **LRFD [Table 10.5.5.2.2-1]**

ϕ_τ = 0.9 for concrete on sand and 1.0 for soil on soil

R_τ = 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 wall could occur during the service life of the structure and lead to partial or complete loss of passive resistance.

Interface sliding resistance between concrete blocks shall be calculated using the corrected wall weight based on the calculated hinge height in accordance with **LRFD [Figure 11.10.6.4.4b-1]**. Interface friction resistance parameters shall be based on NCMA method. Shear between the blocks must be resisted by friction, keys or pins.

14.7.1.2.3 Bearing Resistance

The bearing resistance of the walls shall be computed in accordance with **LRFD [10.6.3.1]**.

Base Pressure, $\sigma_v = \dfrac{\sum V_{tot}}{(B - 2e)}$

The computed vertical stress shall be compared with factored bearing resistance in accordance with the **LRFD [10.6.3.1]**, using following equation:

$q_r = \phi_b q_n \geq \sigma_v$

Where:

q_n	=	Nominal bearing resistance **LRFD [Equation 10.6.3.1.2a-1]**
$\sum V$	=	Summation of Vertical loads
B	=	Base width
e	=	Eccentricity
ϕ_b	=	0.55 **LRFD [Table 11.5.7-1]**

14.7.1.2.4 Eccentricity Check

The eccentricity check shall be performed in accordance with **LRFD [11.6.3.3].** The location of the resultant force should be within the middle two-thirds of the base width (e<B/3) for footings on soil, and within nine-tenths of the base (e<0.45B) for footings on rock.

14.7.1.3 Settlement

The vertical and lateral displacements of prefabricated modular retaining walls must be evaluated for all applicable dead and live load combinations at Service I limit states using procedures described in 14.4.7.2 and compared with tolerable movement criteria presented in 14.4.7.2.1. In general, lateral movements of walls on shallow foundations can be estimated assuming the wall rotates or translates as a rigid body due to the effects of earth loads and differential settlements along the base of the wall.

14.7.1.4 Overall Stability

The overall (global) stability shall be evaluated in accordance with **LRFD [11.6.2.3]** and in accordance with 14.4.7.3, with the exception that the entire mass of the modular walls (or the "foundation load"), may be assumed to contribute to the overall stability of the slope. The overall stability check shall be performed by the Geotechnical Engineering Unit or Consultant of record.

14.7.1.5 Summary of Design Requirements

1. Stability Evaluations
   - External Stability
     - Eccentricity Check
     - Bearing Check
     - Sliding
   - Settlement
   - Overall/Global
2. Block Data
   - One piece block
   - Minimum thickness of front face = 4 inches
   - Minimum thickness of internal cavity walls other than front face = 2 inches
   - 28 day concrete strength = 5000 psi
   - Maximum water absorption rate by weight = 5%
3. Traffic Surcharge
   - Traffic live load surcharge = 240 lb/ft^2
   - If no traffic live load is present, use 100 lb/ft^2 live load for construction equipment
4. Retained Soil
   - Unit weight γ_f = 120 lb/ft^3
   - Angle of internal friction as determined by Geotechnical Engineer

5. Soil Pressure Theory

    - Use Coulomb Theory

6. Maximum Height = 8 ft.

    (This height is measured from top of leveling pad to bottom of cap. It is not the exposed height). In addition this maximum height may be reduced if there is sloping backfill or a sloping surface in front of the wall.)

7. Load Factors

Group	γ_{DC}	γ_{EV}	γ_{LSv}	γ_{LSh}	γ_{EH}	γ_{CT}	Probable use
Strength Ia	0.90	1.00	0.0	1.75	1.50	-	Sliding, eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50	-	Bearing /wall strength
Service I	1.00	1.00	1.00	1.00	1.00	-	Global/settlement/wall crack control

Table 14.7-1
Load Factor Summary for Prefabricated Modular Walls

8. Sliding Resistance Factors

    ϕ_τ = 1.0 **LRFD [Table 11.5.7-1]**

9. Bearing Resistance Factors

    ϕ_b = 0.55 **LRFD [Table 11.5.7-1]**

14.8 Prefabricated Modular Walls

Prefabricated modular walls systems use interconnected structural elements, which use selected in-fill soil or rock fill to resist external pressures by acting as gravity retaining walls. Metal and precast concrete or metal bin walls, crib walls, and gabion walls are considered under the category of prefabricated modular walls. These walls consist of modular elements which are proprietary. The design of these wall systems is provided by the contractor/wall supplier.

Prefabricated modular walls can be used where reinforced concrete walls are considered. Steel modular systems should not be used where aggressive environmental condition including the use of deicing salts or other similar chemicals are used that may corrode steel members and shorten the life of modular wall systems.

14.8.1 Metal and Precast Bin Walls

Metal bin walls generally consist of sturdy, lightweight, modular steel members called as stringers and spacers. The stringers constitute the front and back face of the bin and spacers its sides. The wall is erected by bolting the steel members together. The flexibility of the steel structure allows the wall to flex against minor ground movement. Metal bin walls are subject to corrosion damage from exposure to water, seepage and deicing salts. To improve the service life of metal bin walls, consideration should be given towards increasing the galvanizing requirements and establishing electrochemical requirements for the confined backfill.

Precast concrete bin walls are typically rectangular interlocking prefabricated concrete modules. A common concrete module typically has a face height varying from 4 to 5 feet, a face length up to 8 feet, and a width ranging from 4 to 20 feet. The wall can be assembled vertically or provided with a batter. A variety of surface treatment can be provided to meet aesthetic requirements. A parapet wall can be provided at the top of the wall and held rigidly by a cast in place concrete slab. A reinforced cast-in-place or precast concrete footing is usually placed at the toe and heel of the wall.

Bin walls are not recommended for applications that require a radius of curvature less than 800 ft. The wall face batter shall not be steeper than 10° or 6:1 (V:H). The base width of bin walls is generally 60% of the wall height. Further description and method of construction can be found in FHWA's publication *Earth Retaining Structures* 2008.

14.8.2 Crib Walls

Crib walls are built using prefabricated units which are stacked and interlocked and filled with free draining material. Cribs consist of solid interlocking reinforced concrete members called rails and tiebacks (sometimes called stretchers and headers). The rails run parallel with the wall face at both the front and rear of the cribbing and the tiebacks run transverse to the rails to tie the structure together. Rails and cross sections of tiebacks form the front face of the wall.

The wall face can either be opened or closed. In closed faced cribs, stretchers are placed in contact with each other. In open face cribs, the stretchers are placed at an interval such that the infill material does not escape through the face. The wall face batter for crib walls shall be no steeper than 4:1.

14.8.3 Gabion Walls

The gabion walls are composed of orthogonal wire cages or baskets tied together and filled with rock fragments. These wire baskets are also known as gabion baskets. The basket size can be varied to suit the terrain with a standard width of 3 feet to standard length varying 3 to 12 feet. The standard height of these baskets may vary from 1 foot to 3 feet. Individual wire baskets are filled with rock fragments ranging in size from 4 to 10 inches. After the baskets are filled, the lids are closed and wired shut to form a relatively rigid block. Succeeding rows of the gabions are laced in the field to the underlying gabions and are filled in the same manner until the wall reaches its design height. The rock filled baskets are closed with lids.

The durability of a gabion wall is dependent upon maintaining the integrity of the gabion baskets. Galvanized steel wire is required for all gabion installations. Although gabions are manufactured from a heavy gage wire, there is a potential for damage due to vandalism. While no known case of such vandalism has occurred on any existing WisDOT gabion walls, the potential for such action should be considered at specific sites.

A height of about 18 feet should be considered as a practical limit for gabion walls. Gabion walls have shown good economy for low to moderate heights but lose this economy as height increases. The front and rear face of the wall may be vertical or stepped. A batter is provided for walls exceeding heights of 10 feet, to improve stability. The wall face step shall not be steeper than 6” or 10:1(V:H). The minimum embedment for gabion walls is 1.5 feet. The ratio of the base width to height will normally range from 0.5 to 0.75 depending on backslope, surcharge and angle of internal friction of retained soil. Gabion walls should be designed in cross section with a horizontal base and a setback of 4 to 6 inches at each basket layer. This setback is an aid to construction and presents a more pleasing appearance. The use of a tipped wall base should not be allowed except in special circumstances.

14.8.4 Design Procedure

All prefabricated modular wall systems shall be designed to resist external pressure caused by the supported earth, surcharge loads, and water in accordance with design criteria discussed in **LRFD [11.11.4]** and 14.4 of this chapter. The design requires an external stability evaluation by the WISDOT/Consultant designer, including sliding, eccentricity, 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.

In addition, the structures modules of the bin and crib walls shall be designed to provide adequate resistance against structural failure as part of the internal stability evaluations in accordance with the guidelines presented in **LRFD [11.11.5]**.

No separate guidance is provided in the *AASHTO LRFD* for the gabion walls, therefore, gabion walls shall be evaluated for the external stability at Strength I and the settlement and overall stability checks at Service I using similar process as that of a prefabricated modular walls.

Since structure modules of the prefabricated modular walls are proprietary, the contractor/ supplier is responsible for the internal stability evaluation and the structural design of the structural modules. The design by contractor shall also meet the requirements for any special provisions. The external stability, overall stability check and the settlement evaluation will be performed by Geotechnical Engineer.

14.8.4.1 Initial Sizing and Wall Embedment

Wall backfill shall not be steeper than 2:1(V:H). Where practical, a minimum 4.0 feet wide horizontal bench shall be provided in front of the walls. A base width of 0.4 to 0.5 of the wall height can be considered initially for walls with no surcharge. For walls with surcharge loads or larger backslopes, an initial base width of 0.6 to 0.7 times can be considered.

Wall embedment for prefabricated modular walls shall meet the requirements discussed in 14.4.7.5. A minimum embedment shall be 1.5 ft or the requirement for scouring or erosion due to flooding.

14.8.5 Stability checks

Stability computations for crib, bin, and gabion modular wall systems shall be made by assuming that the wall modules and wall acts as a rigid body. Stability of gabion walls shall be performed assuming that gabions are flexible.

14.8.5.1 Unfactored and Factored Loads

All modular walls shall be investigated for lateral earth and water pressure including any live and/or dead load surcharge. Dead load due to self-weight and soil or rock in-fill shall also be included in computing the unfactored loads. Material properties for selected backfill, concrete, and steel shall be in accordance with guidelines suggested in 14.4.6. The properties of prefabricated modules shall be based on the type of wall modules being supplied by the wall suppliers.

The angle of friction δ between the back of the modules and backfill shall be used in accordance with the **LRFD [3.11.5.9]** and **LRFD [Table C3.11.5.9-1]**. Loading and earth pressure distribution diagram shall be developed as shown in Figure 14.4-6 or Figure 14.4-7

Since infill material and backfill materials of the gabion walls are well drained, no hydrostatic pressure is considered for the gabion walls. The unit weight of the rock-filled gabion baskets shall be computed in accordance with following:

$$\gamma_g = (1-\eta_r)G_s\gamma_w$$

Where:

η_r = Porosity of the rock fill

Gs = Specific gravity of the rock

γ_w = Unit weight of water

Free-draining granular material shall be used as backfill material behind the prefabricated modules in a zone of 1:1 from the heel of the wall. The soil design parameters shall be provided by the Geotechnical Engineer.

Factored loads and moments shall be computed as discussed in 14.4.5.5 and shall be multiplied by 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 14.8-1

14.8.5.2 External Stability

The external stability of the prefabricated modular walls shall be evaluated for sliding, eccentricity check, and bearing resistance in accordance with **LRFD [11.11.4].** It is assumed that the wall acts as a rigid body. **LRFD [11.11.4.1]** requires that wall stability be performed at every module level. The stability can be evaluated using procedure described in 14.7.1.2.

For prefabricated modular walls, the sliding analysis shall be performed by assuming that 80% of the weight of the soil in the modules is transferred to the footing supports with the remaining soil, weight being transferred to the area of the wall between footings.

The load resisting overturning shall also be limited to 80%, because the interior of soil can move with respect to the retaining module.

The bearing resistance shall be evaluated by assuming that 80% weight of the infill soil is transferred to point (or line) supports at the front or rear of the module.

14.8.5.3 Settlement

The vertical and lateral displacements of prefabricated modular retaining walls must be evaluated for all applicable dead and live load combinations at Service I using procedure described in 14.4.7.2 and compared with tolerable movement criteria presented in 14.4.7.2.1. In general, lateral movements of walls on shallow foundations can be estimated assuming the wall rotates or translates as a rigid body due to the effects of earth loads and differential settlements along the base of the wall.

14.8.5.4 Overall Stability

The overall (global) stability shall be evaluated in accordance with **LRFD [11.6.2.3]** and in accordance with 14.4.7.3 with the exception that the entire mass of the modular walls (or the "foundation load"), may be assumed to contribute to the overall stability of the slope. The overall stability check shall be performed by the Geotechnical Engineer.

14.8.5.5 Structural Resistance

Structural design of the modular units or members shall be performed in accordance with **LRFD [11.11.5]**. The design shall be performed using the factored loads developed for the geotechnical design (external stability) and for the factored pressures developed inside the modules in accordance with **LRFD [11.11.5.1]**. Design shall consider any potential failure mode, including tension, compression, shear, bending, and torsion. The contractor/wall supplier is responsible for the structural design of wall components.

14.8.6 Summary of Design Safety Factors and Requirements

Requirements

Stability Checks

- External Stability
  - Sliding
  - Overturning (eccentricity check)
  - Bearing Stress
- Internal Stability
  - Structural Components
- Settlement
- Overall Stability

Foundation Design Parameters

- Use values provided by Geotechnical Engineer

Concrete and steel Design Data

- f'_c = 4000 psi (or as required by design)
- f_y = 60,000 psi

Use uncoated bars or welded wire fabric

Traffic Surcharge

- Traffic live load surcharge = 240 lb/ft^2
- If no traffic live load is present, use 100 lb/ft^2 live load for construction equipment

Retained Soil

- Unit weight = 120 lb/ft^3
- Angle of internal friction =
  - Use value provided by Geotechnical Engineer
- Rock-infill unit weight =
  - Based on porosity and rock type

Soil Pressure Theory

- Coulomb's Theory for prefabricated wall systems
- Rankine theory or Coulomb theory, at the discretion of designer for gabion walls

7 Load Factors

Group	γ_{DC}	γ_{EV}	γ_{LSv}	γ_{LSh}	γ_{EH}	γ_{ES}	Probable use
Strength Ia	0.90	1.00	0.0	1.75	1.50	1.50	Sliding, eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50	1.50	Bearing, wall strength
Service I	1.00	1.00	1.00	1.00	1.00		Global, settlement, wall crack control

Table 14.8-1
Load Factor Summary for Prefabricated Modular Walls

14.9 Soil Nail Walls

Soil nail walls consist of installing reinforcement of the ground behind an excavation face, by drilling and installing closely-spaced rows of grouted steel bars (i.e., soil nails). The soil nails are grouted in place and subsequently covered with a facing; used to stabilize the exposed excavation face, support the sub-drainage system (i.e., composite strip drain, collector and drainage pipes), and distribute the soil nail bearing plate load over a larger area. When used for permanent applications, a permanent facing layer, meeting the aesthetic and structural requirement is constructed directly over the temporary facing.

Soil nail walls are typically used to stabilize excavation during construction. Soil nail walls have been used recently with MSE walls to form hybrid wall systems typically known as 'shored walls'. The soil nails are installed as top down construction. Conventional soil nail wall systems are best suited for sites with dense to very dense, granular soil with some apparent cohesion (sands and gravels), stiff to hard, fine-grained soil (silts and clays) of relatively low plasticity (PI<15), or weak, weathered massive rock with no adversely-oriented planes of weakness. Soil nail wall construction requires that open excavations stand unsupported long enough to allow soil nail drilling and grouting, sub-drainage installation, reinforcement, and temporary shotcrete placement. Soil nail walls should not be used below groundwater.

14.9.1 Design Requirements

AASHTO LRFD currently does not include the design and construction of soil nail walls. It is recommended that soil nail walls be designed using methods recommended in *Geotechnical Engineering Circular (GEC) No. 7 – Soil Nail Walls* (FHWA, 2003). The design life of the soil nail walls shall be in accordance with 14.4.3.

The design of the soil nailing walls requires an evaluation of external, internal, and overall stability and facing-connection failure mode as presented in Sections 5.1 thru Sections 5.6 of *(GEC) No. 7 – Soil Nail Walls* (FHWA, 2003).

A permanent wall facing is required for all permanent soil nail walls. Permanent facing is commonly constructed of cast-in-place (CIP) concrete, welded wire mesh (WWM) reinforced concrete and precast fabricated panels. In addition to meeting the aesthetic requirements and providing adequate corrosion protections to the soil nails, design facings for all facing-connection failure modes indicated in FHWA 2003.

Corrosion protection is required for all permanent soil nail wall systems to assure adequate long-term wall durability. . The level of corrosion protection required should be determined on a project-specific basis based on factors such as wall design life, structure criticality and the electrochemical properties of the supporting soil and rock materials. Criteria for classification of the supporting soil and rock materials as "aggressive" or "non-aggressive" are provided in FHWA 2003.

Soil nails are field tested to verify that nail design loads can be supported without excessive movement and with an adequate margin of safety. Perform both verification and proof testing of designated test nails as recommended in FHWA 2003.

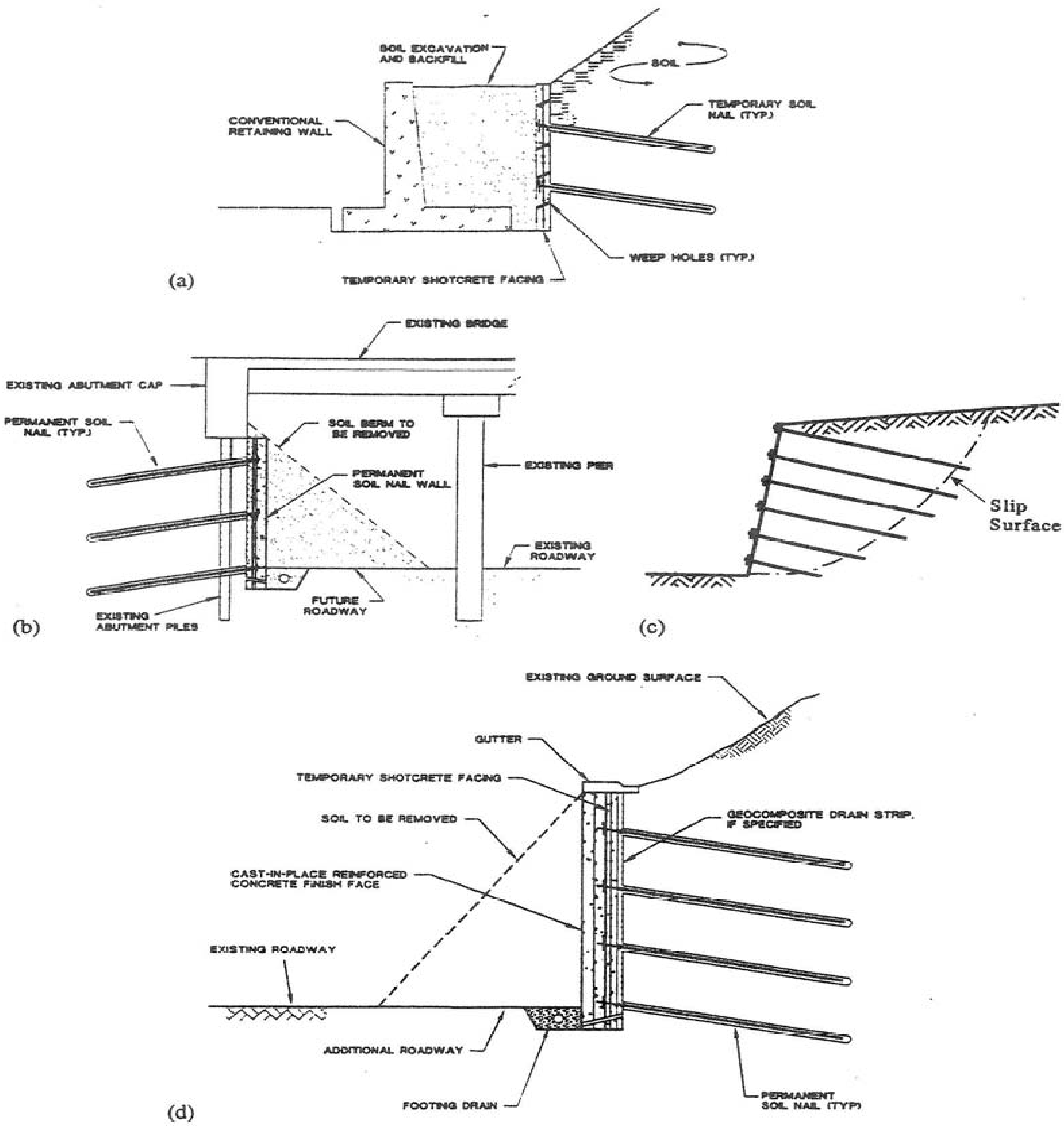


Figure 14.9-1
In-Situ Soil Nailed Walls
(Source: Earth Retaining Structures, 2008)

14.10 Steel Sheet Pile Walls

14.10.1 General

Steel sheet pile walls are a type of non-gravity wall and are typically used as temporary walls, but can also be used for permanent locations.

Sheet piling consists of interlocking steel, precast concrete or wood pile sections driven side by side to form a continuous unit. Steel is used almost exclusively for sheet pile walls. Individual pile sections usually vary from 12 to 21 inches in width, allowing for flexibility and ease of installation. The most common use of sheet piling is for temporary construction of cofferdams, retaining walls or trench shoring. The structural function of sheet piles is to resist lateral pressures due to earth and/or water. The steel manufacturers have excellent design references. Sheet pile walls generally derive their stability from sufficient pile penetration (cantilever walls). When sheet pile walls reach heights in excess of approximately 15 feet, the lateral forces are such that the walls need to be anchored with some form of tieback.

Cofferdams depend on pile penetration, ring action and the tensile strength of the interlocking piles for stability. If a sheet pile cofferdam is to be dewatered, the sheets must extend to a sufficient depth into firm material to prevent a "blow out", that is water coming in from below the base of the excavation. Cross and other bracing rings must be adequate and placed as quickly as excavation permits.

Sheet piling is generally chosen for its efficiency, versatility, and economy. Cofferdam sheet piling and any internal bracing are designed by the Contractor, with the design being accepted by the Department. Other forms of temporary sheet piling are designed by the Department. Temporary sheet piling is not the same as temporary shoring. Temporary shoring is designed by the Contractor and may involve sheet piling or other forms of excavation support.

14.10.2 Sheet Piling Materials

Although sheet piling can be composed of timber or precast concrete members, these material types are seldom, if ever, used on Wisconsin transportation projects.

Steel sheet piles are by far the most extensively used type of sheeting in temporary construction because of their availability, various sizes, versatility and ability to be reused. Also, they are very adaptable to permanent structures such as bulkheads, seawalls and wharves if properly protected from salt water.

Sheet pile shapes are generally Z, arched or straight webbed. The Z and the medium to high arched sections have high section moduli and can be used for substantial cantilever lengths or relatively high lateral pressures. The shallow arched and straight web sections have high interlocking strength and are employed for cellular cofferdams. The Z-section has a ball-and-socket interlock and the arched and straight webbed sections have a thumb-and-finger interlock capable of swinging 10 degrees. The thumb-and-finger interlock provides high tensile strength and considerable contact surface to prevent water passage. Continuous steel sheet piling is not completely waterproof, but does stop most water from passing through the joints. Steel sheet piling is usually 3/8 to 1/2 inch thick. Designers should specify the required section modulus and embedment depths on the plans, based on bending requirements and also account for corrosion resistance as appropriate.

Refer to steel catalogs for typical sheet pile sections. Contractors are allowed to choose either hot or cold rolled steel sections meeting the specifications. Previously used steel sheet piling may be adequate for some temporary situations, but should not be allowed on permanent applications.

14.10.3 Driving of Sheet Piling

All sheets in a section are generally driven partially to depth before all are driven to the final required depths. There is a tendency for sheet piles to lean in the direction of driving producing a net "gain" over their nominal width. Most of this "gain" can be eliminated if the piles are driven a short distance at a time, say from 6 feet to one third of their length before any single pile is driven to its full length. During driving if some sheet piles strike an obstruction, move to the next pile that can be driven and then return to the piles that resisted driving. With interlock guides on both sides and a heavier hammer, it may be possible to drive the obstructed sheet to the desired depth.

Sheet piles are installed by driving with gravity, steam, air or diesel powered hammers, or by vibration, jacking or jetting depending on the subsurface conditions, and pile type. A vibratory or double acting hammer of moderate size is best for driving sheet piles. For final driving of long heavy piles a single acting hammer may be more effective. A rapid succession of blows is generally more effective when driving in sand and gravel; slower, heavier blows are better for penetrating clay materials. For efficiency and impact distribution, where possible, two sheets are driven together. If sheets adjacent to those being driven tend to move down below the required depth, they are stopped by welding or bolting to the guide wales. When sheet piles are pulled down deeper than necessary by the driving of adjacent piles, it is generally better to fill in with a short length at the top, rather than trying to pull the sheet back up to plan location.

14.10.4 Pulling of Sheet Piling

Vibratory hammers are most effective in removing sheets and typically used. Sheet piles are pulled with air or steam powered extractors or inverted double acting hammers rigged for this application. If piles are difficult to pull, slight driving is effective in breaking them loose. Pulled sheet piling is to be handled carefully since they may be used again; perhaps several times.

14.10.5 Design Procedure for Sheet Piling Walls

A description of sheet pile design is given in **LRFD [3.11.5.6]** as "Cantilevered Wall Design" along with the earth pressure diagrams showing some simplified earth pressures. They are also referred to as flexible cantilevered walls. Steel sheet pile walls can be designed as cantilevered walls up to approximately 15 feet in height. Over 15 feet height, steel sheet pile walls may require tie-backs with either prestressed soil anchors, screw anchors, or deadman-type anchors.

The preferred method of designing cantilever sheet piling is by the "Conventional Method" as described in the *United States Steel Sheet Piling Design Manual* (February,1974). The Geotechnical Engineer provides the soil design parameters including cohesion values, angles of internal friction, wall friction angles, soil densities, and water table elevations. The lateral earth pressures for non-gravity cantilevered walls are presented in **LRFD [3.11.5.6].**

Anchored wall design must be in accordance with **LRFD [11.9]**. Anchors for permanent walls shall be fully encapsulated over their entire length. The anchor hardware shall be designed to have a corrosion resistance durability to ensure a minimum design life of 75 years.

All areas of permanent exposed steel sheet piling above the ground line shall be coated or painted prior to driving. Corrosion potential should be considered in all steel sheet piling designs. Special consideration should be given to permanent steel sheet piling used in areas of northern Wisconsin which are inhabited by corrosion causing bacteria (see *Facilities Development Manual*, Procedure 13-1-15).

Permanent sheet pile walls below the watertable may require the use of composite strip drains, collector and drainage pipes before placement of the final concrete facing.

The appearance of permanent steel sheet piling walls may be enhanced by applying either precast concrete panels or cast-in-place concrete surfacing. Welded stud-shear connectors can be used to attach cast-in-place concrete to the sheet piling. Special surface finishes obtained by using form liners or other means and concrete stain or a combination of stain and paint can be used to enhance the concrete facing aesthetics.

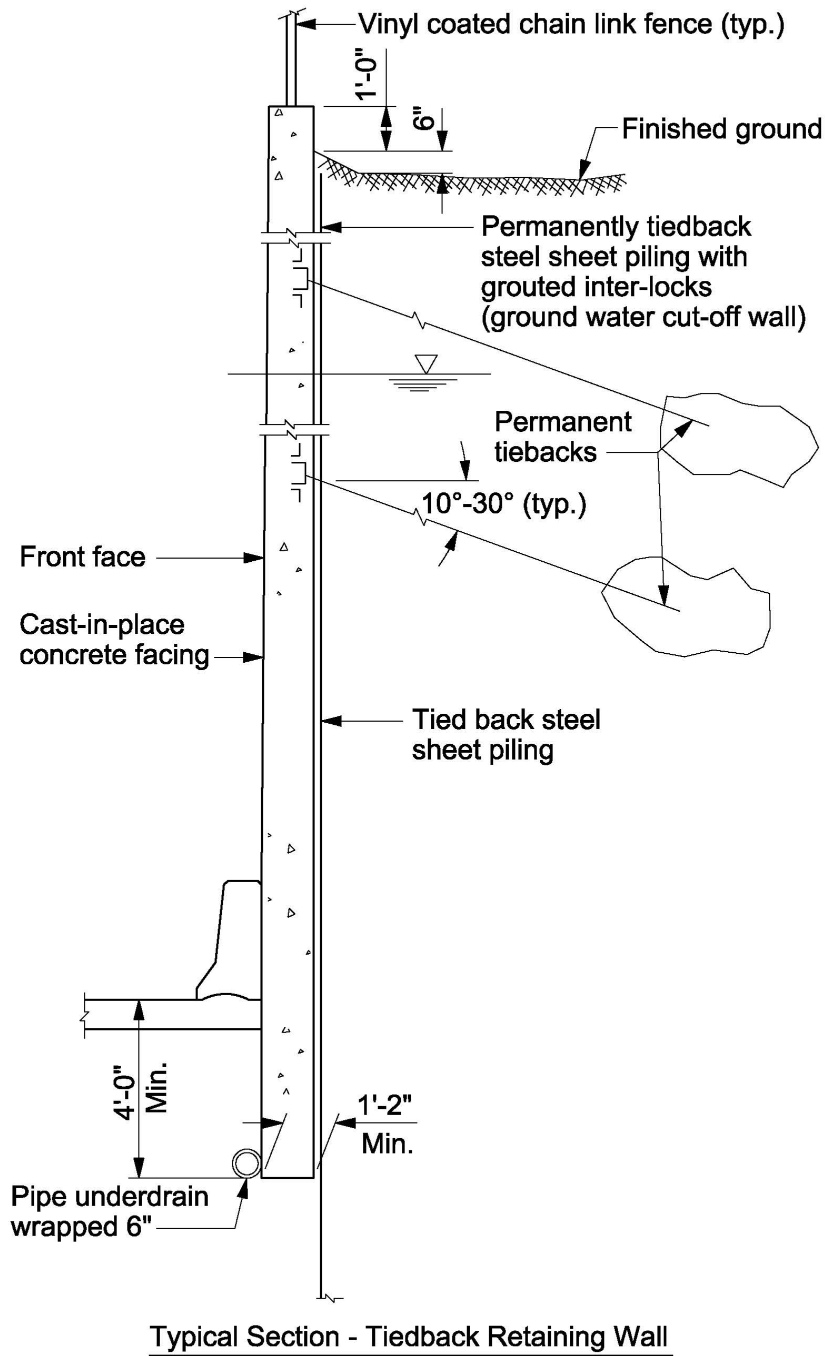


Figure 14.10-1
Typical Anchored Sheet Pile Wall

14.10.6 Summary of Design Requirements

1. Load and Resistance Factor

Load Combination	Load Factors	Resistance Factor
Strength I (maximum)	EH-Horizontal Earth Pressure: δ =1.50 **LRFD [Table 3.4.1-2]**	-----------
Strength I (maximum)	LS-Live Load Surcharge: δ =1.75 **LRFD [Table 3.4.1-1]**	-----------
Strength I (maximum)	-----------	Passive resistance of vertical elements: ϕ=0.75 **LRFD [Table 11.5.7-1]**
Service I	-----------	Overall Stability: ϕ=0.75, when geotechnical parameters are well defined, and the slope does not support or contain a structural element
Service I	-----------	Overall Stability: ϕ=0.65, when geotechnical parameters are based on limited information, or the slope does support or contain a structural element

Table 14.10-1
Summary of Design Requirements

2. Foundation design parameters

   Use values provided by the Geotechnical Engineer of record for permanent sheet pile walls. Temporary sheet pile walls are the Contractor's responsibility.

3. Traffic surcharge

   - Traffic live load surcharge = 240 lb/ft^2 or determined by site condition.
   - If no traffic live load is present, use 100 lb/ft^2 live load for construction equipment

4. Retained soil

   - Unit weight = 120 lb/ft^3
   - Angle of internal friction as determined from the Geotechnical Report.

5. Soil pressure theory

Coulomb Theory.

6. Design life for anchorage hardware

   75 years minimum

7. Steel design properties

   Minimum yield strength = 39,000 psi

14.11 Soldier Pile Walls

Soldier pile walls are comprised of discrete vertical elements (usually steel H piles) and facing members (temporary and/or permanent) that extend between the vertical elements.

14.11.1 Design Procedure for Soldier Pile Walls

LRFD [11.8] Non-Gravity Cantilevered Walls covers the design of soldier pile walls. A simplified earth pressure distribution diagram is shown in **LRFD [3.11.5.6]** for permanent soldier pile walls. Another method that may be used is the "Conventional Method" or “Simplified Method” as described in "*United States Steel Sheet Piling Design Manual*", February, 1974. This method must be modified for the fact that it is based on continuous vertical wall elements whereas, soldier pile walls have discrete vertical wall elements. Using "Broms" method for designing drilled shafts is also acceptable.

The maximum spacing between vertical supporting elements (piles) depends on the wall height and the design parameters of the foundation soil. Spacing of 6 to 12 feet is typical. The piles are set in drilled holes and concrete is placed in the hole after the post is set. The pile system must be designed to handle maximum bending moment along length of embedded shaft. The maximum bending moment at any level in the facing can be determined from formulas in **LRFD [11.8.5.1]**. The minimum structural thickness on wall facings shall be 6 inches for precast panels and 10 inches with cast-in-place concrete.

The diameter of the drilled shaft is also dependent on the wall height and the design parameters of the foundation soil. The larger the diameter of the drilled shaft the smaller will be the required embedment of the shaft. The designer should try various shaft diameters to optimize the cost of the drilled shaft considering both material cost and drilling costs. Note that drilling costs are a function of both hole diameter and depth.

If the vertical elements are steel they shall be shop painted. Wall facings are usually given a special surface treatment created by brooming or tining vertically, using form liners, or using a pattern of rustication strips. The portion of the panel receiving the special treatment may be recessed, forming a border around the treated area. Concrete paints or stains may be used for color enhancements. When panel heights exceed 15 feet anchored walls may be needed. Anchored wall design must be in accordance with **LRFD [11.9]**. Anchors for permanent walls shall be fully encapsulated over their entire length. The anchor hardware shall be designed to have a corrosion resistance durability to ensure a minimum design life of 75 years.

The concrete for soldier pile walls shall have a 28 day compressive strength of 4000 psi if non-prestressed and 5000 psi if prestressed except for the drilled shafts. Concrete for the drilled shafts shall have a 28 day compressive strength of 3500 psi. Reinforcement shall be uncoated Grade 60 in drilled shafts. In lieu of drainage aggregate a membrane may be used to seal the joints between the vertical elements and concrete panels to prevent water leakage. The front face of soldier pile walls shall be battered 1/4" per foot to account for short and long term deflection.

14.11.2 Summary of Design Requirements

Requirements

1. Resistance Factors

   - Overall Stability= 0.65 to 0.75 (based on how well defined the geotechnical parameters are and the support of structural elements)
   - Passive Resistance of vertical Elements = 0.75

2. Foundation Design Parameters

   Use values provided by the Geotechnical Engineer (unit weight, angle of internal friction, and cohesion). Both drained and undrained parameters shall be considered.

3. Concrete Design Data

   - f'_c = 3500 psi (for drilled shafts)
   - f'_c = 4000 psi (non-prestressed panel)
   - f'_c = 5000 psi (prestressed panel)
   - f_y = 60,000 psi

4. Load Factors

   - Vertical earth pressure = 1.5
   - Lateral earth pressure = 1.5
   - Live load surcharge = 1.75

5. Traffic Surcharge

   - Traffic live load surcharge = 2 feet = 240 lb/ft^2
   - If no traffic surcharge, use 100 lb/ft^2

6. Retained Soil

   Use values provided by the Geotechnical Engineer (unit weight, angle of internal friction, and cohesion). Both drained and undrained parameters shall be considered.

7. Soil Pressure Theory

   Rankine's Theory or Coulombs Theory at the discretion of the designer.

8. Design Life for Anchorage Hardware

   75 year minimum

9. Steel Design Properties (H-piles)

   Minimum yield strength = 50,000 psi

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 Structures Design Section (608-266-8494) 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

After the designer has established the suitable wall system(s), the method of contract letting can be determined. The designer has several options based on the contents of the list.

Option 1:

The list contains only non-proprietary systems.

Under Option 1, the designer will furnish a complete design for one of the non-proprietary systems.

Option 2:

The list contains proprietary wall systems only or may contain both proprietary and non-proprietary wall systems, but the proprietary wall systems are deemed more appropriate than the non-proprietary systems.

Under Option 2 the designer will not furnish a design for any wall system. The contractor can build any wall system which is included on the list. The contractor is responsible for providing the complete design of the wall system selected, either by the wall supplier for proprietary walls or by the contractor's engineer for non-proprietary walls. Contract special provisions, if not in the Supplemental Specs., must be included in the contract document for each wall system that is allowed. Under Option 2, at least two and preferably three wall suppliers must have an approved product that can be used at the project site. See the *Facilities Development Manual* (Procedure 19-1-5) for any exceptions.

Option 3:

The list contains proprietary wall systems and non-proprietary wall systems and the non-proprietary systems are deemed equal or more appropriate than the proprietary systems.

Under Option 3 the designer will furnish a complete design for one of the non-proprietary systems, and list the other allowable wall systems.

Step 6: Prepare Contract Plans

Refer to section 14.16 for information required on the contract plans for proprietary systems. If a contractor chooses an alternate wall system, the contractor will provide the plans for the wall system chosen.

Step 7: Prepare Contract Special Provisions

The Structures Design Section and Region Offices have Special Provisions for each wall type and a generic Special Provision to be used for each project. The list of proprietary wall suppliers is maintained by the Materials Quality Assurance Unit.

Complete the generic Special Provision for the project by inserting the list of wall systems allowed and specifying the approved list of suppliers if proprietary wall systems are selected.

Step 8: Submit P.S.& E. (Plans, Specifications and Estimates)

When the plans are completed and all other data is completed, submit the project into the P.S.& E. process. Note that there is one bid item, square feet of exposed wall, for all wall quantities.

Step 9: Preconstruction Review

The contractor must supply the name of the wall system supplier and pertinent construction data to the project manager. This data must be accepted by the Office of Design, Contract Plans Section before construction may begin. Refer to the Construction and Materials Manual for specific details.

Step 10: Project Monitoring

It is the responsibility of the project manager to verify that the project is constructed with the previously accepted contract proposal. Refer to the Construction and Materials Manual for monitoring material certification, construction procedures and material requirements.

14.13.2 Pre-Approval Process

The purpose of the pre-approval process is to ascertain that a particular proprietary wall system has the capability of being designed and built according to the requirements and specifications of WisDOT. Any unique design requirements that may be required for a particular system are also identified during the pre-approval process. A design of a pre-approved system is acceptable for construction only after WisDOT has verified that the design is in accordance with the design procedures and criteria stated in the Certification Method of Acceptance for Noise Barrier Walls.

In addition to design criteria, suppliers must provide materials testing data and certification results for the required tests for durability, etc. The submittal requirements for the pre-approval process and other related information are available from the Materials Quality Assurance Unit, Madison, Wisconsin.

14.14 Contract Plan Requirements

The following minimum information shall be required on the plans.

1. Finish grades at rear and front of wall at 25 foot intervals or less.

2. Final cross sections as required for wall designer.

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

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

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

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

8. General notes on standard insert sheets.

9. 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.

10. Soil borings.

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

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

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

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

15. Groundwater elevations.

16. Drainage provisions at heel of wall foundations.

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

18. 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**
- *Wall Modular Bin or Crib**
- *Wall CIP Facing Mechanically Stabilized Earth**

** SPV under development. Contact the Bureau of Structures for usage.*

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. Berg, Christopher and Samtani. *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes*. Publication No.FHWA-NHI-10-024.2009.
6. Bowles, Joseph E*. Foundation Analysis and Design 4th Edition.* McGraw Hill *1989*
7. Cudoto, Donald P. *Foundation Design Principles and Practices (* 2nd Edition*),* Prentice Halls
8. National Concrete Masonry Association, "Design Manual for Segmental Retaining Walls", 2302 Horse Pen Road, Herndon, Virginia 22071-3406.
9. Lazarte, Elias, Espinoza, Sabatini. *Geotechnical Engineering Circular No 7. Soil Nailing Walls*, FHWA
10. Publication No FHWA-SA-96-069R , "*Manual for design and construction of Soil Nail walls*
11. Publication No.FHWA-HI-98-032, "Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures".
12. Publication No.FHWA-NHI-07-071, "Earth retaining Structures".
13. Publication No.FHWA-NHI-09-083, "Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures".
14. Publication No. FHWA-NHI-09-087, "Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced slopes"
15. Publication No.FHWA-NHI-10-024, "Design and Construction of Mechanically Stabilized earth Walls and Reinforced Soil Slopes-Volume I".
16. Publication No.FHWA-NHI-10-025, "Design and Construction of Mechanically Stabilized earth Walls and Reinforced Soil Slopes-Volume II".

14.18 Design Examples

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$$\Delta_i = \Delta_{PS} - \Delta_{o(DL)}$$

Where:

Δ_i = Prestress camber at release (in)

Camber, however, continues to grow after the initial strand release. For determining substructure beam seats, average concrete haunch values (used for both DL and quantity calculations) and the required projection of the vertical reinforcement from the tops of the prestressed girders, **a camber multiplier of 1.4 shall be used**. This value is multiplied by the theoretical camber at release value.

19.3.3.18.2 Dead Load Deflection

The downward deflection of a prestressed I-girder due to the dead load of the deck and a midspan diaphragm is:

$$\Delta_{nc(DL)} = \frac{5W_{deck}L^4}{384EI_b} + \frac{P_{dia}L^3}{48EI_b} \quad \text{(with all units in inches and kips)}$$

Using span lengths in units of feet, unit weights in kips per foot, E in ksi, and I_b in inches^4, this equation becomes the following:

$$\Delta_s = \frac{5W_{deck}L^4}{384EI_b}\left(\frac{1}{12}\right)\left(\frac{12^4}{1}\right) + \frac{P_{dia}L^3}{48EI_b}\left(\frac{12^3}{1}\right) = \frac{5W_{deck}L^4}{384EI_b}\left(\frac{20736}{12}\right) + \frac{P_{dia}L^3}{48EI_b}\left(\frac{1728}{1}\right)$$

$$\Delta_{o(DL)} = \frac{22.5W_bL^4}{EI_b} + \frac{36P_{dia}L^3}{EI_b} \quad \text{(with units as shown below)}$$

Where:

$\Delta_{nc(DL)}$ = Deflection due to non-composite dead load (deck and midspan diaphragm) (in)

W_{deck} = Deck weight per unit length (k/ft)

P_{dia} = Midspan diaphragm weight (kips)

E = Girder modulus of elasticity at final condition (see 19.3.3.8) (ksi)

A similar calculation is done for parapet and sidewalk loads on the composite section. Provisions for deflections due to future wearing surface shall not be included.

For girder structures with raised sidewalks, loads shall be distributed as specified in Chapter 17, and separate deflection calculations shall be performed for the interior and exterior girders.

19.3.3.18.3 Residual Camber

Residual camber is the camber that remains after the prestress camber has been reduced by the composite and non-composite dead load deflection. Residual camber is computed as follows:

$$RC = \Delta_i - \Delta_{nc(DL)} - \Delta_{c(DL)}$$

19.3.4 Prestressed I-Girder Deck Forming

Deck forming requires computing the relationship between the top of girder and bottom of deck necessary to achieve the desired vertical roadway alignment. Current practice for design is to use a minimum haunch of 2" at the edge of the girder flange. This will facilitate current deck forming practices which use 1/2" removable hangers and 3/4" plywood, and it will allow for variations in prestress camber. Also, future deck removal will be less likely to damage the top girder flanges. See 19.3.3.1 for the method to determine haunch height for section properties. An average haunch height of 3 inches minimum can be used for determining haunch weight for preliminary design. It should be noted that the actual haunch values for weight calculations should be compared with the estimated values during final design. If there are significant differences in these values, the design should be revised. The actual average haunch height should be used to calculate the concrete quantity reported on the plans as well as the value reported on the prestressed girder details sheet. The actual haunch values at the girder ends shall be used for determining beam seat elevations.

For designs involving vertical curves, Figure 19.3-6 shows two different cases.

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36.1 Design Method

36.1.1 Design Requirements

All new box culverts are to be designed using *AASHTO LRFD Bridge Design Specifications*, hereafter referred to as *AASHTO LRFD*.

36.1.2 Rating Requirements

The current version of *AASHTO Manual for Bridge Evaluation (LRFR)* covers rating of concrete box culverts. Refer to 45.8 for additional guidance on load rating various types of culverts.

36.1.3 Standard Permit Design Check

New structures are also to be checked for strength for the 190 kip Wisconsin Standard Permit Vehicle (Wis-SPV), with a single lane loaded, multiple presence factor equal to 1.0, and a live load factor (γ_{LL}) as shown in Table 45.3-3. See 45.12 for the configuration of the Wis-SPV. The structure should have a minimum capacity to carry a gross vehicle load of 190 kips, while also supporting the future wearing surface (where applicable – future wearing surface loads are only applied to box culverts with no fill). When applicable, this truck will be designated as a Single Trip Permit Vehicle. It will have no escorts restricting the presence of other traffic on the culvert, no lane position restrictions imposed and no restrictions on speed to reduce the dynamic load allowance, IM. The maximum Wisconsin Standard Permit Vehicle load that the structure can resist, calculated including current wearing surface loads, is shown on the plans.

36.2 General

Box culverts are reinforced concrete closed rigid frames which must support vertical earth and truck loads and lateral earth pressure. They may be either single or multi-cell. The most common usage is to carry water under roadways, but they are frequently used for pedestrian or cattle underpasses.

Box culverts used to carry water should consider the following items:

- Hydraulic and other requirements at the site determine the required height and area of the box. Hydraulic design of box culverts is described in Chapter 8.
- Once the required height and area is determined, the selection of a single or multi-cell box is determined entirely from economics. Barrel lengths are computed to the nearest 6 inches. For multi-cell culverts the cell widths are kept equal.
- A minimum vertical opening of 5 feet is desirable for cleaning purposes.

Pedestrian underpasses should consider the following items:

- The minimum opening for pedestrian underpasses is 8 feet high by 10 feet wide. However, when considering maintenance and emergency vehicles or bicyclists the minimum opening should be 10 feet high by 12 feet wide. For additional guidance refer to the Wisconsin Bicycle Facility Design Handbook and the FDM.
- The entire top and 1 foot below the bottom of the top slab should be waterproofed.
- The top of the bottom slab should be sloped with a 1% normal crown to minimize moisture collecting on the travel path. Additionally, 0.5% to 1% longitudinal slope for drainage is recommended.
- Flared wings are recommended at openings. For long underpasses, lighting systems (recessed lights and skylights) should be considered, as well. For additional guidance on user's comfort, safety measures, and lighting refer to the Wisconsin Bicycle Facility Design Handbook.

Cattle underpasses should consider the following items:

- The minimum size for cattle underpasses is 6 feet high by 5 feet wide.
- Consider providing a minimum longitudinal slope of 1%, desirable 3%, to allow for flushing, but not so steep that the stock will slip. Slopes steeper than 5% should be avoided.
- For additional guidance refer to the FDM.

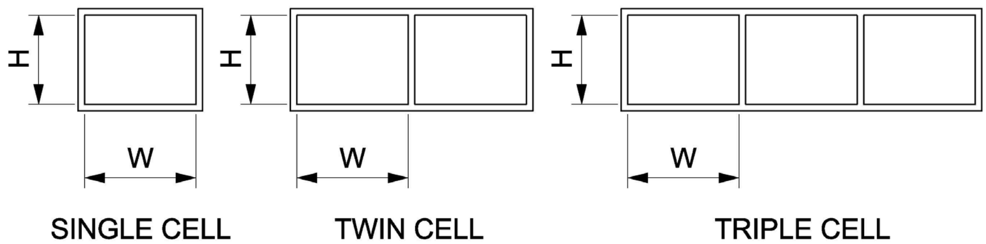

Figure 36.2-1
Typical Cross Sections

36.2.1 Material Properties

The properties of materials used for concrete box culverts are as follows:

f'_c = specified compressive strength of concrete at 28 days, based on cylinder tests

= 3.5 ksi for concrete in box culverts

f_y = 60 ksi, specified minimum yield strength of reinforcement (Grade 60)

E_s = 29,000 ksi, modulus of elasticity of steel reinforcement **LRFD [5.4.3.2]**

E_c = modulus of elasticity of concrete in box **LRFD [C5.4.2.4]**
= $(33,000)(K_1)(w_C)^{1.5}(f'_C)^{1/2}$ = 3586 ksi

Where:

K_1 = 1.0

W_C = 0.15 kcf, unit weight of concrete

n = Es / Ec = 8, modular ratio **LRFD [5.6.1]**

36.2.2 Bridge or Culvert

Occasionally, the waterway opening(s) for a highway-stream crossing can be provided for by either culvert(s) or bridge(s). Consider the hydraulics of the highway-stream crossing system in choosing the preferred design from the available alternatives. Estimates of life cycle costs and risks associated with each alternative help indicate which structure to select. Consider construction costs, maintenance costs, and risks of future costs to repair flood damage. Other considerations which may influence structure-type selection are listed in Table 36.2-1.

36.4 Design Loads

36.4.1 Self-Weight (DC)

Include the structure self-weight based on a unit weight of concrete of 0.150 kcf. When there is no fill on the top slab of the culvert, the top slab thickness includes a ½" wearing surface. The weight of the wearing surface is included in the design, but its thickness is not included in the section properties of the top slab.

36.4.2 Future Wearing Surface (DW)

If the fill depth over the culvert is greater than zero, the weight of the future wearing surface shall be taken as zero. If there is no fill depth over the culvert, the weight of the future wearing surface shall be taken as 20 psf. This load is designated as, DW, dead load of wearing surfaces and utilities, for application of load factors and limit state combinations.

36.4.3 Vertical and Horizontal Earth Pressure (EH and EV)

WisDOT Policy Item:

Box Culverts are assumed to be rigid frames. Use Vertical Earth Pressure load factors for rigid frames, in accordance with **LRFD [Table 3.4.1-2]**.

The weight of soil above the buried structure is taken as 0.120 kcf. A coefficient of lateral earth pressure of 0.5 is used for the lateral pressure from the soil. This coefficient of lateral earth pressure is based on an at-rest condition and an effective friction angle of 30º, **LRFD [3.11.5.2]**. The lateral earth pressure is calculated per **LRFD [3.11.5.1]**:

$$p = k_o \gamma_s z$$

Where:

p	=	Lateral earth pressure (ksf)
k_o	=	Coefficient of at-rest lateral earth pressure
γ_s	=	Unit weight of backfill (kcf)
z	=	Depth below the surface of earth fill or top of roadway pavement (ft)

WisDOT Policy Item:

For modification of earth loads for soil-structure interaction, embankment installations are always assumed for box culvert design, in accordance with **LRFD [12.11.2.2]**.

Soil-structure interaction for vertical earth loads is computed based on **LRFD [12.11.2.2]**. For embankment installations, the total unfactored earth load is:

$$W_E = F_e \gamma_s B_c H$$

In which:

$$F_e = 1 + 0.20\frac{H}{B_c}$$

Where:

W_E	=	Total unfactored earth load (kip/ft width)
F_e	=	Soil-structure interaction factor for embankment installations (F_e shall not exceed 1.15 for installations with compacted fill along the sides of the box section)
γ_s	=	Unit weight of backfill (kcf)
B_c	=	Outside width of culvert, as specified in Figure 36.4-1 (ft)
H	=	Depth of fill from top of culvert to surface of earth fill or top of roadway pavement (ft)

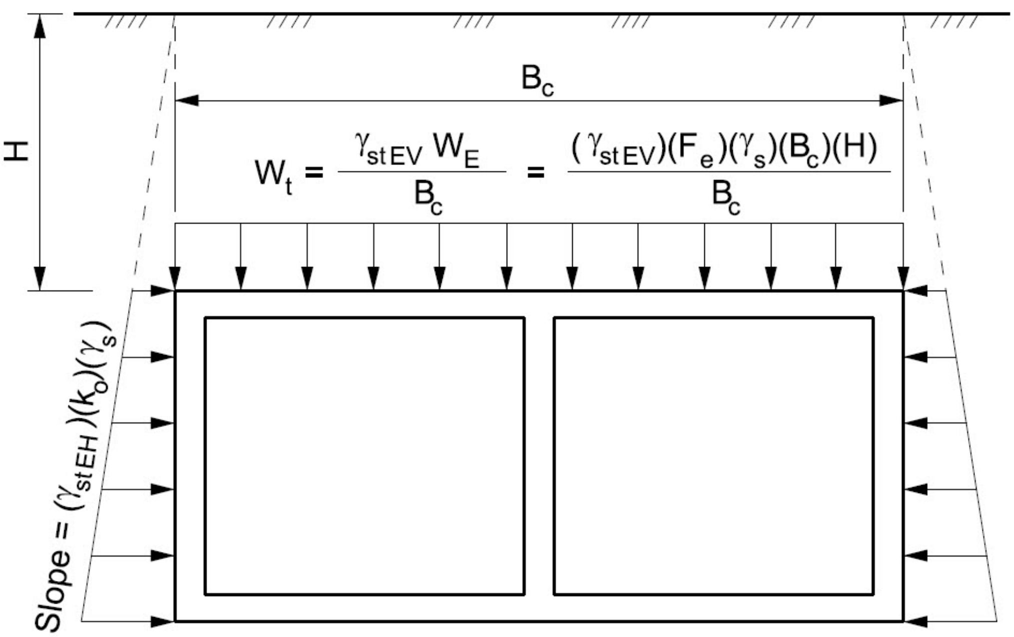

Figure 36.4-1
Factored Vertical and Horizontal Earth Pressures

Where:

W_t = Factored earth pressure on top of box culvert (ksf)

γ_{stEV} = Vertical earth pressure load factor

γ_{stEH} = Horizontal earth pressure load factor

k_o = Coefficient of at-rest lateral earth pressure

γ_s = Unit weight of backfill (kcf)

Figure 36.4-1 shows the factored vertical and horizontal earth load pressures acting on a box culvert. The soil pressure on the bottom of the box is not shown, but shall be determined for the design of the bottom slab. Note: vertical earth pressures, as well as other loads (e.g. DC and DW), are typically distributed equally over the bottom of the box when determining pressure distributions for the bottom slab. Pressure distributions from a refined analysis is typically not warranted for new culvert designs, but should be considered when evaluating existing culvert sections on culvert extension projects.

36.4.4 Live Load Surcharge (LS)

Per **LRFD [3.11.6.4]**, a live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the distance from top of pavement to bottom of the box culvert.

Per **LRFD [Table 3.11.6.4-1]**, the following equivalent heights of soil for vehicular loading shall be used. The height to be used in the table shall be taken as the distance from the bottom of the culvert to the roadway surface. Use interpolation for heights other than those listed in the table.

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

Table 36.4-1
Equivalent Height of Soil for Vehicular Loading

Surcharge loads are computed based on a coefficient of lateral earth pressure times the unit weight of soil times the height of surcharge. A coefficient of lateral earth pressure of 0.5 is used for the lateral pressure from the soil, as discussed in 36.4.3. The uniform distributed load is applied to both exterior walls with the load directed toward the center of the box culvert. The load is designated as, LS, live load surcharge, for application of load factors and limit state combinations. Refer to **LRFD [3.11.6.4]** for additional information regarding live load surcharge.

36.4.5 Water Pressure (WA)

Static water pressure loads are computed when the water height on the outside of the box is greater than zero. The water height is measured from the bottom inside of the box culvert to the water level. The load is designated as, WA, water pressure load, for application of load factors and limit state combinations. Water pressure in culvert barrels is ignored. Refer to **LRFD [3.7.1]** for additional information regarding water pressure.

36.4.6 Live Loads (LL)

Live load consists of the standard AASHTO LRFD trucks and tandem. Per **LRFD [3.6.1.3.3]**, design loads are always axle loads (single wheel loads should not be considered) and the lane load is not used. The depth of fill is measured from top of culvert to surface of earth fill or top roadway pavement.

Where the depth of fill over the box is less than 2 feet, the wheel loads are distributed per **LRFD [4.6.2.10]**. Where the depth of fill is 2 feet or more, the wheel loads shall be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area **LRFD [3.6.1.2.5]**, increased by the live load distribution factor (LLDF) in **LRFD [Table 3.6.1.2.6a-1]**, using the provisions of **LRFD [3.6.1.2.6b-c]**. Where areas from distributed wheel loads overlap at the top of the culvert, the total load is considered as uniformly distributed over the rectangular area (A_{LL}) defined by the outside limits described in **LRFD [3.6.1.2.6b-c]**.

Per **LRFD [3.6.1.2.6a]**, for single-span culverts, the effects of live load may be neglected where the depth of fill is more than 8.0 feet and exceeds the span length. For multiple span culverts, the effects may be neglected where the depth of fill exceeds the distance between inside faces of end walls.

Skew is not considered for design loads.

36.4.6.1 Depth of Fill Less than 2.0 ft.

Where the depth of fill is less than 2.0 ft, follow **LRFD [4.6.2.10]**.

36.4.6.1.1 Case 1 – Traffic Travels Parallel to Span

When the traffic travels primarily parallel to the span, follow **LRFD [4.6.2.10.2]**. Use a single lane and the single lane multiple presence factor of 1.2.

Distribution length perpendicular to the span:

$$E = (96 + 1.44(S))$$

Where:

E = Equivalent distribution width perpendicular to span (in.)

S = Clear span (ft)

The distribution of wheel loads perpendicular to the span for depths of fill less than 2.0 feet is illustrated in Figure 36.4-2.

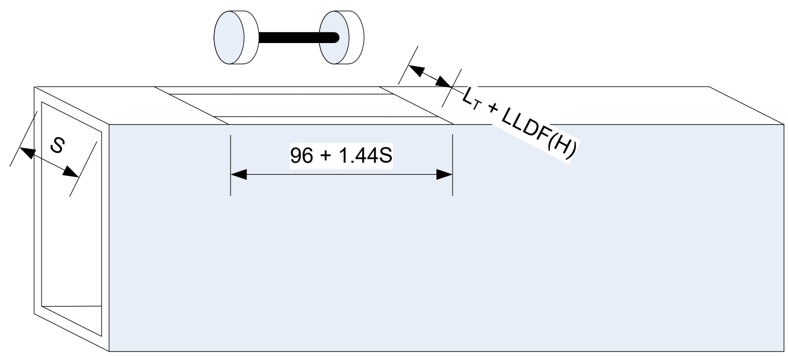

Figure 36.4-2
Distribution of Wheel Loads Perpendicular to Span, Depth of Fill Less than 2.0 feet

Distribution length parallel to the span:

$$E_{span} = (L_T + LLDF\ (H))$$

Where:

E_{span}	=	Equivalent distribution length parallel to span (in.)
L_T	=	Length of tire contact area parallel to span, as specified in **LRFD [3.6.1.2.5]** (in.)
LLDF	=	Factor for distribution of live load with depth of fill, 1.15, as specified in **LRFD [Table 3.6.1.2.6a-1]**.
H	=	Depth of fill from top of culvert to top of pavement (in.)

The distribution of wheel loads parallel to the span for depths of fill less than 2.0 feet is illustrated in Figure 36.4-3.

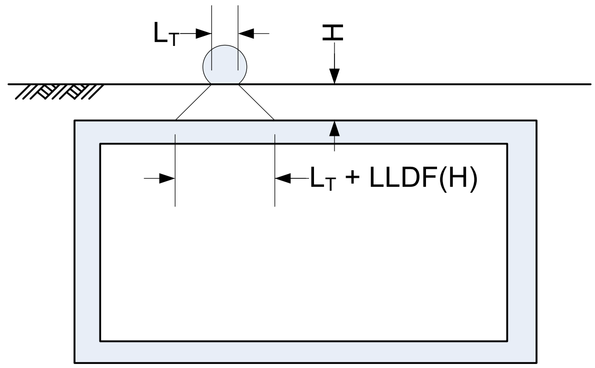

Figure 36.4-3
Distribution of Wheel Loads Parallel to Span, Depth of Fill Less than 2.0 feet

36.4.6.1.2 Case 2 - Traffic Travels Perpendicular to Span

When traffic travels perpendicular to the span, live load shall be distributed to the top slab using the equations specified in **LRFD [4.6.2.1]** for concrete decks with primary strips perpendicular to the direction of traffic per **LRFD[4.6.2.10.3]**. The effect of multiple lanes shall be considered. Use the multiple presence factor, m, as required per **LRFD [3.6.1.1.2]**.

For a cast-in-place box culvert, the width of the primary strip, in inches is:

+M: 26.0 + (6.6)(S)

-M: 48.0 + (3.0)(S)

as stated in **LRFD [Table 4.6.2.1.3-1]**

Where:

S	=	Spacing of supporting components (ft)
+M	=	Positive moment
-M	=	Negative moment

36.5 Design Information

Sidesway of the box is not considered because of the lateral support of the soil.

The centerline of the walls and top and bottom slabs are used for computing section properties and dimensions for analysis.

WisDOT Policy Item:

For skews 20 degrees or less, place the reinforcing steel along the skew. For skews over 20 degrees, place the reinforcing steel perpendicular to the centerline of box.

Culverts are analyzed as if the reinforcing steel is perpendicular to the centerline of box for all skew angles.

The minimum thickness of the top and bottom slab is 6½ inches. For pedestrian underpasses and slabs with fills less than 2 feet, the minimum thickness of the top slab should be 1 foot. Minimum wall thickness is based on the inside opening of the box (height) and the height of the apron wall above the floor. Use the following table to select the minimum wall thickness that meets or exceeds the three criteria in the table.

Minimum Wall Thickness (Inches)	Cell Height (Feet)	Apron Wall Height Above Floor (Feet)
8	< 6	< 6.75
9	6 to < 10	6.75 to < 10
10	10 to > 10	10 to < 11.75
11		11.75 to < 12.5
12		12.5 to 13

Table 36.5-1
Minimum Wall Thickness Criteria

All slab thicknesses are rounded to the next largest ½ inch.

Top and bottom slab thicknesses are determined by shear and moment requirements. Slab thickness shall be adequate to carry the factored shear without shear reinforcement.

All bar steel is detailed as being 2 inches clear with the following exceptions:

- The bottom steel in the bottom slab is detailed with 3 inches clear
- The top steel in the top slab of a box culvert with no fill is detailed with 2½ inches clear

A haunch is provided only when the slab depth required at the interior wall is more than 2 inches greater than that required for the remainder of the span. Only 45º haunches shall be used. Minimum haunch depth and length is 6 inches. Haunch dimensions are increased in 3 inch increments.

The slab thickness required is determined by moment or shear, whichever governs.

The shear in the top and bottom slabs is assumed to occur at a distance "d" from the face of the walls. The value for "d" equals the distance from the centroid of the reinforcing steel to the face of the concrete in compression. When a haunch is used, shear must also be checked at the end of the haunch.

For multi-cell culverts make interior and exterior walls of equal thickness.

Culverts shall be designed for live load and the range of fill between the shoulders of the roadway. The depth of fill is measured from the top of culvert to the surface of earth fill or top of roadway pavement. To accommodate future widening of the roadway, reduced sections may not be used on the ends of the culvert where there is less fill. Exceptions may be made with the approval of the Bureau of Structures where the culvert has high fills and a reduced section could be used for at least two panel pours per end of culvert. Culvert extensions shall be designed for the same range of fills as the original culvert. The extension design shall not have lower capacity than the original culvert. Maximum length of panel pour is 40 feet.

Barrel lengths are based on the roadway sections and wing lengths are based on a minimum 2 1/2:1 slope of fill from the top of box to apron. Consideration shall be given to match the typical roadway cross slope.

Dimensions on drawings are given to the nearest ¼ inch only.

36.13 Other Buried Structures

The following section provides general guidance on cross-drain alternatives to concrete box culverts.

36.13.1 General

Typical alternatives to four-sided (box) concrete structures include three-sided (bottomless) concrete structures and metal buried structures. These structures are available in a variety of shapes, sizes, and material types. In general, three-sided structures may be cost prohibitive when deep foundations are required.

Concrete buried structures are rigid structures that can be constructed using cast-in-place or precast concrete. These structures obtain strength through reinforced concrete sections that have proven to be durable and long-lasting. Refer to 36.13.2 for additional information on three-sided concrete structures.

Metal buried structures are typically constructed with factory assembled corrugated sections or field assembled structural plates. Commonly used shapes include pipes and pipe-arches consisting of steel or aluminum alloy. These flexible structures obtain strength through soil-structure interactions that allow for the use of thin-walled sections. Some advantages of metal buried structures include; increased speed of installation, potential initial cost savings, and the variety of available shapes. Some disadvantages include their susceptibility to damage and/or degradation and performance being dependent on the quality of installation. Refer to 36.13.3 and FDM 13-1 for additional information on metal buried structures.

Buried structures assigned a structure number shall be coordinated with the Bureau of Structures and follow the policies and procedures as stated in the Bridge manual and FDM 13-1. Refer to 2.5 for information on assigning structure numbers.

Refer to AASHTO LRFD Section 12 – Buried Structures and Tunnel Liners for additional information.

36.13.2 Three-Sided Concrete Structures

Three-sided box culvert structures are divided into two categories: cast-in-place three sided structures and precast three-sided structures. These structures shall follow the criteria outlined below.

36.13.2.1 Cast-In-Place Three-Sided Structures

To be developed

36.13.2.2 Precast Three-Sided Structures

Three-sided precast concrete structures offer a cost effective, convenient solution for a variety of bridge needs. The selection of whether a structure over a waterway should be a culvert, a three-sided precast concrete structure or a bridge is heavily influenced by the hydraulic opening. As the hydraulic opening becomes larger, the selection process for structure type progresses from culvert to three-sided precast concrete structure to bridge. Cost, future maintenance, profile grade, staging, skew, soil conditions and alignment are also important variables which should be considered. Culverts generally have low future maintenance; however, culverts should not be considered for waterways with a history or potential of debris to avoid channel cleanout maintenance. In these cases a three-sided precast concrete structure may be more appropriate. Three-sided precast concrete structures have the advantage of larger single and multiple openings, ease of construction, and low future maintenance costs.

A precast-concrete box culvert may be recommended by the Hydraulics Team. The side slope at the end or outcrop of a box culvert should be protected with guardrail or be located beyond the clear zone.

The hydraulic recommendations will include the Q_{100} elevation, the assumed flowline elevation, the required span, and the required waterway opening for all structure selections. The designer will determine the rise of the structure for all structure sections.

A cost comparison is required to justify a three-sided precast concrete structure compared to other bridge/culvert alternatives.

To facilitate the initiation of this type of project, the BOS is available to assist the Owners and Consultants in working out problems which may arise during plan development.

Some of the advantages of precast three-sided structures are listed below:

- Speed of Installation: Speed of installation is more dependent on excavation than product handling and placement. Precast concrete products arrive at the jobsite ready to install. Raw materials such as reinforcing steel and concrete do not need to be ordered, and no time is required on site to set up forms, place concrete, and wait for the concrete to cure. Precast concrete can be easily installed on-demand and immediately backfilled.

- Environmentally Friendly: Precast concrete is ready to be installed right off the delivery truck, which means less storage space needed for scaffolding and rebar. There is less noise pollution from ready-mix trucks continually pulling up on site and less waste as a result of using precast (i.e. no leftover steel, no pieces of scaffolding and no waste concrete piles). The natural bottom on a three-sided structure is advantageous to meet fish passage and DNR requirements.

- Quality Control: Because precast concrete products are produced in a quality-controlled environment with proper curing conditions, these products exhibit higher quality and uniformity over cast-in-place structures.

- Reduced Weather Dependency: Weather does not delay production of precast concrete as it can with cast-in-place concrete. Additionally, weather conditions at the jobsite do not significantly affect the schedule because the "window" of time required for installation is small compared to other construction methods, such as cast-in-place concrete.

- Maintenance: Single span precast three-sided structures are less susceptible to clogging from debris and sediment than multiple barrel culvers with equivalent hydraulic openings.

36.13.2.2.1 Precast Three-Sided Structure Span Lengths

WisDOT BOS allows and provides standard details for the following precast three-sided structure span lengths:

14’-0, 20’-0, 24’-0, 28’-0, 36’-0, 42’-0

Dimensions, rises, and additional guidance for each span length are provided in the standard details.

36.13.2.2.2 Segment Configuration and Skew

It is not necessary for the designer to determine the exact number and length of segments. The final structure length and segment configuration will be determined by the fabricator and may deviate from that implied by the plans.

A zero degree skew is preferable but skews may be accommodated in a variety of ways. Skew should be rounded to the nearer most-practical 5 deg., although the nearer 1 deg. is permissible where necessary. The range of skew is dependent on the design span and the fabrication limitations. Some systems are capable of fabricating a skewed segment up to a maximum of 45 degrees. Other systems accommodate skew by fabricating a special trapezoidal segment. If adequate right-of-way is available, skewed projects may be built with all right angle segments provided the angle of the wingwalls are adjusted accordingly. The designer shall consider the layout of the traffic lanes on staged construction projects when determining whether a particular three-sided precast concrete structure system is suitable.

Square segments are more economical if the structure is skewed. Laying out the structure with square segments will result in the greatest right-of-way requirement and thus allow ample space for potential redesign by the contractor, if necessary, to another segment configuration.

For a structure with a skew less than or equal to 15 deg., structure segments may be laid out square or skewed. Skewed segments are preferred for short structures (approximately less than 80 feet in length). Square segments are preferred for longer structures. However, skewed segments have a greater structural span. A structure with a skew of greater than 15 deg. requires additional analysis per the AASHTO LRFD Bridge Design Specifications. Skewed segments and the analysis both contribute to higher structure cost.

For a structure with a skew greater than 15 deg, structure segments should be laid out square. The preferred layout scheme for an arch-topped structure with a skew of greater than 15 deg should assume square segments with a sloping top of headwall to yield the shortest possible wingwalls. Where an arch-topped structure is laid out with skewed ends (headwalls parallel to the roadway), the skew will be developed within the end segments by varying the lengths of the legs as measured along the centerline of the structure. The maximum attainable skew is controlled by the difference between the full-segment leg length as recommended by the arch-topped-structure fabricator and a minimum leg length of 2 feet.

36.13.2.2.2.1 Minimum Fill Height

Minimum fill over a precast three-sided structure shall provide sufficient fill depth to allow adequate embedment for any required beam guard plus 6". Refer to Standard 36.10 for further information.

Barriers mounted directly to the precast units are not allowed, as this connection has not been crash tested.

36.13.2.2.2.2 Rise

The maximum rises of individual segments are shown on the standard details. This limit is based on the fabrication forms and transportation. The maximum rise of the segment may also be limited by the combination of the skew involved because this affects transportation on the truck. Certain rise and skew combinations may still be possible but special permits may be required for transportation. The overall rise of the three-sided structure should not be a limitation when satisfying the opening requirements of the structure because the footing is permitted to extend above the ground to meet the bottom of the three-sided segment.

36.13.2.2.2.3 Deflections

Per **LRFD [2.5.2.6.2]**, the deflection limits for precast reinforced concrete three-sided structures shall be considered mandatory.

36.13.2.3 Plans Policy

If a precast or cast-in-place three-sided culvert is used, full design calculations and plans must be provided and sealed by a professional engineer to the Bureau of Structures for approval. The plans must be in accordance with the *Bridge Manual* and Standards.

The designer should use the span and rise for the structure selection shown on the plans as a reference for the information required on the title sheet. The structure type to be shown on the Title, Layout and General Plan sheets should be Precast Reinforced Concrete Three-Sided Structure.

The assumed elevations of the top of the footing and the base of the structure leg should be shown. For preliminary structure layout purposes, a 2-foot footing thickness should be assumed with the base of the structure leg seated 2 inches below the top-of-footing elevation. With the bottom of the footing placed at the minimum standard depth of 4 feet below the flow line elevation, the base of the structure leg should therefore be shown as 2'-2" below the flow line. An exception to the 4-foot depth will occur where the anticipated footing thickness is known to exceed 2 feet, where the footing must extend to rock, or where poor soil conditions and scour concerns dictate that the footing should be deeper.

The structure length and skew angle, and the skew, length and height of wingwalls should be shown. For a skewed structure, the wingwall geometrics should be determined for each wing. The sideslope used to determine the wing length should be shown on the plans.

If the height of the structure legs exceeds 10 feet, pedestals should be shown in the structure elevation view.

The following plan requirements shall be followed:

1. Preliminary plans are required for all projects utilizing a three-sided precast concrete structure.

2. Preliminary and Final plans for three-sided precast concrete structures shall identify the size (span x rise), length, and skew angle of the bridge.

3. Final plans shall include all geometric dimensions and a detailed design for the three-sided precast structure, all cast-in-place foundation units and cast-in-place or precast wingwalls and headwalls.

4. Final plans shall include the pay item Three-Sided Precast Concrete Structure and applicable pay items for the remainder of the substructure elements.

5. Final plans shall be submitted along with all pertinent special provisions to the BOS for review and approval.

In addition to foundation type, the wingwall type shall be provided on the preliminary and final plans. Similar to precast boxes, a wingwall design shall be provided which is supported independently from the three-sided structure. The restrictions on the use of cast-in-place or precast wings and headwalls shall be based on site conditions and the preferences of the Owner. These restrictions shall be noted on the preliminary and final plans.

36.13.2.4 Foundation Requirements

Precast and cast-in-place three-sided structures that are utilized in pedestrian or cattle underpasses can be supported on continuous spread or pile supported footings. Precast and cast-in-place three-sided structures that are utilized in waterway applications shall be supported on piling to prevent scour.

The footing should be kept level if possible. If the stream grade prohibits a level footing, the wingwall footings should be laid out to be constructed on the same plane as the structure footings. Continuity shall be established between the structural unit footing and the wingwall footing.

The allowable soil bearing pressure should be shown on the plans. Weak soil conditions could require pile foundations. If the footing is on piling, the nominal driving resistance should be shown. Where a pile footing is required, the type and size of pile and the required pile spacing, and which piles are to be battered, should be shown on the plans.

The geotechnical engineer should provide planning and design recommendations to determine the most cost effective and feasible foundation treatment to be used on the preliminary plans.

36.13.2.5 Precast Versus Cast-in-Place Wingwalls and Headwalls

The specifications for three-sided precast concrete structures permits the contractor to substitute cast-in-place for precast wingwalls and headwalls, and vice versa when cast-in-place is specified unless prohibited on the plans. Three-sided structures should be provided with adequate foundation support to satisfy the design assumptions permitting their relatively thin concrete section. These foundations are designed and provided in the plans. Spread footing foundations are most commonly used since they prove cost effective when rock or scour resistant soils are present with adequate bearing and sliding resistance. The use of precast spread footings shall be controlled by the planner and shall only be allowed when soil conditions permit and shall not be allowed to bear directly on rock or when rock is within 2 feet of the bottom of the proposed footing. When lower strength soils are present, or scour depths become large, a pile supported footing shall be used. The lateral loading design of the foundation is important because deflection of the pile or footing should not exceed the manufacturers' recommendations to preclude cracks developing.

36.13.3 Metal Buried Structures

The following section provides guidance on metal buried structures. This guidance should be used in addition to the guidance provided by FDM 13-1.

Use of metal buried structures shall be evaluated on a project-by-project basis to ensure hydraulic, geotechnical, and structural criteria are satisfied. This should include a comparison of alternatives considering, but not limited to; hydraulic sizing, scour potential, costs, project schedule, and structure durability. The evaluation should then be followed by a material selection investigation for structure type justifications.

Use of metal buried structures for long spans, generally defined as spans greater than 7 ft, has been limited. The Department has experienced some corrosion issues with metal structures, which includes metal pipe failures and severe section loss. These issues are likely due to the following sources: low pH environment, low resistivity environment, active anaerobic sulfate reducing bacteria, and exposure to chlorides. While research has shown corrosion and/or abrasion concerns can be addressed to better ensure structures can satisfy their intended service life [1], <u>reinforced concrete structures are still recommended over metal structures, especially for higher volume roadways</u>. To ensure that a metal buried structure is suitable for a given site, the following criteria shall be followed:

<u>Site Investigation:</u> The geotechnical investigation shall investigate corrosion potential and abrasion classification. Document site-specific pH, resistivity, sulfate, and chloride levels of the soil and water. This information shall be used when selecting an appropriate structure material type, size, and foundation support.

<u>Design Life</u>: The minimum service life shall be 75 years.

<u>Usage:</u> Limited to lower-volume roadways (ADT < 1500), unless approved otherwise by Bureau of Structures. Not allowed on Interstate Highways or Divided US Highways.

Cover: The minimum depth of cover shall be 2 ft measured from top of pavement to top of structure. For pipe and pipe arches, refer to FDM 13-1 for maximum depth of cover. For metal box culverts, the maximum depth of cover shall be 5 ft.

Backfill: Place structural backfill equally on both sides of the structure in 8-inch maximum loose lifts. Compact all backfill to 95% of maximum dry density as determined by AASHTO T-99. Backfill shall be free draining and meet the gradation and electrochemical requirements as provided in the most current special provision bid item “Wall Concrete Panel Mechanically Stabilized Earth”.

Membrane: Provide an impervious isolation membrane that extends 10-feet beyond each side of the structure with a minimum thickness of 30 mils (ASTM 5199), regardless of the service life analysis. Membrane shall be sloped to suitable drainage with watertight seams.

Wingwalls: If wingwalls are used, a design shall be provided and supported independently from the metal structure. Metal wingwalls or headers are prohibited, unless approved otherwise by Bureau of Structures.

Guidelines for selecting material type shall be based on engineering judgement and industry practices. Refer to FDM 13-1 for additional requirements on material selection.

36.13.3.1 Metal Pipes and Pipe-Arches

FDM 13-1 provides design guidance and design fill height tables for pipe and pipe-arch shapes. This includes corrugated and structural plates sections for steel and aluminum alloy structures. These fill height tables provide a list of available sizes, minimum metal thicknesses, and depth of cover requirements. Note: the provided minimum metal thicknesses do not consider corrosive and/or abrasive conditions. Structure selection shall be evaluated on a project-by-project basis.

36.13.3.2 Other Shapes

The box culvert shape has been used on locally funded projects and may be an alternative for sites with low clearance that require a wide waterway opening. These semi-rigid structures gain strength through soil-structure interactions and flexural resistance through structural steel plates and reinforcing ribs. While the metal box culvert shape does have its benefits, corrosion concerns and the inability to inspect soil-side flexural members should be considered when selecting a structure type.

36.14 References

1. Wisconsin Highway Research Program (WHRP), *Performance and Policy Related to Aluminum Culverts in Wisconsin*, WisDOT, May 2019. Report No. 0092-17-05

36.15 Design Example

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37.1 Structure Selection

Most pedestrian bridges are located in urban areas and carry pedestrian and/or bicycle traffic over divided highways, expressways and freeway systems. The structure type selected is made on the basis of aesthetics and economic considerations. A wide variety of structure types are available and each type is defined by the superstructure used. Some of the more common types are as follows:

- Concrete Slab
- Prestressed Concrete Girder
- Steel Girder
- Prefabricated Truss

Several pedestrian bridges are a combination of two structure types such as a concrete slab approach span and steel girder center spans. One of the more unique pedestrian structures in Wisconsin is a cable stayed bridge. This structure was built in 1970 over USH 41 in Menomonee Falls. It is the first known cable stayed bridge constructed in the United States. Generally, pedestrian bridges provide the designer the opportunity to employ long spans and medium depth sections to achieve a graceful structure.

Pedestrian boardwalks should not be assigned a bridge structure (B-Structure) when their clear spans are less than or equal to 20 feet (between faces of supports). Boardwalks not meeting the B-Structure criteria will not be required to follow the design requirements in the WisDOT Bridge Manual, but will need to follow the standards established in the *Wisconsin Bicycle Facility Design Handbook* (Article 4.17.6).

Refer to 2.5 for guidance on assigning structure numbers.

37.2 Specifications and Standards

The designer shall refer to the following related specifications:

- "*AASHTO LRFD Bridge Design Specifications*"
- *"AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges",* hereafter referred to as the *"Pedestrian Bridge Guide"*
- See *Standardized Special Provision (STSP-506-085)* titled "*Prefabricated Steel Truss Pedestrian Bridge LRFD*" for the requirements for this bridge type

For additional design information, refer to the appropriate Wisconsin Bridge Manual chapters relative to the structure type selected.

The pedestrian live load (PL) shall be as follows: (from "*Pedestrian Bridge Guide")*

- 90 psf **[Article 3.1]**
- Dynamic load allowance is not applied to pedestrian live loads **[Article 3.1]**

The vehicle live load shall be applied as follows: (from *"Pedestrian Bridge Guide")*

- Design for an occasional single maintenance vehicle live load (LL) **[Article 3.2]**

Clear Bridge Width (w)	Maintenance Vehicle
7 ft $\leq$ w $\leq$ 10 ft	H5 Truck (10,000 lbs)
w > 10 ft	H10 Truck (20,000 lbs)

- Clear bridge widths of less than 7 feet need not be designed for maintenance vehicles. **[Article 3.2]**
- The maintenance vehicle live load shall not be placed in combination with the pedestrian live load. **[Article 3.2]**
- Dynamic load allowance is not applied to the maintenance vehicle. **[Article 3.2]**
- Strength I Limit State shall be used for the maintenance vehicle loading. **[Article 3.2, 3.7]**

The FHWA Pedestrian and Accessible Design guidelines and the ADA Standards for Accessible Design both recommend a limiting gradient of 8.33 percent (1:12) on ramps for pedestrian facilities to accommodate the physically handicapped and elderly.

The minimum inside clear width of a pedestrian bridge on a pedestrian accessible route is 8 feet. (*AASHTO Guide for the Planning, Design, and Operation of Pedestrian Facilities, 2004)*, (Article 3.5.3).

The width required is based on the type, volume, and direction of pedestrian and/or bicycle traffic.

The vertical clearance on the pedestrian bridge shall be a minimum of 10 feet for bicyclists' comfort and to allow access for maintenance and emergency vehicles. The Wisconsin Department of Natural Resources recommends a vertical clearance on the bridge of at least 12 feet to accommodate maintenance and snow grooming equipment on state trails. Before beginning the design of the structure, the Department of Natural Resources and the Bureau of Structures should be contacted for the vertical clearance requirements for all vehicles that require access to the bridge.

In addition, ramps should have rest areas or landings 5 feet to 6 feet in length which are level and safe. Rest area landings are mandatory when the ramp gradient exceeds 5 percent. Recommendations are that landings be spaced at 30 foot maximum intervals, as well as wherever a ramp turns. This value is based on a maximum gradient of 8.33 percent on pedestrian ramps, and limiting ramps to a maximum rise of 30 inches per ramp run. Also, ramps are required to have handrails on both sides. See Standard Details for handrail location and details.

Minimum vertical clearance for a pedestrian overpass can be found in the *Facilities Development Manual (FDM)* Procedure 11-35-1, Attachment 1.8 and 1.9. Horizontal clearance is provided in accordance with the requirement found in *(FDM)* Procedure 11-35-1, Attachment 1.5 and 1.6.

Live load deflection limits shall be in accordance with the provisions of **LRFD [2.5.2.6.2]** for the appropriate structure type.

Pedestrian loads, as described in the *AASHTO LRFD Bridge Design Specifications*, shall be used to not only design the pedestrian railings on the structure, but shall also be used to design stairway railings that are adjacent to the structure and are part of the contract.

37.3 Protective Screening

Protective Screening is recommended on all pedestrian overpasses due to the increased number of incidents where objects were dropped or thrown onto vehicles traveling below. Several types of screening material are available such as aluminum, fiberglass and plastic sheeting, and chain link type fencing. A study of the various types of protective screening available indicates that chain link fencing is the most economical and practical for pedestrian overpasses. For recommended applications refer to the Standard Details.

The top of the protective screening may be enclosed (not required) with a circular section in order to prevent objects from being thrown over the sides and to discourage people from climbing on (over) the top. The opening at the bottom is held at a 1 inch clearance to prevent objects from being pushed under the fence.

The core wire of the fence fabric shall be a minimum of 9 gauge (0.148 inch) thickness, galvanized and woven in a 2 inch mesh. A 1 inch mesh may be used in highly vulnerable areas. A vinyl coating may also be used for aesthetic purposes. Add a special provision to the contract if these additional features are used. Special provisions for common items are available as STSP's or on the Wisconsin Bridge Manual website.

Region project staff should be consulted with regards to fencing preferences.

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39.1.3 Additional Terms

Type I Sign: Larger signs on an extruded aluminum base material, typically mounted on steel I-beams. Large guide and message signs with green backgrounds on interstate routes are Type I signs.

Type II Sign: Signs consisting of direct applied message on either plywood or sheet aluminum base material, typically mounted on wood or steel posts.

Dynamic Message Sign (DMS): An electronic traffic sign, often used in urban settings to inform drivers of unique and variable information. These signs are generally smaller in wind loaded area than Type I signs, but are heavier and load the truss eccentrically.

OSS Standard Designs: A group of pre-designed sign structures. The standard design includes both the structure and its foundation. The limitations for use is provided in section 39.1.5 and 39.1.6. See for further information on OSS Standard Designs.

OSS Non-Standard Design: Refers to sign structures that fall outside the OSS Standard Design parameters. It also applies to sign structure types not covered by standard design. These sign structures require a structural engineer provide a unique individual design of the structure and/or its foundation. See 39.4.5 for further information on OSS Non-standard Designs.

OSS Contractor Designed: Refer to sign structures that are designed and detailed by the contractor as part of the construction contract. The limitations for use is provided in section 39.1.5 and 39.1.6. The contractor does not design the foundation. For this, pre-designed foundations are available for use with these types of sign structures. See 39.4.6 for further information on OSS Contractor Designed.

OSS Standard Design Drawings: Refers to a library of WisDOT developed detail drawings for the *OSS Standard Designs* and the foundations for *OSS Contractor Designed*, otherwise indicated by a “yes” in Table 39.1-1. These standard design drawings are inserted into the contract plans with no additional design or detailing effort required.

39.1.4 OSS Selection Criteria

Chapter 11-55-20 of the Facilities Development Manual (FDM) provides selection guidance for determining sign structure type. The selection guidance was developed based on the design limitations of Table 39.1-1 and Table 39.1-2 and the information provided in the OSS Standard Design Drawings.

39.1.5 Cantilever OSS Selection Criteria

Cantilever OSS Type	Design	Cantilever Length [1]	Vertical Support Height [1]	Static Sign Total Area & Max. Dimensions [2]		DMS Total Area & Weight [1]
Monotube	Contractor Designed	40'-0" Max.	25'-0" Max. Column Base Plate to CL of Monotube Arm	Sign Area ≤ 75 SF Max. Sign Height ≤ 5'-0"		Not Used
2-Chord Truss	Contractor Designed	40'-0" Max. (static) / 20'-0" Max. (DMS)	27'-0" Max. Column Base Plate to CL of Top Chord	Sign Area ≤ 150 SF Max. Sign Height ≤ 10'-0"		13'-9"W x 8'-0"H Max. 750 Lbs. Max
4-Chord Truss	Standard Design	20'-0" Min. 30'- 0" Max.[2]	30'-0" Max. Column Base Plate to CL of Top Chord	Sign Area ≤ 264 SF Max. Sign Height ≤ 15'-0"	OR	19'-0"W x 6'-0"H 2,500 Lbs. Max.
4-Chord Truss	Standard Design	>30'-0" 38'-0" Max.[2]	30'-0" Max. Column Base Plate to CL of Top Chord	Sign Area ≤ 240 SF Max. Sign Height ≤ 15'-0"		19'-0"W x 6'-0"H 2,500 Lbs. Max.
4-Chord Truss	Non-Standard Design	>38'-0"	Column Height Exceeds Limit for Standard Design	Sign Area or Max. Sign Height Exceeds Limits For Standard Design		DMS Dimensions or Weight Exceeds Limits For Standard Design

Table 39.1-2
Cantilever OSS Selection Criteria

Note 1: The limiting parameters of length, height and sign area are depicted in 39.6 for Contractor Designed OSS and on the Standard Design Drawings for the 4-Chord OSS.

Note 2: Static Type I sign panels may extend 1'-0" beyond end of Cantilever 4-Chord Truss.

39.1.6 Full Span OSS Selection Criteria

Full Span OSS Type	Design	Span Length [1]	Vertical Support Height [1]	Static Sign Total Area & Max. Dimensions		DMS Max. Dimensions & Max. Weight [1]
Monotube	Contractor Designed	40'-0" Min. 75'-0" Max.	25'-0" Max. Column Base Plate to CL of Monotube Arm	Sign Area ≤ 150 SF Max. Sign Height ≤ 5'-0"	OR	Not Used
2-Chord Truss	Contractor Designed	40'-0" Min. 100'-0" Max. (static) / 70'-0" Max. (DMS)	27'-0" Max. Column Base Plate to CL of Top Chord	150 SF < Sign Area ≤ 300 SF Max. Sign Height ≤ 10'-0"		10'-6"W x 6'-0"H Max. 850 Lbs. Max
4-Chord Truss	Standard Design	40'-0" Min. 130'-0" Max.	30'-0" Max. Column Base Plate to CL of Top Chord	300 SF < Sign Area ≤ Note 2 Max. Sign Height ≤ 12'-0"		26'-0"W x 9'-0"H 4,500 Lbs. Max.
4-Chord Truss	Non-Standard Design	>130'-0"	Column Height Exceeds Limit for Standard Design	Sign Area or Height Exceeds Limits For Standard Design		DMS Dimensions or Weight Exceeds Limits For Standard Design

Table 39.1-3
Full Span OSS Selection Criteria

Note 1: The limiting parameters of length, height and sign area are depicted in 39.6 for Contractor Designed OSS and on the Standard Design Drawings for the 4-Chord OSS.

Note 2: Maximum sign area for full span 4-chord standard design = 12' x (90% * Span Length).

39.1.7 Butterfly and Butterfly Truss OSS

OSS Type	Design	Static Sign Total Area & Max. Dimensions[2]		DMS Total Area & Weight
Butterfly	Non-Standard Design	Sign Area ≤ 240 Sq. Ft. Sign Height ≤ 10'-0"	OR	N.A.
Butterfly Truss [1]	Non-Standard Design	Sign area > 240 sq. ft. Sign Height > 10'-0"		See 4-Chord full span requirements. Limit 2 per structure.

Table 39.1-4
Butterfly and Butterfly Truss OSS Selection Criteria

Note 1: Butterfly Trusses should use the WisDOT 4-chord cantilever truss dimensions (3'-9"W x 5'-0"H). Details similar to the 4-chord cantilever should be used in the design of these structures

Note 2: The above sign areas are for one side only. Butterfly and Butterfly Truss structures can have double the total sign area listed with back to back signs mounted on each side of the structure.

39.1.8 Design Process

The design process for sign structures generally follows the process for bridge structures as detailed in chapter 6. There are some notable exceptions. First, the design of sign structures are usually initiated later in the overall process because they are dependent on a fairly established roadway plan. Second, a certain subset of sign structure types are permitted to be designed and detailed by a contractor, with other types requiring a department structural engineer (in-house or consultant) providing the design and detailing.

As outlined in 11-55-20.3 of the FDM, the Region initiates the sign structure design process by submitting to BOS an SSR. For *Contractor Designed* or *Standard Design* OSS types, as defined in 39.1.3, the Region or their consultant prepare final contract plans and submits via the structure e-submit process at least two months prior to PS&E. BOS must be notified if there are changes to the sign structure type after the SSR is submitted.

Region or consultant staff assemble final contract plans using the lead sheet templates and the *OSS Standard Design Drawings*, available on the BOS website under the Chapter 39 Bridge Standards - *LRFD Standardized Plans.* See 39.4.4 and 39.4.6 for more information on preparing standardized plans.

Involvement of a Department structural engineer in the design and detailing of individual sign structures is generally limited to *Non-standard* design types. If a Non-standard design is warranted, for reasons detailed in 39.4.5, then the design process follows the normal flow as defined in Chapter 6, requiring either BOS design staff or an engineering consultant provide a unique design and the final contract plans. Non-standard designs should make use of the OSS Standard Design Drawings where appropriate.

39.7 Design Examples

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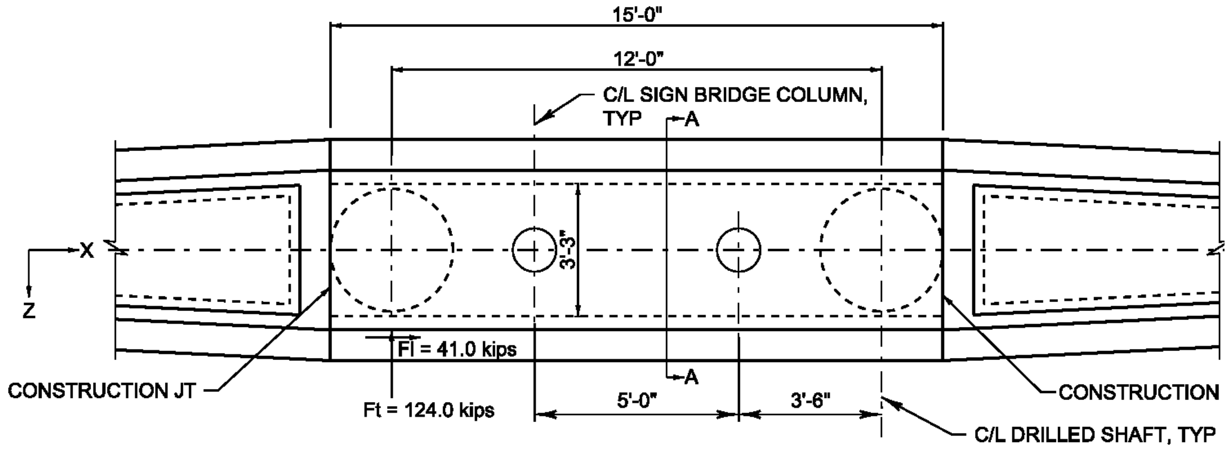
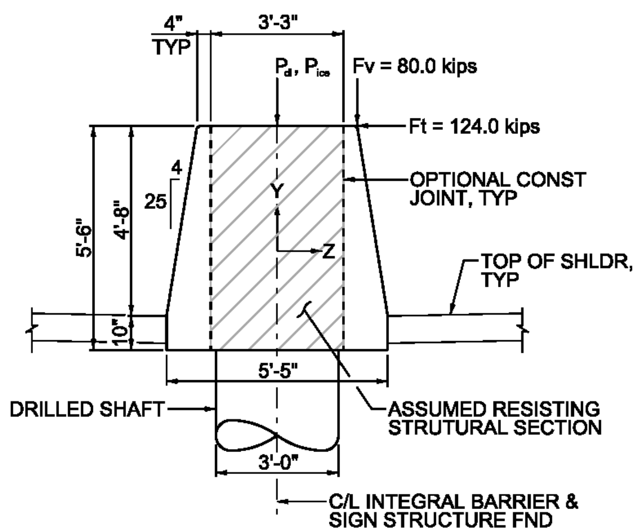
E39-1 Design of Foundation Cap Beam / Integral Barrier - TL-5 Loading

This example shows design calculations for a four chord sign bridge foundation cap beam supported on two drilled shafts that is integral with a roadway barrier. The *AASHTO LRFD Bridge Design Specifications 8th Edition - 2017* are followed for the cap beam design using a TL-5 design force for traffic railings.



PLAN



SECTION A-A

E39-1.1 Design Criteria

Cap/Intergral Barrier Material Properties

$f'_c := 3.5$	ksi	Concrete Strength
$f_y := 60$	ksi	Yield Strength of Reinforcement
$E_s := 29000$	ksi	Modulus of elasticity of steel
$w_c := 0.150$	kcf	Unit Weight of concrete

Barrier and Foundation Geometry

$H_{barrier} := 66.00$	in	Height of Barrier
$H_{barrier\_vert} := 10.00$	in	Height of Barrier Vertical Section
$W_{barrier\_top} := 47.00$	in	Width of Barrier at Top
$W_{barrier\_bott} := 65.00$	in	Width of Barrier at Bottom
$W_{barrier\_str} := 39.00$	in	Width of Barrier Structural Section
$L_{barrier} := 15.00$	ft	Length of Barrier Section
$Diam_{shaft} := 3.00$	ft	Diameter of Drilled Shaft
$Shaft\_Spa := 12.00$	ft	Spacing Between Drilled Shafts

E39-1.2 Design Forces for Traffic Railings

From **LRFD Table A13.2-1**, use Test Level Five (TL-5) design forces for integral barrier/cap check.Forces are conservatively applied as point loads instead of being distributed longitudinally along the integral barrier/cap foundation length.

$F_t := 124.0$	kips	Transverse design load
$F_L := 41.0$	kips	Longitudinal design load
$F_v := 80.0$	kips	Vertical design load (down)
$H_e := 56.0$	in	Minimum height of transverse design load = 42". Apply transverse load at top of barrier.

E39-1.3 Loads

Barrier/Cap Uniform Dead Load

Note - Uniform Dead Load is for the full area of the integral barrier including portions of the barrier outside the structural section.

$H_{barrier\_slope} := H_{barrier} - H_{barrier\_vert}$ $\boxed{H_{barrier\_slope} = 56.00}$ in

$$Area_{barrier} := \left(H_{barrier\_slope} \cdot mean\left(W_{barrier\_top}, W_{barrier\_bott}\right) \cdots + H_{barrier\_vert} \cdot W_{barrier\_bott}\right) \frac{1}{144}$$

$\boxed{Area_{barrier} = 26.292}$ ft^2

$W_{DC} := Area_{barrier} \cdot w_c$ $\boxed{W_{DC} = 3.944}$ kips/ft

Sign Structure Dead and Ice Loads - bottom of column reaction taken from SAP2000 analysis for an 82 ft span sign bridge with 30 ft column height.

$P_{dl} := 8.05$ kips $P_{ice} := 3.34$ kips

Barrier Live Load - There is no live load on the barrier since there is no live load on the sign structure.

E39-1.4 Limit States and Combinations

Limit State Extreme Event II for vehicle collision shall be applied using the following equation and load factors from **LRFD Table 3.4.1-1 & Table 3.4.1-4.**

$M_u := 1.0 \cdot DC + 0.5 \cdot LL + 1.0IC + 1.0 \cdot CT$

E39-1.5 Analysis Case I

Maximize moments in integral barrier/foundation cap by placing TL-5 loads at midspan between the drilled shafts. Assume barrier is a simply supported span between the centerlines of the drilled shafts.

Moments due to transverse forces:

$M_{y\_DC} := 0.0$ $M_{y\_LL} := 0.0$ $M_{y\_IC} := 0.0$

$M_{y\_CT} := F_t \cdot \frac{\left(L_{barrier} - Diam_{shaft}\right)}{4}$ $\boxed{M_{y\_CT} = 372.0}$ $ft \cdot kips$

$M_{uy} := 1.0M_{y\_DC} + 0.5M_{y\_LL} + 1.0 \cdot M_{y\_IC} + 1.0 \cdot M_{y\_CT}$ $\boxed{M_{uy} = 372.0}$ $ft \cdot kips$

Moments due to vertical forces:

$M_{z\_DC} := \dfrac{W_{DC} \cdot (Shaft\_Spa)^2}{8} + P_{dl} \cdot 3.5$ $\boxed{M_{z\_DC} = 99.2}$ kip·ft

$M_{z\_IC} := P_{ice} \cdot 3.5$ $\boxed{M_{z\_IC} = 11.7}$ kip·ft

$M_{z\_LL} := 0.0$ $\boxed{M_{z\_LL} = 0}$ kip·ft

$M_{z\_CT} := \dfrac{F_v \cdot Shaft\_Spa}{4}$ $\boxed{M_{z\_CT} = 240.0}$ kip·ft

$M_{uz} := 1.0M_{z\_DC} + 0.5M_{z\_LL} + 1.0 \cdot M_{z\_IC} + 1.0 \cdot M_{z\_CT}$ $\boxed{M_{uz} = 350.9}$ kip·ft

E39-1.6 Analysis Case II

Maximize shears in integral barrier/foundation cap by placing TL-5 loads at centerline of drilled shaft. Assume barrier is a simply supported span between centerline of drilled shafts.

Shears due to transverse forces:

$V_{z\_DC} := 0.0$ $V_{z\_LL} := 0.0$ $V_{z\_IC} := 0$

$V_{z\_CT} := F_t$ $\boxed{V_{z\_CT} = 124.0}$ kips

$V_{uz} := 1.0V_{z\_DC} + 0.5V_{z\_LL} + 1.0 \cdot V_{z\_IC} + 1.0 \cdot V_{z\_CT}$ $\boxed{V_{uz} = 124.0}$ kips

Shears due to vertical forces:

$V_{y\_DC} := P_{dl}$ $\boxed{V_{y\_DC} = 8.05}$ kips

$V_{y\_IC} := P_{ice}$ $\boxed{V_{y\_IC} = 3.34}$ kips

$V_{y\_LL} := 0.0$

$V_{y\_CT} := F_v$ $\boxed{V_{y\_CT} = 80.00}$ kips

$V_{uy} := 1.0V_{y\_DC} + 0.5V_{y\_LL} + 1.0 \cdot V_{y\_IC} + 1.0 \cdot V_{y\_CT}$ $\boxed{V_{uy} = 91.39}$ kips

E39-1.7 Flexural Strength Capacity

For rectangular section behavior (vertical loading):

$$c := \frac{A_s \cdot f_y}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b}$$

LRFD [5.6.2.2] $\alpha_1 := 0.85$ $\left(\text{for } f'_c < 10.0\text{ksi}\right)$

$$\beta_1 := \max\left[0.85 - \left(f'_c - 4\right)\cdot 0.05, 0.65\right] \qquad \boxed{\beta_1 = 0.875}$$

$$b := W_{barrier\_str} \qquad \boxed{b = 39.00} \text{ in}$$

The 82 ft span sign bridge with 30 ft column height standard foundation cap provides #6 bars for bottom reinforcement and #6 bar stirrups. For the vehicular collision force, which is an extreme limit event state not included in the standard foundation cap designs, it is necessary to increase the bottom reinforcement to at least 7 - #7 bars:

$A_{st\_7} := 0.60$ in^2 $\qquad Num\_bars := 7$

$$A_s := A_{st\_7} \cdot Num\_bars \qquad \boxed{A_s = 4.20} \text{ in}$$

$$c := \frac{A_s \cdot f_y}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b} \qquad \boxed{c = 2.48} \text{ in}$$

$$a := \beta_1 \cdot c \qquad \boxed{a = 2.17} \text{ in}$$

$Clr\_cov := 3.00$ in Bottom bar clear cover

$dia_6 := 0.75$ in Diameter of stirrup bars

$$d_{vert} := H_{barrier} - Clr\_cov - dia_6 - 0.5 dia_7 \qquad \boxed{d_{vert} = 61.81} \text{ in}$$

$$M_{nz} := A_s \cdot f_y \cdot \left(d_{vert} - \frac{a}{2}\right) \cdot \frac{1}{12} \qquad \boxed{M_{nz} = 1275.3} \text{ kip}\cdot\text{ft}$$

For reinforced concrete sections:

$\phi_f := 0.9$ **LRFD [5.5.4.2].** Therefore, the factored flexural resistance is:

$$M_{rz} := \phi_f \cdot M_{nz} \qquad \boxed{M_{rz} = 1147.7} \text{ kip}\cdot\text{ft}$$

For rectangular section behavior (transverse loading):

$b := H_{barrier}$ $\boxed{b = 66.00}$ in

Assume side reinforcement is #6 bars and stirrups are #6 bars:

$A_{st\_6} := 0.44$ in^2 $Num\_bars := 8$

$A_s := A_{st\_6} \cdot Num\_bars$ $\boxed{A_s = 3.52}$ in

$$c := \frac{A_s \cdot f_y}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b}$$ $\boxed{c = 1.23}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 1.08}$ in

$Clr\_cov := 2.00$ in Side bar clear cover

$dia_6 := 0.75$ in Diameter of stirrup/side bars

$d_{horiz} := W_{barrier\_str} - Clr\_cov - dia_6 - 0.5dia_6$ $\boxed{d_{horiz} = 35.88}$ in

$$M_{ny} := A_s \cdot f_y \cdot \left(d_{horiz} - \frac{a}{2}\right) \cdot \frac{1}{12} = 621.934$$ $\boxed{M_{ny} = 621.9}$ kip·ft

For reinforced concrete sections:

$\phi_f := 0.9$ **LRFD [5.5.4.2].** Therefore, the factored flexural resistance is:

$M_{ry} := \phi_f \cdot M_{ny}$ $\boxed{M_{ry} = 559.7}$ kip·ft

If the factored axial load is less than $\phi_c f'_c A_g$: **LRFD [5.6.4.5]**

$$\frac{M_{uy}}{M_{ry}} + \frac{M_{uz}}{M_{rz}} < 1.00$$ $$\frac{M_{uy}}{M_{ry}} + \frac{M_{uz}}{M_{rz}} = 0.97$$

Is 0.97 < 1.0 ? **Yes** $\boxed{check = OK}$

E39-1.8 Shear Capacity

For rectangular section behavior (vertical loading):

The nominal shear resistance of the section is calculated as follows, **LRFD [5.7.3.3]**:

$$V_n := \min\left(V_c + V_s + V_p, 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p\right)$$

The nominal shear of the concrete is calculated as follows:

$$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$$

$\beta := 2$ Simplified procedure **LRFD 5.7.3.4.1**

$\lambda := 1$ Concrete density modification factor **LRFD 5.4.2.8**

$b_v := b$ $\boxed{b_v = 66.00}$ in

$Clr\_cov := 3.00$ in Bottom bar clear cover

Determine effective shear depth, dv:

For non-prestressed sections:

$d_e := d_{vert}$ **LRFD 5.7.2.8-2** $d_e = 61.81$ in

dv is the maximum of the following three equations: **LRFD 5.7.2.8**

$d_{v1} := d_{vert} - \frac{a_{vert}}{2}$ $\boxed{d_{v1} = 60.73}$ in

$d_{v2} := 0.9 \cdot d_e$ $\boxed{d_{v2} = 55.63}$ in

$d_{v3} := 0.72 \cdot H_{barrier}$ $\boxed{d_{v3} = 47.52}$ in

$d_v := \max\left(d_{v1}, d_{v2}, d_{v3}\right)$ $\boxed{d_v = 60.73}$ in

$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$ $\boxed{V_c = 473.9}$ kips

The shear resistance provided by transverse reinforcement

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{s}$$

$\theta := 45$ deg Simplified procedure **LRFD 5.7.3.4.1**

$A_v := 0.88$ in^2 #6 stirrups (2 legs)

$s := 6.00$ in Stirrup spacing

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot\left(\theta \cdot \frac{\pi}{180}\right)}{s} \qquad \boxed{V_s = 534.4} \text{ kips}$$

$$V_{n1} := V_c + V_s + V_p \qquad \boxed{V_{n1} = 1008.3} \text{ kips}$$

$$V_{n2} := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p \qquad \boxed{V_{n2} = 3507.0} \text{ kips}$$

$$V_n := \min\left(V_{n1}, V_{n2}\right) \qquad \boxed{V_n = 1008.3} \text{ kips}$$

For reinforced concrete sections:

$\phi_v := 0.9$ **LRFD [5.5.4.2].** Therefore, the factored shear resistance is:

$$V_{ry} := \phi_v \cdot V_n \qquad \boxed{V_{ry} = 907.5} \text{ kips}$$

For rectangular section behavior (transverse loading):

The nominal shear resistance of the section is calculated as follows, **LRFD [5.7.3.3]**:

$$V_n := \min\left(V_c + V_s + V_p, 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p\right)$$

The nominal shear of the concrete is calculated as follows:

$$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$$

$\beta := 2$ Simplified procedure **LRFD 5.7.3.4.1**

$\lambda := 1$ Concrete density modification factor **LRFD 5.4.2.8**

$b_v := b$ $\boxed{b_v = 66.00}$ in

$Clr\_cov := 2.00$ in Side bar clear cover

Determine effective shear depth, dv:

For non-prestressed sections:

$d_e := d_{horiz}$ **LRFD 5.7.2.8-2** $d_e = 35.88$ in

dv is the maximum of the following three equations: **LRFD 5.7.2.8**

$$d_{v1} := d_{horiz} - \frac{a_{horiz}}{2} \qquad \boxed{d_{v1} = 35.34} \text{ in}$$

$$d_{v2} := 0.9 \cdot d_e \qquad \boxed{d_{v2} = 32.29} \text{ in}$$

$$d_{v3} := 0.72 \cdot W_{barrier\_str} \qquad \boxed{d_{v3} = 28.08} \text{ in}$$

$d_v := \max(d_{v1}, d_{v2}, d_{v3})$ $\boxed{d_v = 35.34}$ in

$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$ $\boxed{V_c = 275.8}$ kips

The shear resistance provided by transverse reinforcement

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{s}$$

$\theta := 45$ deg Simplified procedure **LRFD 5.7.3.4.1**

$A_v := 0.88$ in^2 #6 stirrups (2 legs)

$s := 6.00$ in Stirrup spacing

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot\left(\theta \cdot \frac{\pi}{180}\right)}{s}$$ $\boxed{V_s = 311.0}$ kips

$V_{n1} := V_c + V_s + V_p$ $\boxed{V_{n1} = 586.7}$ kips

$V_{n2} := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$ $\boxed{V_{n2} = 2040.7}$ kips

$V_n := \min(V_{n1}, V_{n2})$ $\boxed{V_n = 586.7}$ kips

For reinforced concrete sections:

$\phi_v := 0.90$ **LRFD [5.5.4.2].** Therefore, the factored shear resistance is:

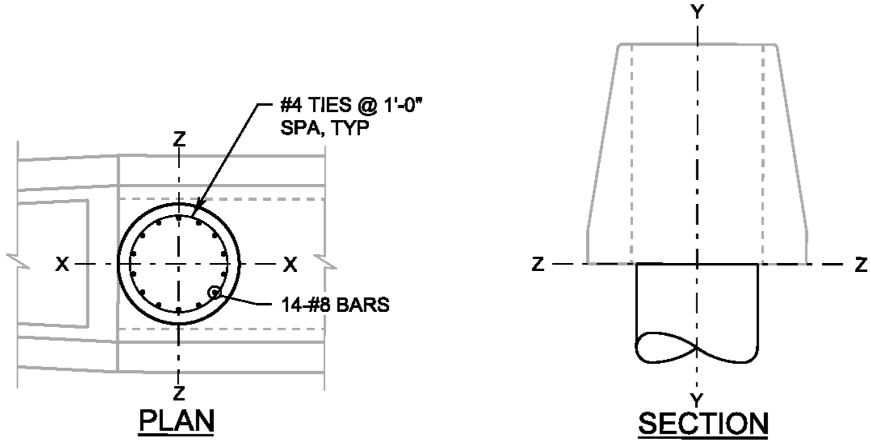
$V_{rz} := \phi_v \cdot V_n$ $\boxed{V_{rz} = 528.1}$ kips

Check combined shear::

$$\frac{V_{uy}}{V_{ry}} + \frac{V_{uz}}{V_{rz}} < 1.0$$

$$\frac{V_{uz}}{V_{rz}} + \frac{V_{uz}}{V_{rz}} = 0.47$$ Is 0.47 < 1.0 ? **Yes** $\boxed{check = OK}$

E39-1.9 Check Cap Beam/Top of Drilled Shaft Interface



Check Case II - TL-5 Loading at C/L of drilled shaft, this develops the maximum moment and shear at the top of the drilled shaft

$F_x := F_L$ $\boxed{F_x = 41.0}$ kips

$F_y := 0.5 \cdot W_{DC} \cdot L_{barrier} + F_v$ $\boxed{F_y = 109.6}$ kips

$F_z := F_t$ $\boxed{F_z = 124.0}$ kips

$M_x := F_z \cdot H_{barrier} \cdot \frac{1}{12}$ Conservatively ignore vertical load $\boxed{M_x = 682.0}$ kip·ft

$M_y := F_z \cdot H_{barrier} \cdot \frac{1}{12}$ $\boxed{M_y = 682.0}$ kip·ft

$M_z := F_y \cdot (0.5 \cdot W_{barrier\_top}) \cdot \frac{1}{12}$ $\boxed{M_z = 214.6}$ kip·ft

Check shear resistance:

Assume shaft reinforcement is #8 bars vertical with #4 ties:

$dia_4 := 0.50$ in $dia_8 := 1.00$ in $clr\_cov := 3.50$ in

The nominal shear resistance of the section is calculated as follows, **LRFD [5.7.3.3]**:

$$V_n := \min(V_c + V_s + V_p, 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p)$$

The nominal shear of the concrete is calculated as follows:

$$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$$

$\beta := 2$ Simplified procedure **LRFD 5.7.3.4.1**

$\lambda := 1$ Concrete density modification factor **LRFD 5.4.2.8**

$b_v := Diam_{shaft} \cdot 12$ $\boxed{b_v = 36.00}$ in

$$d_e := \frac{Diam_{shaft} \cdot 12}{2} + \frac{\left[Diam_{shaft} \cdot 12 - 2\left(clr\_cov + dia_4\right) - dia_8\right]}{\pi}$$ $\boxed{d_e = 26.59}$ in

$d_v := 0.9 d_e$ Effective shear depth **LRFD C5.7.2.8-2** $\boxed{d_v = 23.93}$ in

$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$ $\boxed{V_c = 101.9}$ kips

The shear resistance provided by transverse reinforcement

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{s}$$

$\theta := 45$ deg Simplified procedure **LRFD 5.7.3.4.1**

$A_v := 0.40$ in^2 #4 ties (2 legs)

$s := 12.00$ in Stirrup spacing

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot\left(\theta \cdot \frac{\pi}{180}\right)}{s}$$ $\boxed{V_s = 47.9}$ kips

$V_{n1} := V_c + V_s + V_p$ $\boxed{V_{n1} = 150.7}$ kips

$V_{n2} := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$ $\boxed{V_{n2} = 755.0}$ kips

$V_n := \min\left(V_{n1}, V_{n2}\right)$ $\boxed{V_n = 150.7}$ kips

For reinforced concrete sections:

$\phi_v := 0.9$ **LRFD [5.5.4.2].** Therefore, the factored shear resistance is:

$V_r := \phi_v \cdot V_n$ $\boxed{V_r = 135.7}$ kips

$V_u := \sqrt{F_x^{\,2} + F_z^{\,2}}$ $\boxed{V_u = 130.6}$ k

Is V_u = 130.6 kips < V_r = 142.0 kips? **Yes** $\boxed{check = OK}$

Check the top of drilled shaft as a reinforced concrete column:

The assessment of the resistance of a compression member with biaxial flexure is dependent upon the magnitude of the factored axial load. If the factored axial load is less than ten percent of the gross concrete strength multiplied by the phi-factor for compression members, then use Equation 5.6.4.5-3. Otherwise, use Equation 5.7.4.5-1. Regardless of which equation in the above paragraph controls, commercially available software is generally used to obtain the moment and axial load resistances.

The procedure as discussed above is carried out as follows:

$\phi := 0.75$ **LRFD [5.5.4.2].** Therefore, the factored shear resistance is:

$$A_g := \frac{\pi \cdot (Diam_{shaft} \cdot 12)^2}{4} \qquad \boxed{A_g = 1017.9} \ in^2$$

$0.10 \cdot \phi \cdot f'_c \cdot A_g = 267.2$ kips

P_z = 89.1 kips < 305.4 kips Therefore, use **LRFD [Equation 5.6.4.5-3]**

$$M_{uy} := M_y \qquad \boxed{M_{uy} = 682.0} \ kip \cdot ft$$

$$M_{uz} := M_z \qquad \boxed{M_{uz} = 214.6} \ kip \cdot ft$$

$$M_u := \sqrt{M_{uy}^2 + M_{uz}^2} \qquad \boxed{M_u = 715.0} \ kip \cdot ft$$

$M_r := 723.1$ kip·ft

$$\frac{M_u}{M_r} = 0.99$$ Is 0.99 < 1.0 ? **Yes** $\boxed{check = OK}$

The factored flexural resistances shown above, M_r, was obtained by the use of commercial software.

E39-1.10 Interface Shear Transfer

Check interface shear capacity across construction joint between transition barrier section and foundation cap per **LRFD 5.7.4**.

Calculate factored interface shear force due to TL-5 vehicular collision forces only:

$$V_{CT} := \left(V_{z\_CT}^{\,2} + V_{y\_CT}^{\,2}\right)^{0.5} \qquad \boxed{V_{CT} = 148} \text{ Kips}$$

Vehicle collision force is extreme event limit state, therefore load factor = 1.0:

$$V_{ui} := 1.0 \cdot V_{CT} \qquad \boxed{V_{ui} = 148} \text{ Kips}$$

Calculate interface shear resistance. For purpose of determining shear transfer contact area, use gross combined area of resisting foundation cap section and integral barriers.

$$A_{cv} := Area_{barrier} \cdot 144 \qquad \boxed{A_{cv} = 3786} \text{ in}^2$$

Per SDD-14B32 the standard barrier transition section has 6 - #5 horizontal bars on each face continuing across the interface construciton joint between the barrier transition and foundation cap sections.

$A_{st\_5} := 0.31$ in^2 Area of #5 bar

Area of shear reinforcement crossing the shear plane

$$A_{vf} := 2 \cdot 6 \cdot A_{st\_5} \qquad \boxed{A_{vf} = 3.72} \text{ in}^2$$

Assume clean construction joint, not intentionally roughened. Per **LRFD 5.7.4.3**:

$$c_{cv} := 0.075$$
$$\mu := 0.6$$
$$K_1 := 0.2$$
$$K_2 := 0.8$$

Permanent axial compression across shear interface = 0

The nominal shear interface (shear friction) capacity is the smallest of following three equations:

$$V_{nsf1} := c_{cv} \cdot A_{cv} + \mu \cdot A_{vf} \cdot f_y \qquad \textbf{LRFD 5.7.4.3-3} \qquad \boxed{V_{nsf1} = 417.87} \text{ Kips}$$

$$V_{nsf2} := K_1 \cdot f'_c \cdot A_{cv} \qquad \textbf{LRFD 5.7.4.3-4} \qquad \boxed{V_{nsf2} = 2650.2} \text{ Kips}$$

$$V_{nsf3} := K_2 \cdot A_{cv} \qquad \textbf{LRFD 5.7.4.3-5} \qquad \boxed{V_{nsf3} = 3028.8} \text{ Kips}$$

Nominal shear interface (shear friction) capacity:

$V_{nsf} := \min(V_{nsf1}, V_{nsf2}, V_{nsf3})$ $\boxed{V_{nsf} = 417.87}$ Kips

Factored shear interface resistance; for extreme event loading:

$\phi_{si} := 1.0$ **LRFD 5.7.4.3**

Therefore, the factored interface shear resistance is:

$V_{ri} := \phi_{si} \cdot V_{nsf}$ $\boxed{V_{ri} = 417.87}$ Kips

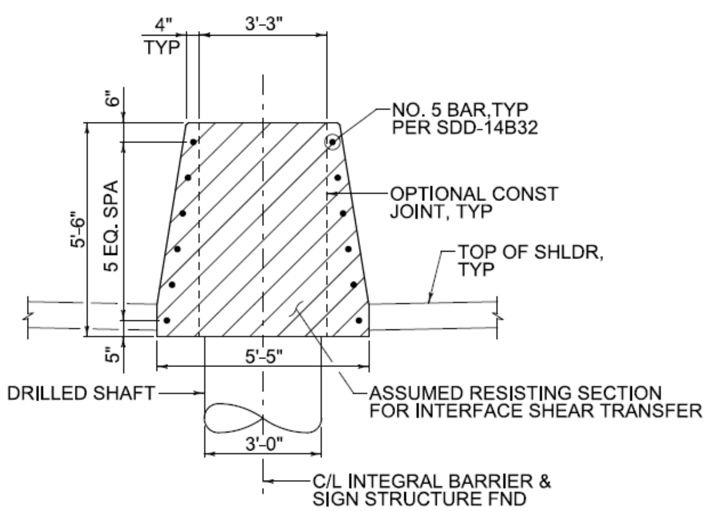
Is $V_{ui} < V_{ri}$ = 417.87 kips? **Yes** $\boxed{\text{check} = \text{OK}}$

Check that minimum shear interface reinforcement is provided per LRFD 5.7.4.2:

$A_{vf\_min} := \dfrac{0.05 \cdot A_{cv}}{f_y}$ **LRFD 5.7.4.2-1** $\boxed{A_{vf\_min} = 3.15}$ in^2

Is $A_{vf\_min} < A_{vf}$ = 3.72 in^2 **Yes** $\boxed{\text{check} = \text{OK}}$

Summary: Shear interface reinforcment of 12 - #5 bars per SDD-14B32 is adequate.



INTEGRAL BARRIER SECTION

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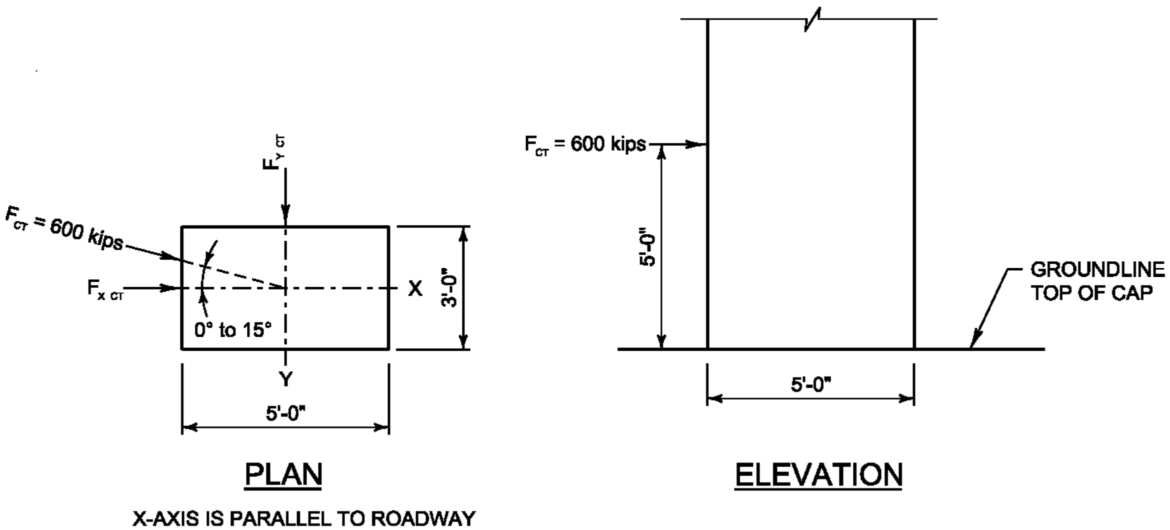
E39-2 Design of Sign Bridge Concrete Column for Vehicle Impact

This example shows design calculations for a four chord sign bridge concrete column supported on a concrete foundation cap beam that is impacted by a vehicular collision force. The *AASHTO LRFD Bridge Design Specifications 8th Edition - 2017* are followed for the column design assuming the equivalent static force acts in a direction of zero to 15 degrees with the edge of pavement in a horizontal plane.



E39-2.1 Design Criteria

Column Material Properties

$f'_{c\_col} := 3.5$ ksi	Concrete Strength
$f_y := 60$ ksi	Yield Strength of Reinforcement
$E_s := 29000$ ksi	modulus of elasticity of steel
$w_c := 0.150$ kcf	Unit Weight of concrete

Footing Material Properties

$f'_{c\_ftg} := 3.5$ ksi	Concrete Strength

Column Geometry

$W_{col} := 3.00$ ft	Width of Column
$L_{col} := 5.00$ ft	Length of Column at Base

Footing Geometry

$W_{ftg} := 3.25$ ft	Width of Footing
$L_{ftg} := 12.00$ ft	Length of Footing

E39-2.2 Vehicular Collision Force

$F_{CT} := 600.0$ kips Vehicular impact design force **[LRFD 3.6.5.1]**

$H_{CT} := 5.00$ ft Height of vehicular impact design force above ground **[LRFD 3.6.5.1]**

Equivalent static force is assumed to act in a direction of zero to 15 degrees with the edge of the pavement. Two load cases will be analyzed:

Case I - Angle of Force = 15 deg
Case II - Angle of Force = 0 deg

E39-2.3 Limit States and Combinations

Limit State Extreme Event II for vehicle collision shall be applied using the following equation and load factors from **LRFD Table 3.4.1-1 & Table 3.4.1-4.**

$$V_u := 1.0 \cdot DC + 0.5 \cdot LL + 1.0IC + 1.0 \cdot CT$$

$$M_u := 1.0 \cdot DC + 0.5 \cdot LL + 1.0IC + 1.0 \cdot CT$$

E39-2.4 Analysis

Sign bridge column will be analyzed as a cantilever fixed at the column base.

Case I - Equivalent Static Load acting at 15 deg with edge of pavement

$V_{x\_DC} := 0.0$ $V_{x\_LL} := 0.0$ $V_{x\_IC} := 0.0$

$$V_{x\_CT} := F_{CT} \cdot \cos\left(15 \cdot \frac{\pi}{180}\right) \qquad \boxed{V_{x\_CT} = 579.6} \text{ kips}$$

$$V_{ux} := 1.0V_{x\_DC} + 0.5V_{x\_LL} + 1.0 \cdot V_{x\_IC} + 1.0 \cdot V_{x\_CT} \qquad \boxed{V_{ux} = 579.6} \text{ kips}$$

$V_{y\_DC} := 0.0$ $V_{y\_LL} := 0.0$ $V_{y\_IC} := 0.0$

$$V_{y\_CT} := F_{CT} \cdot \sin\left(15 \cdot \frac{\pi}{180}\right) \qquad \boxed{V_{y\_CT} = 155.3} \text{ kips}$$

$$V_{uy} := 1.0V_{y\_DC} + 0.5V_{y\_LL} + 1.0 \cdot V_{y\_IC} + 1.0 \cdot V_{y\_CT} \qquad \boxed{V_{uy} = 155.3} \text{ kips}$$

$M_{x\_DC} := 0.0$ $M_{x\_LL} := 0.0$ $M_{x\_IC} := 0.0$

$$M_{x\_CT} := \left(F_{CT} \cdot \sin\left(15 \cdot \frac{\pi}{180}\right)\right) \cdot H_{CT} \qquad \boxed{M_{x\_CT} = 776.5} \text{ kip} \cdot \text{ft}$$

$$M_{ux} := 1.0M_{x\_DC} + 0.5M_{x\_LL} + 1.0 \cdot M_{x\_IC} + 1.0 \cdot M_{x\_CT} \qquad \boxed{M_{ux} = 776.5} \text{ kip} \cdot \text{ft}$$

$M_{y\_DC} := 0.0$ $M_{y\_LL} := 0.0$ $M_{y\_IC} := 0.0$

$$M_{y\_CT} := F_{CT} \cdot H_{CT} \cdot \cos\left(15 \cdot \frac{\pi}{180}\right)$$ $\boxed{M_{y\_CT} = 2897.8}$ kip·ft

$$M_{uy} := 1.0M_{y\_DC} + 0.5M_{y\_LL} + 1.0 \cdot M_{y\_IC} + 1.0 \cdot M_{y\_CT}$$ $\boxed{M_{uy} = 2897.8}$ kip·ft

E39-2.5 Flexural Strength Capacity

For rectangular section behavior (longitudinal loading):

$$c := \frac{A_s \cdot f_y}{\alpha_1 \cdot f'_{c\_col} \cdot \beta_1 \cdot b}$$

LRFD [5.6.2.2] $\alpha_1 := 0.85$ $\left(\text{for } f'_c < 10.0\text{ksi}\right)$

$$\beta_1 := \max\left[0.85 - \left(f'_{c\_col} - 4\right) \cdot 0.05, 0.65\right]$$ $\boxed{\beta_1 = 0.875}$

$$b := W_{col} \cdot 12$$ $\boxed{b = 36.00}$ in

It is assumed that bundled #11 bars are used for the column vertical reinforcement. The bars are fully developed at the bottom of the column by utilizing standard 180 degree hooks.

Try: Bar size #11 $A_{st\_11} := 1.56$ in^2 $Num\_bars := 12$

$$A_s := A_{st\_11} \cdot Num\_bars$$ $\boxed{A_s = 18.72}$ in

$$c := \frac{A_s \cdot f_y}{\alpha_1 \cdot f'_{c\_col} \cdot \beta_1 \cdot b}$$ $\boxed{c = 11.99}$ in

$$a := \beta_1 \cdot c$$ $\boxed{a = 10.49}$ in

$Clr\_cov := 2.50$ in Column tie clear cover

$dia_5 := 0.625$ in Diameter of tie bars

$dia_{11} := 1.41$ in Diameter of vertical column bars

$$d_{long} := L_{col} \cdot 12 - Clr\_cov - dia_5 - dia_{11}$$ $\boxed{d_{long} = 55.47}$ in

$$M_{ny} := A_s \cdot f_y \cdot \left(d_{long} - \frac{a}{2}\right) \cdot \frac{1}{12}$$ $\boxed{M_{ny} = 4700.7}$ kip·ft

For reinforced concrete sections:

$\phi_f := 0.9$ **LRFD [5.5.4.2].** Therefore, the factored flexural resistance is:

$$M_{ry} := \phi_f \cdot M_{ny}$$ $\boxed{M_{ry} = 4230.6}$ kip·ft

For rectangular section behavior (transverse loading):

$b := L_{col} \cdot 12$ $\boxed{b = 60.00}$ in

Try: Bar size #11 $A_{st\_11} := 1.56$ in^2 $Num\_bars := 14$

$A_s := A_{st\_11} \cdot Num\_bars$ $\boxed{A_s = 21.84}$ in

$$c := \frac{A_s \cdot f_y}{\alpha_1 \cdot f'_{c\_col} \cdot \beta_1 \cdot b}$$ $\boxed{c = 8.39}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 7.34}$ in

$d_{tran} := W_{col} \cdot 12 - Clr\_cov - dia_5 - dia_{11}$ $\boxed{d_{tran} = 31.46}$ in

$$M_{nx} := A_s \cdot f_y \cdot \left(d_{tran} - \frac{a}{2}\right) \cdot \frac{1}{12}$$ $\boxed{M_{nx} = 3035.1}$ $kip \cdot ft$

For reinforced concrete sections:

$\phi_f := 0.9$ **LRFD [5.5.4.2].** Therefore, the factored flexural resistance is:

$M_{rx} := \phi_f \cdot M_{nx}$ $\boxed{M_{rx} = 2731.6}$ $kip \cdot ft$

If the factored axial load is less than $\phi_c f'_c A_g$: **LRFD [5.6.4.5]**

$$\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} < 1.00$$

$$\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} = 0.97$$

Is 0.97 ≤ 1.0 ? **Yes** $\boxed{check = OK}$

E39-2.6 Shear Capacity

Compute shear resistance in the longitudinal direction (V_{rx}):

The nominal shear resistance of the section is calculated as follows, **LRFD [5.7.3.3]**:

$$V_n := \min\left(V_c + V_s + V_p, 0.25 \cdot f'_{c\_col} \cdot b_v \cdot d_v + V_p\right)$$

The nominal shear of the concrete is calculated as follows:

$$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_{c\_col}} \cdot b_v \cdot d_v$$

$\beta := 2$ Simplified procedure **LRFD 5.7.3.4.1**

$\lambda := 1$ Concrete density modification factor **LRFD 5.4.2.8**

$b_v := b$ $\boxed{b_v = 60.00}$ in

Determine effective shear depth, dv:

For non-prestressed sections:

$d_e := d_{long}$ **LRFD 5.7.2.8-2** $d_e = 55.47$ in

dv is the maximum of the following three equations: **LRFD 5.7.2.8**

$d_{v1} := d_{long} - \frac{a_{long}}{2}$ $\boxed{d_{v1} = 50.22}$ in

$d_{v2} := 0.9 \cdot d_e$ $\boxed{d_{v2} = 49.92}$ in

$d_{v3} := 0.72 \cdot L_{col} \cdot 12$ $\boxed{d_{v3} = 43.20}$ in

$d_v := \max(d_{v1}, d_{v2}, d_{v3})$ $\boxed{d_v = 50.22}$ in

$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_{c\_col}} \cdot b_v \cdot d_v$ $\boxed{V_c = 356.3}$ kips

The shear resistance provided by transverse reinforcement

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{s}$$

deg 45 Simplified procedure **LRFD 5.7.3.4.1**

$A_v := 1.24$ in^2 #5 double stirrups (4 legs of stirrups)

$s := 6.0$ in Stirrup spacing

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot\left(\theta \cdot \frac{\pi}{180}\right)}{s}$$ $\boxed{V_s = 622.7}$ kips

$V_{n1} := V_c + V_s + V_p$ $\boxed{V_{n1} = 979.0}$ kips

$V_{n2} := 0.25 \cdot f'_{c\_col} \cdot b_v \cdot d_v + V_p$ $\boxed{V_{n2} = 2636.6}$ kips

$V_n := \min(V_{n1}, V_{n2})$ $\boxed{V_n = 979.0}$ kips

For reinforced concrete sections:

$\phi_v := 0.90$ **LRFD [5.5.4.2].** Therefore, the factored shear resistance is:

$V_{rx} := \phi_v \cdot V_n$ $\boxed{V_{rx} = 881.1}$ kips

Compute shear resistance in the transverse direction (V_{ry}):

The nominal shear resistance of the section is calculated as follows, **LRFD [5.7.3.3]**:

$$V_n := \min\left(V_c + V_s + V_p, 0.25 \cdot f'_{c\_col} \cdot b_v \cdot d_v + V_p\right)$$

The nominal shear of the concrete is calculated as follows:

$$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_{c\_col}} \cdot b_v \cdot d_v$$

$\beta := 2$ Simplified procedure **LRFD 5.7.3.4.1**

$\lambda := 1$ Concrete density modification factor **LRFD 5.4.2.8**

$b_v := b$ $\boxed{b_v = 60.00}$ in

Determine effective shear depth, dv:

For non-prestressed sections:

$d_e := d_{tran}$ **LRFD 5.7.2.8-2** $d_e = 31.46$ in

dv is the maximum of the following three equations: **LRFD 5.7.2.8**

$d_{v1} := d_{tran} - \dfrac{a_{tran}}{2}$ $\boxed{d_{v1} = 27.79}$ in

$d_{v2} := 0.9 \cdot d_e$ $\boxed{d_{v2} = 28.32}$ in

$d_{v3} := 0.72 \cdot W_{col} \cdot 12$ $\boxed{d_{v3} = 25.92}$ in

$d_v := \max\left(d_{v1}, d_{v2}, d_{v3}\right)$ $\boxed{d_v = 28.32}$ in

$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_{c\_col}} \cdot b_v \cdot d_v$ $\boxed{V_c = 200.9}$ kips

The shear resistance provided by transverse reinforcement

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{s}$$

$\theta := 45$ deg Simplified procedure **LRFD 5.7.3.4.1**

$A_v := 1.24$ in^2 #5 double stirrups (4 legs of stirrups)

$s := 6.0$ in Stirrup spacing

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot\left(\theta \cdot \frac{\pi}{180}\right)}{s}$$ $\boxed{V_s = 351.1}$ kips

$V_{n1} := V_c + V_s + V_p$ $\boxed{V_{n1} = 552.0}$ kips

$V_{n2} := 0.25 \cdot f'_{c\_col} \cdot b_v \cdot d_v + V_p$ $\boxed{V_{n2} = 1486.7}$ kips

$V_n := \min\left(V_{n1}, V_{n2}\right)$ $\boxed{V_n = 552.0}$ kips

For reinforced concrete sections:

$\phi_v := 0.90$ **LRFD [5.5.4.2].** Therefore, the factored shear resistance is:

$V_{ry} := \phi_v \cdot V_n$ $\boxed{V_{ry} = 496.8}$ kips

Check combined shear::

$$\frac{V_{uy}}{V_{ry}} + \frac{V_{uz}}{V_{rz}} < 1.0 \qquad \frac{V_{ux}}{V_{rx}} + \frac{V_{uy}}{V_{ry}} = 0.97$$

Is 0.97 $\leq$ 1.0 ? **Yes** $\boxed{check = OK}$

E39-2.7 Analysis and Design Check for Case II Loading

Case II - Equivalent Static Load acting at 0 deg with edge of pavement

$V_{x\_DC} := 0.0$ $V_{x\_LL} := 0.0$ $V_{x\_IC} := 0.0$

$V_{x\_CT} := F_{CT} \cdot \cos\left(0 \cdot \frac{\pi}{180}\right)$ $\boxed{V_{x\_CT} = 600.0}$ kips

$V_{ux} := 1.0V_{x\_DC} + 0.5V_{x\_LL} + 1.0 \cdot V_{x\_IC} + 1.0 \cdot V_{x\_CT}$ $\boxed{V_{ux} = 600.0}$ kips

$M_{y\_DC} := 0.0$ $M_{y\_LL} := 0.0$ $M_{y\_IC} := 0.0$

$M_{y\_CT} := F_{CT} \cdot H_{CT} \cdot \cos\left(0 \cdot \frac{\pi}{180}\right)$ $\boxed{M_{y\_CT} = 3000.0}$ kip·ft

$M_{uy} := 1.0M_{y\_DC} + 0.5M_{y\_LL} + 1.0 \cdot M_{y\_IC} + 1.0 \cdot M_{y\_CT}$ $\boxed{M_{uy} = 3000.0}$ kip·ft

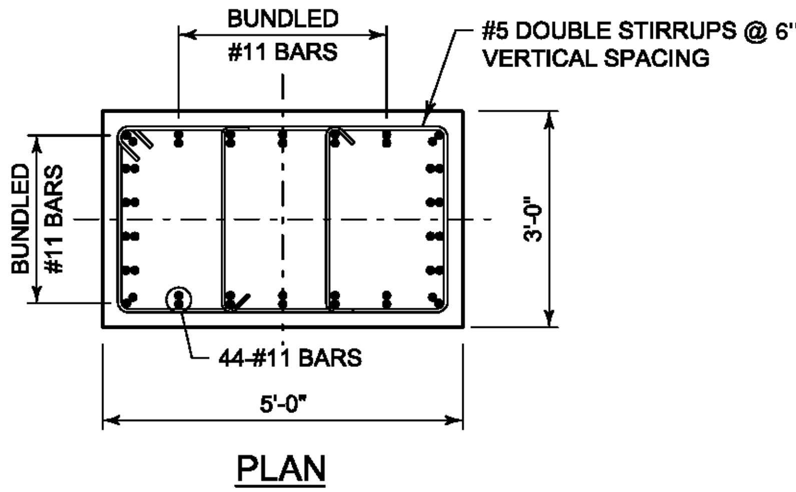
Check Shear:

$V_{rx} = 881.1$ kips Is 940.4 > 600.0 ? **Yes** $\boxed{check = OK}$

Check Moment:

$M_{ry} = 4230.6$ kip·ft Is 4285.9 > 3000.0 ? **Yes** $\boxed{check = OK}$

E39-2.8 Summary Sketch



E39-2.9 Column to Foundation Cap Interface Shear Check

Confirm the shear capacity at the column to foundation cap interface per **LRFD 5.7.4**.

Refer to **E13-1.9.3** for an example of this calculation. Following this example calculation the factored interface shear resistance is determined to be 1,512 kips with ϕ = 1.0 for the extreme limit state per **LRFD 5.7.4.3**. This far exceeds the factored shear force V_u = 600 kips due to the vehicular collision force and therefore the column to foundation cap interface shear capacity is adequate.

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40.1 General

New bridges are designed for a minimally expected life of 75 years. Preliminary design considerations are site conditions, structure type, geometrics, and safety. Refer to Bridge Manual Chapters 9 and 17 for Materials and Superstructure considerations, respectively. Comprehensive specifications and controlled construction inspection are paramount to obtaining high quality structures. Case history studies show that adequately consolidated and properly cured concrete with low water-cement ratios and good air void systems have lower absorption rates and provide greater resistance to scaling and chloride penetration under heavy traffic and exposure to de-icing chemicals. Applying protective surface treatments to new decks improves their resistance to first year applications of de-icing chemicals.

Most interstate and freeway structures are not subject to normal conditions and traffic volumes. Under normal environmental conditions and traffic volumes, original bridge decks have an expected life of 40 years. Deck deterioration is related to the deck environment which is usually more severe than for any of the other bridge elements. Decks are subjected to the direct effects of weather, the application of chemicals and/or abrasives, and the impact of vehicular traffic. For unprotected bar steel, de-icing chemicals are the primary cause of accelerated bridge deck deterioration. Chlorides cause the steel to corrode and the corrosion expansion causes concrete to crack along the plane of the top steel. Traffic breaks up the delaminated concrete leaving potholes on the deck surfaces. In general, deck rehabilitation on Wisconsin bridges has occurred after 15 to 22 years of service due to abnormally high traffic volumes and severe environment.

Full depth transverse floor cracks and longitudinal construction joints leak salt water on the girders below causing deterioration and over time, section loss.

Leaking expansion joints allow salt water seepage which causes deterioration of girder ends and steel bearings located under them. Also, concrete bridge seats will be affected in time. Concrete bridge seats should be finished flat, and sealed with a penetrating epoxy coating.

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

Thin polymer overlays can be placed on new concrete once it has been fully cured and dried to an acceptable moisture content. However, cracks will develop in the concrete deck throughout the first couple of years in response to vehicular and environmental loads. As a result, the preferred time to place the overlay is after initial concrete cracking, which should occur within the first two years of a new deck. Placement after this time allows the overlay to seal existing cracks and may reduce reflective cracking in the overlay. It should be noted that this application window is not ideal for projects, since it will usually require an additional contract for the overlay application. As a result, it is recommended that decks be sealed for the first several years and then receive a thin polymer overlay.

Sufficient bond strength is critical in maximizing the overlay's service life. The bond strength can be reduced by poor surface preparations, traffic conditions, moisture, and distressed concrete. As a result, TPO's should be used based on the following restrictions:

- Recommended on decks with a NBI rating greater than 7 to help mitigate chloride infiltration. The deck should be in good condition with wearing surface distressed areas not exceeding 2% of the total deck area.

- Not recommended on decks that have been exposed to chlorides for more than 10 years old or with a NBI rating less than 7. These restrictions assume that significant chloride infiltration has already occurred. When a robust deck washing and sealing program has been used, TPO's may be placed on decks 10-15 years old with above average deck condition. Roadway traffic volume should also be a consideration for determining when to apply a TPO. As roadway volumes increase, it is assumed that chloride infiltration occurs significantly faster due to the increased application of deicing salts.

- TPO's should not be placed on concrete decks or Portland cement concrete patches less than 28 days. Patch and crack repairs shall be compatible with the overlay material.

- Use of TPO's on the concrete approaches should be avoided. Slab-on-grade conditions may cause the overlay to fail prematurely due to moisture issues.

- Not recommended on decks with widespread cracking, large cracks (>0.04 in), or active cracks (e.g. longitudinal reflective cracks between PS box girders). These cracks are likely to reflect through the overlay, even when fully repaired.

- Decks with an existing TPO may be considered for a TPO re-application provided that the previously discussed restrictions can be assumed to be satisfied. Generally, this assumes the existing overlay performed well over its expected service life and the effective deck exposure did not exceed 15 years, such that significant chloride infiltration has not occurred. If signification chloride infiltration is expected, a re-application would not be recommended.

Thin polymer overlays may be considered where friction needs to be restored or improved. In most cases, the two-layer polymer overlay system should be used as it will improve surface friction and protect the deck against future chloride infiltration. For situations requiring a high skid resistance, calcined bauxite or other alternative aggerates may be considered in lieu of natural or synthetic aggregates.

40.5.1.2 Low Slump Concrete Overlay

A low slump concrete overlay, also referred to as a concrete overlay, is expected to extend the service life of a bridge deck for 15 to 20 years. This system is comprised of low slump Grade E concrete and has a 1-1/2 inch minimum thickness. The overlay thickness can accommodate profile and cross-slope differences, but typically does not exceed 4-1/2 inches. Thicker overlays become increasingly unpractical due to load and cost implications.

Low slump Grade E concrete requires close adherence to the specification, including equipment, consolidation, and curing requirements. A properly cured concrete overlay will help limit cracks, but inevitably the concrete overlay will crack. After the concrete overlay has been placed, it is beneficial to seal cracks in the overlay to minimize deterioration of the underlying deck. The overlay may require crack sealing the following year and periodically thereafter.

On delaminated but structurally sound decks, a rehabilitation concrete overlay is often the only alternative to deck replacement. Typically, prior to placing the concrete overlay a minimum of 1” of existing deck surface is removed along with any unsound material and asphaltic patches.

> Rehabilitation concrete overlays are performed when significant distress of the wearing surface has occurred. If more than 25% of the wearing surface is distressed, an in-depth cost analysis should be performed to determine if a concrete overlay is cost effective verses a deck replacement.
>
> The quantity of distress on the underside of deck or slab should be negligible, less than 5%, indicating that the bottom mat of reinforcement steel is not significantly deteriorated. If significant quantities of distress are present under the deck, a deck replacement may be required in the future; an overlay at this time might not achieve full service life, but may be placed to provide a good riding surface until replacement.

If the structure has an existing overlay, the overlay condition should be evaluated in addition to the other previously discussed considerations. If the concrete deck remains structurally sound, it may be practical to remove an existing overlay and place a new overlay before replacing the entire deck. Prior to placing the concrete overlay, the existing overlay should be removed to at least the original deck surface. Additional surface milling may not be practical if the previous overlay included a milling operation.

40.5.1.3 Polyester Polymer Concrete Overlay

A polyester polymer concrete (PPC) is expected to extend the service life of a bridge deck for 20 to 30 years. This system is a mixture of aggregate, polyester polymer resin, and initiator; which can be placed as a deck overlay using conventional concrete mixing and placement equipment, albeit most likely dedicated to PPC usage. The main advantages of a PPC overlay is that it is impermeable and causes minimal traffic disruptions due to its quick cure time. High costs and lack of performance data are the main disadvantages.

40.5.6 Other Considerations

- Bridges with Inventory Ratings less than HS10 after rehabilitation shall not be considered for overlays, unless approved by the Bureau of Structures Design Section.

- Inventory and Operating Ratings shall be provided on the bridge rehabilitation plans.

- Verify the desired transverse cross slope with the Regions as they may want to use current standards.

- On continuous concrete slab bridges with extensive spalling in the negative moment area, not more than 1/3 of the top bar steel should be exposed if the bar ends are not anchored. This is to maintain the continuity of the continuous spans and should be stated on the final structure plans. If more than 1/3 of the steel is exposed and the bar ends are not anchored, either adjacent spans must be shored or a special analysis and removal plan are required. Reinforcement shall be anchored using Portland cement concrete.

- Asphaltic overlays should not be considered on a bridge deck which has a longitudinal grade in excess of four percent or an extensive amount of stopping and starting traffic.

- All full-depth repairs shall be made with Portland cement concrete.

- Joints and floor drains should be modified to accommodate the overlay

- Concrete chloride thresholds – Chloride content tests measure the chloride ion concentrations at various depths. Generally, research has shown initiation of corrosion is expected when the chloride content is between 1 to 2 lbs/CY in concrete for uncoated bars and 7 to 12 lbs /CY for epoxy coated bars at the reinforcement. These limits are referred to as the threshold for corrosion. Threshold limits do not apply to stainless steel rebar.

  When the chloride ion content is greater than 0.8 lbs/CY in concrete for uncoated bars and 5 lbs /CY for epoxy coated bars at the reinforcement depth, measures should be considered to limit additional chloride infiltration.

- See Chapter 6-Plan Preparation and Chapter 40 Standards for additional guidance.

- Refer the standard details for the most current bid items.

- Overlay transitional areas should be used and coordinated when accommodating profile differences. These transitions are intended to improve ride quality and protect against snowplow damage. Ideally, transitions are placed such that the overlay thickness remains constant, which requires a tapered removal of the existing surface over a sufficient distance. For profile adjustments 1 1/2-inch or greater, transitional areas should consider a minimum taper rate of 1:250 for low-speed applications (RSD< 50 mph) and for high-speed applications up to a 1:400 taper rate. Typically, thicker profile adjustments are provided off the bridge deck and are coordinated by the roadway designer. For profile adjustments less than 1 1/2-inch, a minimum rate of 1:250 may be used regardless of the roadway design speed. For a 3/4-inch minimum PPC overlay, provide a 16-feet minimum transition length. For a 1/4-inch TPO overlay, a 3-feet minimum transition length is sufficient. See Chapter 40 Standards for additional guidance.

40.5.7 Past Bridge Deck Protective Systems

In the past, several bridge deck protective systems have been employed on the original bridge deck or while rehabilitating the existing deck as described in 17.8. The following systems have been used to protect bridge decks:

- Epoxy coated deck reinforcement – Prior to the 1980's, uncoated (black) bars were used throughout structures, including bridge decks. Criteria for epoxy coated reinforcement was first introduced in 1981 as a deck protective system. At first, usage was limited to the top mat of deck reinforcement. By 1987, coated bars were required in the top and bottom mats for high volume roadways (ADT > 5000). By 1991, coated bars were required for all State bridges and on some local bridges (ADT > 1000). Currently, use of epoxy coated deck reinforcement is required on all bridge decks.

- Asphaltic overlay with Membranes – Use of this overlay system was largely discontinued in the 1990's.

- High Performance Concrete (HPC) - Use of HPC has been limited to Mega Projects.

- Thin Polymer Overlays – Use of this overlay system is currently being used.

- Polyester Polymer Concrete Overlays – Use of this overlay system currently being used limitedly.

- Additional Concrete Cover – Use of additional clear cover (> 2 ½ inches) has been used on bridges with high volume and high truck traffic.

- Stainless steel deck reinforcement – Use of stainless steel has been very limited.

- Fiber reinforce polymer (FRP) deck reinforcement - Use of FRP reinforcement has only be used for experimental purposes.

As-built plans should be reviewed for past deck protective systems to assist with the appropriate rehabilitation measures.

40.5.8 Railings and Parapets

Overlays may decrease the parapet height when the existing overlay is not milled off and replaced in-kind. See Chapter 30-Railings for guidance pertaining to railings and parapets associated with rehabilitation structures projects.

40.6 Deck Replacements

Depending on the structure age or site conditions, the condition of original deck or deck overlay, a complete deck replacement may be the most cost effective solution and extend the life of the bridge by 40 years or more. Epoxy coated rebars are required on bridge deck replacements under the same criteria as for new bridges. The new deck and parapet or railing shall be designed per the most recent edition of the WisDOT Bridge Manual, including continuity bars and overhang steel. The top flange of steel girders should be painted.

The following condition or rating criteria are the minimum requirements on STN bridges (does not include local roadways over STN routes) eligible for deck replacements:

Item	Existing Condition	Condition after Construction
Deck Condition	≤ 4	≥ 8
Inventory Rating	---	≥ HS15*
Superstructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Substructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Horizontal and Vertical Alignment Condition	> 3	---
Shoulder Width	6 ft	6 ft

Table 40.6-1
Condition Requirements for Deck Replacements

*Rating evaluation is based on the criteria found in Chapter 45-Bridge Rating. An exception to requiring a minimum Inventory Rating of HS15 is made for continuous steel girder bridges, with the requirement for such bridges being a minimum Inventory Rating of HS10. For all steel girder bridges, assessment of fatigue issues as well as paint condition (and lead paint concerns) should be included in the decision as to whether a deck replacement or a superstructure/bridge replacement is the best option.

For any bridge not meeting the conditions for deck replacement, a superstructure, or likely a complete bridge replacement is recommended. For all Interstate Highway deck replacements, the bridges are to have a minimum Inventory Rating of HS20 after the deck is replaced.

WisDOT policy item:
Contact the Bureau of Structures Development Section Ratings Unit if a deck replacement for an Interstate Highway bridge would result in an Inventory Rating less than HS20.

For slab superstructure replacements on concrete pier columns, it is reported that the columns often get displaced and/or cracked during removal of the existing slab. Plans should be detailed showing full removal of these pier columns to the existing footing and replacement with current standard details showing a concrete cap with pier columns or shaft.

See the FDM and FDM SDD 14b7 for anchorage/offset requirements for temporary barrier used in staged construction. Where temporary bridge barriers are being used, the designer should attempt to meet the required offsets so that the barrier does not require anchorage which would necessitate drilling holes in the new deck.

In general, the substructure need not be analyzed for additional dead load provided the new deck is comparable to the existing. Exceptions include additional sidewalk or raised medians that would significantly alter the dead load of the superstructure.

For prestressed girder deck replacements, replace existing intermediate concrete diaphragms with new steel diaphragms at existing diaphragm locations (i.e. don't add intermediate lines of diaphragms). See Chapter 19 Standard Details and Steel Diaphragm Insert Sheets for additional information. Existing concrete diaphragms, in good condition, that are full-depth to the bottom of the girder (typically located at the abutments and piers) shall not be removed for a deck replacement.

For deck replacement projects that change global continuity of the structure, the existing superstructure and substructure elements shall be evaluated using LFD criteria. One example of this condition is an existing, multi-simple span structure with expansion joints located at the pier locations. From a maintenance perspective, it is advantageous to remove the joints from the bridge deck and in order to do so; the continuity of the deck must be made continuous over the piers.

40.7 Rehabilitation Girder Sections

WisDOT BOS has retired several girder shapes from standard use on new structures. The 45", 54" and 70" girder sections shall be used primarily for bridge rehabilitation projects such as girder replacements or widening.

These sections may also be used on a limited basis on new curved structures when the overhang requirements cannot be met with the wide-flange girder sections. These sections employ draped strand patterns with undraped alternates where feasible. Undraped strand patterns, when practical, should be specified on the designs. See the Standard Details for the girder sections' draped and undraped strand patterns.

The 45", 54", and 70" girders in Chapter 40-Bridge Rehabilitation standards have been updated to include the proper non-prestressed reinforcement so that these sections are LRFD compliant. Table 40.7-1 provides span lengths versus interior girder spacings for HL-93 live loading on single-span and multiple-span structures for prestressed I-girder rehabilitation sections. Girder spacing and span lengths are based on the following criteria:

- Interior girders with low relaxation strands at $0.75f_{pu}$,
- A concrete haunch of 2-1/2",
- Slab thicknesses from Chapter 17-Superstructure - General,
- A future wearing surface of 20 psf,
- A line load of 0.300 klf is applied to the girder to account for superimposed dead loads,
- 0.5" or 0.6" dia. strands (in accordance with the Standard Details),
- f'_c girder = 8,000 psi,
- f'_c slab = 4,000 psi, and
- Required f'_c girder at initial prestress < 6,800 psi

45" Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	102	112
6'-6"	100	110
7'-0"	98	108
7'-6"	96	102
8'-0"	94	100
8'-6"	88	98
9'-0"	88	96
9'-6"	84	90
10'-0"	84	88
10'-6"	82	86
11'-0"	78	85
11'-6"	76	84
12'-0"	70	80

54" Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	130	138
6'-6"	128	134
7'-0"	124	132
7'-6"	122	130
8'-0"	120	128
8'-6"	116	124
9'-0"	112	122
9'-6"	110	118
10'-0"	108	116
10'-6"	106	112
11'-0"	102	110
11'-6"	100	108
12'-0"	98	104

70" Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	150*	160*
6'-6"	146*	156*
7'-0"	144*	152*
7'-6"	140*	150*
8'-0"	138*	146*
8'-6"	134*	142*
9'-0"	132*	140*
9'-6"	128*	136
10'-0"	126*	134
10'-6"	122	132
11'-0"	118	128
11'-6"	116	126
12'-0"	114	122

Table 40.7-1
Maximum Span Length vs. Girder Spacing

*For lateral stability during lifting these girder lengths will require pick up point locations greater than distance d (girder depth) from the ends of the girder. The designer shall assume that the pick-up points will be at the 1/10 points from the end of the girder and provide extra non-prestressed steel in the top flange if required.

40.8 Widenings

Deck widenings, except on the Interstate, are attached to the existing decks if they are structurally sound and the remaining width is more than 50 percent of the total new width. If the existing deck is over 20 percent surface delaminated or spalled, the existing deck shall be replaced. For all deck widenings on Interstate Highway bridges, consideration shall be given to replacing the entire deck in order that total deck life is equal and costs are likely to be less when considering future traffic control. Evaluate the cost of traffic control for deck widenings on other highway bridges. The total deck should be replaced in these cases where the life-cycle cost difference is minimal if future maintenance costs are substantially reduced.

The design details must provide a means of moment and shear transfer through the joint between the new and existing portions of the deck. Lapped reinforcing bars shall have adequate development length and are preferable to doweled bars. The reinforcing laps must be securely tied or the bars joined by mechanical methods. When practical, detail lapped rebar splices. Mechanical splice couplers or threaded rebar couples are expensive and should only be detailed when required. Generally, shear transfer is more than sufficient without a keyway. Bridge Maintenance Engineers have observed that if the existing bar steel is uncoated, newly lapped coated bars will accelerate the uncoated bar steel deterioration rate.

When widening a deck on a prestressed girder bridge, if practical, use the latest standard shape (e.g. 54W” rather than 54”). Spacing the new girder(s) to maintain comparable bridge stiffness is desirable. *The girders used for widenings may be the latest Chapter 19-Prestressed Concrete sections designed to LRFD or the sections from Chapter 40-Bridge Rehabilitation designed LFD, or LRFD with non-prestressed reinforcement as detailed in the Standard Details.*

For multi-columned piers, consider the cap connection to the existing cap as pinned and design the new portion to the latest LRFD criteria. The new column(s) are not required to meet **LRFD [3.6.5]** (600 kip loading) as a widening is considered rehabilitation. Abutments shall be widened to current LRFD criteria as well as the Standard for Abutment Widening.

For foundation support, use the most current method available.

All elements of the widening shall be designed to current LRFD criteria. Railings and parapets placed on the new widened section shall be up to current design and safety standards. For the non-widened side, substandard railings and parapets should be improved. Contact the Bureau of Structures Development Section to discuss solutions.

For prestressed girder widenings, only use intermediate steel diaphragms in-line with existing intermediate diaphragms (i.e. don’t add intermediate lines of diaphragms).

40.9 Superstructure Replacement

Various types of superstructure replacements include replacing prestressed girders in-kind, replacing slabs in-kind and replacing steel girders with prestressed girders or slabs. When considering replacement of a deck on steel girders, consideration of the cost of painting the structural steel should be included in the evaluation.

Approval is required from BOS for all superstructure replacement projects. To ensure that the cost of a superstructure replacement is warranted, the substructure should be in good condition. In general, the superstructure replacement should remain the same as the original design to better ensure that substructure reuse is practical. See 40.10 for considerations regarding substructure reuse criteria.

WisDOT policy item:

Provided that the substructure meets the criteria in 40.10, the superstructure may be replaced. The superstructure shall be designed to current LRFD criteria.

Reuse of the existing substructure is contingent on the fixity of the substructure units remaining the same. If the fixity is changed, the substructure must be evaluated per the design loading of the original structure.

With the substructure needing further evaluation for increased dead load and/or change in fixity, discuss with BOS the acceptability of the evaluation results prior to continuing with final design.

40.10 Substructure Reuse and Replacement

When practical, substructure reuse may be an acceptable alternative to replacing the entire bridge. However, reuse will require early coordination with BOS, engineering judgement, and will be evaluated on a project-by-project basis. This evaluation should determine if the substructure can be reused "as-is" with or without minor surface repairs, reused with major repairs and/or strengthening, or needs to be replaced.

In general, "as-is" reuse of substructures should be reserved for in-kind superstructure replacements with little to no change in geometry, fixity, and service dead loads. Additionally, substructures should be in good condition and only require minor surface repairs. If satisfied, evaluation of the existing substructure with the load rating methodology as discussed in 45.3.2 for an existing (in-service) bridge (e.g. LFR) may be acceptable. An example of this condition would be an in-kind slab superstructure replacement with a substructure that remains in good condition. For other conditions (i.e. reuse with major repairs and/or strengthening), the substructure should be evaluated with the current load rating methodology (LRFR) as discussed in 45.3.1.1 for new bridge construction. If substructure reuse is found to be not practical due the expensive repairs and/or excessive strengthening, the substructure should be completely replaced.

<u>Approval is required from BOS for all substructure reuse projects.</u>

Normally it is acceptable to assume that the original bridge design was done correctly, however pier caps, either for multi-columned piers or open pile bents, have occasionally been under-designed. Further investigation is warranted for pier caps with nominal shear stirrups, rather than stirrups that appear to be designed for the girder configuration, etc.

See 40.15 for more information on substructure inspection.

Additional guidance regarding substructure reuse can be found in the FHWA publication *Foundation Reuse for Highway Bridges*.

40.10.1 Substructure Rehabilitation

Substructure rehabilitation work can vary significantly from minor concrete surface repairs to major repairs that includes strengthening members.

40.10.1.1 Piers

Pier caps and/or columns/shafts may show signs of distress due to spalled concrete. The spalling may be completely around some of the longitudinal bar steel, thus destroying the bond. The concrete usually remains sound under the bearing plates, possibly due to compressive forces preventing salt intrusion and/or deterioration from freeze thaw cycles.

If the bond of the structural reinforcement is <u>not</u> compromised (at least half of the bar is bonded), rehabilitation measures include:

1. Concrete surface repair for smaller areas. Fiber Reinforced Polymer (FRP), either non-structural or structural, may be required. See 40.20 for more information on FRP.

2. Column encapsulation. Even if the bars are bonded, the encapsulation provides protection to further damage from snow impact produced by plowing. For encapsulation:

    a. Place adhesive anchors

    b. Place wire mesh around column

    c. Pour 6" concrete encapsulation

If the bond of the structural reinforcement is compromised (at least half of the bar is not bonded), rehabilitation measures include:

1. Cap and/or column/shaft encapsulation. Fiber Reinforced Polymer (FRP), either non-structural or structural, may be required. See 40.20 for more information on FRP.

2. Column encapsulation. The encapsulation provides protection to further damage from snow impact produced by plowing. For encapsulation:

    a. Place adhesive anchors

    b. Place wire mesh around column

    c. Pour 6" concrete encapsulation

40.10.1.2 Bearings

Bearings being replaced should follow the Chapter 27 Standard Details, as well as the Chapter 40 Standard for Expansion Bearing Replacement Details. Replace lubricated bronze bearings with either laminated elastomeric bearings (preferred, if feasible) or Stainless Steel TFE bearings. If only outside bearings are replaced, the difference in friction/resistance values between adjacent girders can be ignored. In addition to the bid item for the new bearing, the STSP Removing Bearings is required.

For bearings requiring maintenance, consider the SPV Cleaning and Painting Bearings. Special Provisions Bearing Maintenance and Bearing Repair may also be worthy of consideration.

40.11 Other Considerations

40.11.1 Replacement of Impacted Girders

When designing a replacement project for girders that have been damaged by vehicular impact, replace the girder in-kind using the latest details. Either HS loading or HL-93 loading may be used if originally designed with non-LRFD methods. Consider using the latest deck details, especially with regard to overhang bar steel.

For the parapet or railing, the designer should match the existing.

40.11.2 New Bridge Adjacent to Existing Bridge

For a new bridge being built adjacent to an existing structure, the design of the new structure shall be to current LRFD criteria for the superstructure and abutment.

The pier design shall be to current LRFD criteria, including the 600 kip impact load for the new bridge. It is not required to strengthen or protect the existing adjacent pier for the 600 kip impact load. However, it would be prudent to discuss with the Region the best course of action. If the Region wants to provide crash protection, it may be desirable to provide TL-5 barrier/crash wall protection for both structures, thus eliminating the need to design the new pier for the 600 kip impact load. The Region may also opt to provide typical barrier protection (< TL-5) to both sets of piers, in which case the design engineer would still be required to design the new pier for the 600 kip impact load. This last option is less expensive than providing TL-5 barrier to both structures. Aesthetics are also a consideration in the above choices.

40.12 Timber Abutments

The use of timber abutments shall be limited to rehabilitation or widening of off-system structures.

Timber abutments consist of a single row of piling capped with timber or concrete, and backed with timber to retain the approach fill. The superstructure types are generally concrete slab or timber. Timber-backed abutments currently exist on Town Roads and County Highways where the abutment height does not preclude the use of timber backing.

Piles in bents are designed for combined axial load and bending moments. For analysis, the assumption is made that the piles are supported at their tops and are fixed 6 feet below the stream bed or original ground line. For cast-in-place concrete piling, the concrete core is designed to resist the axial load. The bending stress is resisted by the steel shell section. Due to the possibility of shell corrosion, steel reinforcement is placed in the concrete core equivalent to a 1/16-inch steel shell perimeter section loss. The reinforcement design is based on equal section moduli for the two conditions. Reinforcement details and bearing capacities are given on the Standard Detail for Pile Details. Pile spacing is generally limited to the practical span lengths for timber backing planks.

The requirements for tie rods and deadmen is a function of the abutment height. Tie rods with deadmen on body piling are used when the height of "freestanding" piles is greater than 12 feet for timber piling and greater than 15 feet for cast-in-place concrete and steel "HP" piling. The "freestanding" length of a pile is measured from the stream bed or berm to grade. If possible, all deadmen should be placed against undisturbed soil.

Commercial grade lumber as specified in AASHTO having a minimum flexural resistance of 1.2 ksi is utilized for the timber backing planks. The minimum recommended nominal thickness and width of timber backing planks are 3 and 10 inches, respectively. If nominal sizes are specified on the plans, analysis computations must be based on the dressed or finished sizes of the timber. Design computations can be used on the full nominal sizes if so stated on the bridge plans. For abutments constructed with cast-in-place concrete or steel "HP" piles, the timber planking is attached with 60d common nails to timber nailing strips which are bolted to the piling.

40.13 Survey Report and Miscellaneous Items

Prior to scheduling bridge plans preparation, a Rehabilitation Structure Survey Report Form is to be completed for the proposed work with field and historical information. The Rehabilitation Structure Survey Report provides the Bridge Engineer pertinent information such as location, recommended work to be performed, and field information needed to accomplish the rehabilitation plans. A brief history of bridge construction date, type of structure, repair description and dates are also useful in making decisions as to the most cost effective rehabilitation. Along with this information, it is necessary to know the extent of deck delamination and current deck, super and substructure condition ratings. A thorough report recommending structural repairs and rehabilitation with corresponding notes is very useful for the designer including information from special inspections.

Because new work is tied to existing work, it is very important that accurate elevations be used on structure rehabilitation plans involving new decks and widenings. Survey information of existing beam seats is essential. Do not rely on original plans or a plan note indicating that it is the responsibility of the contractor to verify elevations.

Experience indicates that new decks open to traffic and subjected to application of de-icing salts within the first year show signs of early deck deterioration. Therefore, a protective surface treatment is applied to all new bridge deck concrete as given in the standard specifications.

For existing sloped faced parapets on decks under rehabilitation, the metal railings may be removed. On deck replacements and widenings, all existing railings are replaced. Refer to Bridge Manual Chapter 30-Railings for recommended railings.

On rehabilitation plans requiring removal of the existing bridge name plates; provide details on the plans for new replacement name plates if the existing name plate cannot be reused. The existing bridge number shall be used on the name plates for bridge rehabilitation projects.

If floor drain removal is recommended, review these recommendations for conformance to current design standards and remove any floor drains not required. Review each structure site for length of structure, grade, and water erosion at the abutments.

Bridge rehabilitation plans for steel structures are to provide existing flange and web sizes to facilitate selecting the proper length bolts for connections. If shear connectors are on the existing top flanges, additional shear connectors are not to be added as welding to the top flange may be detrimental. Do not specify any aluminized paint that will come in contact with fresh concrete. The aluminum reacts severely with the fresh concrete producing concrete volcanoes.

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

Structure plans (using a sheet border with a #8 tab) are required for all structure rehabilitation projects. This includes work such as superstructure painting projects and all overlay projects, including polymer overlay projects. See Chapter 6-Plan Preparation guidance for plan minimum requirements.

Existing steel expansion devices shall be modified or replaced with watertight expansion devices as shown in Bridge Manual Chapter 28-Expansion Devices. If the hinge is repaired, consideration should be given to replacing the pin plates with the pins. Replace all pins with stainless steel pins conforming to ASTM 276, Type S20161 or equal. On unpainted steel bridges, the end 6' or girder depth, whichever is greater, of the steel members adjacent to an expansion joint and/or hinge are required to have two shop coats of paint. The second coat is to be a brown color similar to rusted steel. Exterior girder faces are not painted for aesthetic reasons, but paint the hanger on the side next to the web.

A bridge on a steep vertical grade may slide downhill closing any expansion joint on this end and moving the girders off center from the bearings. One possible corrective action is to block the ends of the girder during the winter when the girders have shortened due to cold temperatures. Continued blocking over a few seasons should return the girders to the correct position. The expansion device at the upslope end may have to be increased in size.

Raised pavement markers require embedment in the travel way surface. As a general rule, they are not to be installed on bridge decks due to future leakage problems if the marker comes out. They shall not be installed thru the membranes on asphalt overlays and not at all on asphaltic decks where the waterproofing is integral with the surface.

40.14 Superstructure Inspection

40.14.1 Prestressed Girders

On occasion prestressed concrete girders are damaged during transport, placement, or as a result of vehicle impact. Damage inspection and assessment procedures are necessary to determine the need for traffic restrictions, temporary falsework for safety and/or strength. Three predominant assessment areas are damage to prestressing strands, damage to concrete, and remaining structural integrity.

Where damage to a girder results in any significant loss of concrete section, an engineering analysis should be made. This analysis should include stress calculations for the damaged girder with comparison to the original design stresses. These calculations will show the loss of strength from the damage.

Assessment of damage based on the loss of one or more prestressing strands or loss of prestress force is given as reason for restriction or replacement of girders. Some of the more common damages are as follows with the recommended maintenance action. However, an engineering assessment should be made on all cases.

1. If cracking and spalling are limited to the lower flange, patching is usually performed. This assessment should be based on calculations that may allow repair.

2. If cracking continues from flange into web, the girder is normally replaced. Findings indicate that sometimes repair-in-place may be the preferred decision.

3. Termini of cracks are marked, and if the cracks continue to grow the girder may be replaced. This is a good method to determine the effect of loads actually being carried.

4. When large areas of concrete are affected, or when concrete within stirrups is fractured, replace. At times it may be more appropriate to repair-in-place than replace.

Repair and/or replacement decisions are based on structural integrity. Load capacity is by far the most important rationale for selection of repair methods. Service load capacity needs to be calculated if repair-in-place is contemplated.

Evaluations of criteria for assessment of damage related to the two predominant areas associated with structural integrity are the following:

1. A structural analysis is made to determine the stresses in a damaged girder on the basis of the actual loads the girder will carry. This technique may result in a girder that has less capacity than the original, but can still safely carry the actual loads. This assessment results in consideration of possible repair-in-place methods which normally are cost effective.

   or

2. A structural analysis is made to determine the load capacity and rating of the girder. If the capacity and rating of the girder is less than provided by the original design, the girder shall be replaced. This assessment will provide a girder equal to the original design, but precludes possible repair-in-place methods that are normally less costly.

Location and size of all spalled and unsound concrete areas shall be recorded. Location, length, and width of all visible cracks shall be documented. All damage to prestress strands and reinforcing steel shall be reported. Location of hold-down devices in the girder shall be shown in relation to the damage. Horizontal and vertical misalignment along the length of the girder, and at points of localized damage, shall be reported. (These measurements might best be made by string-lining). Growth of cracks shall be monitored to determine that the cracked section has closed before extending to the web.

Critical damage is damage to concrete and/or the reinforcing elements of prestressed concrete girders such as:

1. Cracks extend across the bottom flange and/or in the web directly above the bottom flange. (This indicates that the prestressing strands have exceeded yield strength).

2. An abrupt lateral offset is measured along the bottom flange or lateral distortion of exposed prestressing strands. (This also indicates that the prestressing strands have exceeded yield strength).

3. Loss of prestress force to the extent that calculations show that repairs cannot be made.

4. Vertical misalignment in excess of the normal allowable.

5. Longitudinal cracks at the interface of the web and the top flange that are not substantially closed below the surface. (This indicates permanent deformation of stirrups).

40.14.2 Steel Beams

These are three alternate methods of repairing damaged steel beams. They are:

1. Replace the total beam,

2. Replace a section of the beam, or

3. Straighten the beam in-place by heating and jacking.

The first alternate would involve removing the concrete deck over the damaged beam, remove the damaged section and weld in a new piece; then reconstruct the deck slab and railing over the new girder. Falsework support is required at the locations where the beams are cut and probably in the adjacent span due to an unbalanced condition.

The second alternate involves cutting out a section of the beam after placing the necessary supporting members. The support is placed using calibrated jacks. The section is cut out as determined by the damage. A new section plus any vertical stiffeners and section of cover plates would be welded in. This involves butt welds on both the flange and web. The welding of the web is difficult due to minor misalignments to start with plus the tendency of thin plates to move from the heat of welding.

The third alternate of heating and jacking the in-place beam to straighten it is a difficult procedure but can be done by personnel familiar and knowledgeable of the process. It is important to maintain heat control under 1300°F maximum. Use an optical pyrometer to determine heat temperature. There is no specified tolerance for the straightened member. The process is deemed satisfactory when a reasonable alignment is obtained.

Based on the three alternates available, the estimated costs involved and the resultant restoration of the beam to perform its load carrying capacity, heat straightening is a viable option in many cases.

The structural engineer who will be responsible for plan preparation should field review the site with the Regional Bridge Maintenance Engineer.

40.15 Substructure Inspection

The inspection of substructure components may reveal deteriorating concrete in areas exposed to de-icing chemicals from roadway drainage or concrete disintegration where exposed in splash zones or stream flows. Footings and pilings exposed due to erosion and undermining could result in loss of bearing capacity and/or section. Utilize HSIS data to flag potential scour concerns (code 6000), with scour defects in condition state 4 being a significant concern.

Abutment and pier concrete reuse may require core tests to determine the quality and strength of the concrete. Original pile capacities are determined from plans, or if available, the pile driving records. Reuse of steel pile sections will require checking the remaining load carrying capacity if section loss is determined to be present. Steel piling should be checked:

- Immediately below the splash zone or water line for deterioration and possible loss of section. High section loss occurs in some areas due to corrosion from bacterial attack at 3 to 6 feet below the water line.

- Below abutments where the berm soil (material beneath riprap) has settled below the abutment bottom and water appears to be flowing from beneath the abutment or stream water has direct access to the piling.

If there is piling section loss or undermined spread footings, capacities of existing piling and/or footings will need to be recomputed for load rating purposes.

Timber substructure components may exhibit deterioration due to fungus decay, abrasion wear and weathering. Also, physical damage may be caused by vermin attacks, chemicals, fires, and collisions. Prior to reuse, timber backed abutments and pier bents shall be checked, by boring, for material and mechanical condition, section loss and structural adequacy. Generally, timber substructures are not good candidates for substructure reuse due to their limited service life.

Bearing condition needs to be evaluated. When possible, replacement expansion bearings should be laminated elastomeric bearings. Replacing expansion devices to reduce chloride infiltration is often warranted.

40.16 Concrete Anchors for Rehabilitation

Concrete anchors are used to connect concrete elements with other structural or non-structural elements and can either be cast into concrete (cast-in-place anchors) or installed after concrete has hardened (post-installed anchors). This section discusses post installed anchors used on bridge rehabilitation projects. Note: this section is also applicable for several cases where post installed anchors may be allowed in new construction.

This section includes guidance based on the ACI 318-14 manual, hereafter referred to as ACI. (AASHTO currently does not have guidance for anchors.)

40.16.1 Concrete Anchor Type and Usage

Concrete anchors installed in hardened concrete, post-installed anchors, typically fall into two main groups – adhesive anchors and mechanical anchors. For mechanical anchors, subgroups include undercut anchors, expansion (torque-controlled or displacement controlled) anchors, and screw anchors.

Mechanical anchors are seldom used for bridge rehabilitations and current usage has been restricted due to the following concerns: anchor installation (hitting rebar, abandoning holes, and testing), the number of different anchor types, design requirements that are more restrictive than adhesive anchors, the ability to remove and reuse railings/fences, and the collection of salt water within the hole. Note: mechanical anchors may be considered when it has been determined cast-in-place anchors or through bolts are cost prohibitive, adhesive anchors are not recommended, and the above concerns for mechanical anchors have been addressed. See post-installed anchor usage restrictions for additional information.

An Approved Products List addresses some of the concerns for creep, shrinkage, and deterioration under load and freeze-thaw cycles for adhesives anchors. Bridge rehabilitations projects typically use adhesive anchors for abutment and pier widenings. Other bridge rehabilitation applications may also warrant the use of adhesive anchors when required to anchor into existing concrete. Refer to the Standards for several examples of anchoring into existing concrete.

In limited cases, post installed concrete anchors may be allowed for new construction. One application is the allowance for the contractor to use adhesive anchors in lieu of cast-in-place concrete anchors for attaching pedestrian railings/fencing. Refer to Chapter 30 Standards for pedestrian railings/fencing connections.

The following is a list of current usage restrictions for post installed anchors:

Usage Restrictions:

- Pier cap extensions for multi-columned piers require additional column(s) to be utilized. See Chapter 13 – Piers for structural modeling concepts regarding multi-columned piers.

- **Contact the Bureau of Structures if considering any extension of a hammerhead pier (without additional vertical support from an added column).**

- Adhesive anchors installed in the overhead or upwardly inclined position and/or under sustained tension loads shall not be used.

- The department has placed a moratorium on mechanical anchors. Usage is subject to prior-approval by the Bureau of Structures.

40.16.1.1 Adhesive Anchor Requirements

For adhesive anchors, there are two processes used to install the adhesive. One option uses a two-part adhesive that is mixed and poured into the drilled hole. The second option pumps a two-part adhesive into the hole by a dispenser which combines the two components at the nozzle just prior to entering the hole or within the hole. With either process, the hole must be properly cleaned and a sufficient amount of adhesive must be used so that the hole is completely filled with adhesive when the rebar or bolt is inserted. The adhesive bond stresses, as noted in Table 40.16 1, are determined by the 5 percent fractile of results of tests performed and evaluated according to ICC-ES AC308 or ACI 355.4.

The required minimum anchor spacing is 6 times the anchor diameter. The minimum edge distance is 6 times the anchor diameter. The maximum embedment depth for is 20 times the anchor diameter.

The manufacturer and product name of adhesive anchors used by the contractor must be on the Department's approved product list for "Concrete Adhesive Anchors".

Refer to the *Standard Specifications* for additional requirements.

40.16.1.2 Mechanical Anchor Requirements

The required minimum anchor spacing is 6 times the anchor diameter. The minimum edge distance is 10 times the anchor diameter. The minimum member is the great of the embedment depth plus 4 inches and 3/2 of the embedment depth. ***Mechanical anchors are currently not allowed.***

40.16.2 Concrete Anchor Reinforcement

Reinforcement used to transfer the full design load from the anchors into the structural member is considered anchor reinforcement. **ACI [17.4.2.9]** and **ACI [17.5.2.9]** provide guidance for designing anchor reinforcement. When anchor reinforcement is used, the design strength of the anchor reinforcement can be used in place of concrete breakout strength per 40.16.3 and 40.16.4. Reinforcement that acts to restrain the potential concrete breakout but is not designed to transfer the full design load is considered to be supplementary reinforcement.

Per **ACI [2.3],** concrete anchor steel is considered ductile if the tensile test elongation is at least 14 percent and reduction in area is at least 30 percent. Additionally, steel meeting the requirements of ASTM A307 is considered ductile. Steel that does not meet these requirements is considered brittle. Rebar used as anchor steel is considered ductile.

40.16.3 Concrete Anchor Tensile Capacity

Concrete anchors in tension fail in one of four ways: steel tensile rupture, concrete breakout, pullout strength of anchors in tension, or adhesive bond. The pullout strength of anchors in tension only applies to mechanical anchors and the adhesive bond only applies to adhesive anchors. Figure 40.16-1 shows the concrete breakout failure mechanism for anchors in tension.

The minimum pullout capacity (Nominal Tensile Resistance) of a single concrete anchor is determined according to this section; however, this value is only specified on the plan for mechanical anchors. The minimum pullout capacity is not specified on the plan for adhesive anchors because the anchors must be designed to meet the minimum bond stresses as noted in Table 40.16-1. If additional capacity is required, a more refined analysis (i.e., anchor group analysis) per the current version of ACI 318-14 Chapter 17 is allowable, which may yield higher capacities.

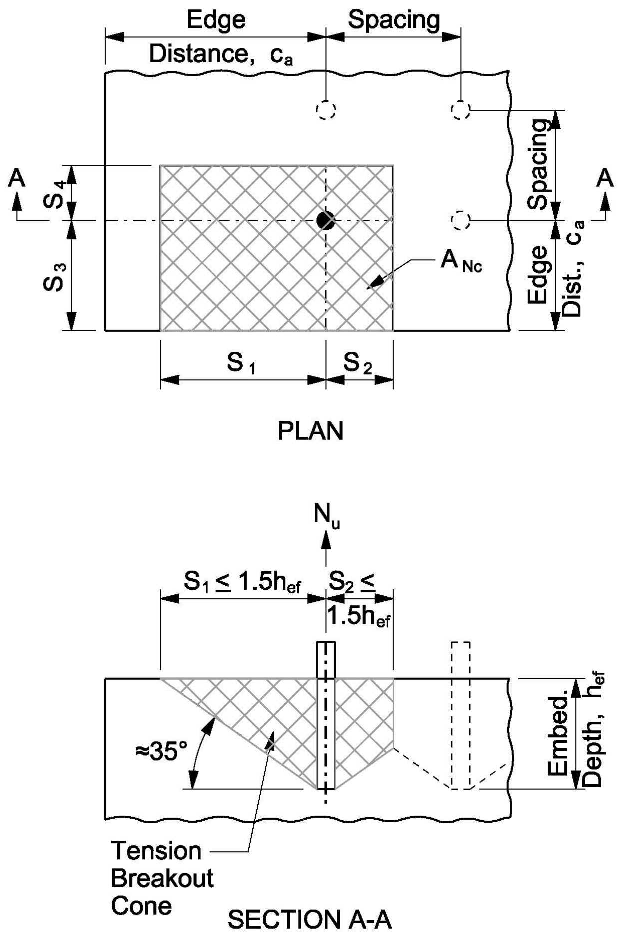

Figure 40.16-1
Concrete Breakout of Concrete Anchors in Tension

The projected concrete breakout area, A_{Nc}, shown in Figure 40.16-1 is limited in each direction by S_i:

S_i = Minimum of:

1. 1.5 times the embedment depth (h_{ef}),
2. Half of the spacing to the next anchor in tension, or
3. The edge distance (c_a) (in).

Figure 40.16-2 shows the bond failure mechanism for concrete adhesive anchors in tension.

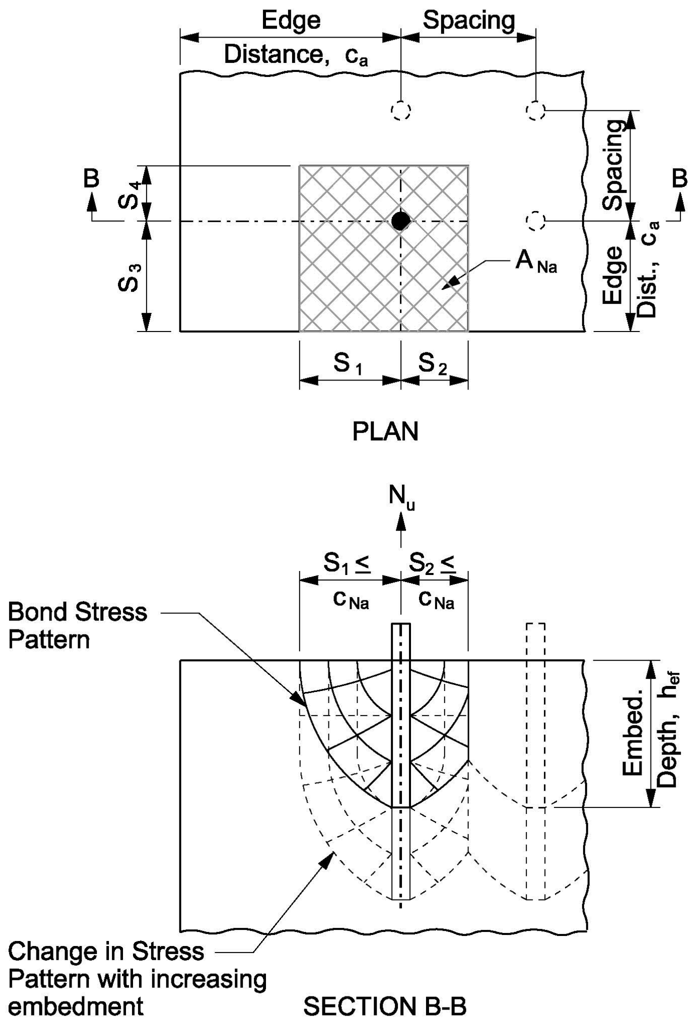

Figure 40.16-2

Bond Failure of Concrete Adhesive Anchors in Tension

The projected influence area of a single adhesive anchor, A_{Na}, is shown in Figure 40.16-2. Unlike the concrete breakout area, it is not affected by the embedment depth of the anchor. A_{Na} is limited in each direction by S_i:

S_i = Minimum of:

1. $c_{Na} = 10 d_a \sqrt{\frac{\tau_{uncr}}{1100}}$,

2. Half of the spacing to the next anchor in tension, or

3. The edge distance (c_a) (in).

Anchor Size, d_a	Adhesive Anchors			
	Dry Concrete		Water-Saturated Concrete	
	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)
#4 or 1/2"	990	460	370	280
#5 or 5/8”	970	460	510	390
#6 or 3/4"	950	490	500	410
#7 or 7/8”	930	490	490	340
#8 or 1”	770	490	600	340

Table 40.16-1
Tension Design Table for Concrete Anchors

The minimum bond stress values for adhesive anchors in Table 40.16-1 are based on the Approved Products List for “Concrete Adhesive Anchors”. The designer shall determine whether the concrete adhesive anchors are to be utilized in dry concrete (i.e., rehabilitation locations where concrete is fully cured, etc.) or water-saturated concrete (i.e., new bridge decks, box culverts, etc.) and shall design the anchors accordingly.

The factored tension force on each anchor, N_u, must be less than or equal to the factored tensile resistance, N_r. For mechanical anchors:

$$N_r = \phi_{ts} N_{sa} \leq \phi_{tc} N_{cb} \leq \phi_{tc} N_{pn}$$

In which:

ϕ_{ts} = Strength reduction factor for anchors in concrete, **ACI [17.3.3]**
= 0.65 for brittle steel as defined in 40.16.1.1
= 0.75 for ductile steel as defined in 40.16.1.1

N_{sa} = Nominal steel strength of anchor in tension, **ACI [17.4.1.2]**
= $A_{se,N} f_{uta}$

$A_{se,N}$ = Effective cross-sectional area of anchor in tension (in^2)

f_{uta} = Specified tensile strength of anchor steel (psi)

$\leq 1.9f_{ya}$

$\leq$ 125 ksi

f_{ya} = Specified yield strength of anchor steel (psi)

ϕ_{tc} = Strength reduction factor for anchors in concrete
= 0.65 for anchors without supplementary reinforcement per 40.16.2
= 0.75 for anchors with supplementary reinforcement per 40.16.2

N_{cb} = Nominal concrete breakout strength in tension, **ACI [17.4.2.1]**

$$= \frac{A_{Nc}}{9(h_{ef})^2}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$$

A_{Nc} = Projected concrete failure area of a single anchor, see Figure 40.16-1
= $(S_1 + S_2)(S_3 + S_4)$

h_{ef} = Effective embedment depth of anchor per Table 40.16-1. May be reduced per **ACI [17.4.2.3]** when anchor is located near three or more edges.

$\psi_{ed,N}$ = Modification factor for tensile strength based on proximity to edges of concrete member, **ACI [17.4.2.5]**
= 1.0 if $c_{a,min} \geq 1.5h_{ef}$

$$= 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} \text{ if } c_{a,min} < 1.5h_{ef}$$

$c_{a,min}$ = Minimum edge distance from center of anchor shaft to the edge of concrete, see Figure 40.16-1 (in)

$\psi_{c,N}$ = Modification factor for tensile strength of anchors based on the presence or absence of cracks in concrete, **ACI [17.4.2.6]**
= 1.0 when post-installed anchors are located in a region of a concrete member where analysis indicates cracking at service load levels
= 1.4 when post-installed anchors are located in a region of a concrete member where analysis indicates no cracking at service load levels

$\psi_{cp,N}$ = Modification factor for post-installed anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, **ACI [17.4.2.7]**

= 1.0 if $c_{a,min} \geq c_{ac}$

= $\frac{c_{a,min}}{c_{ac}} \geq \frac{1.5h_{ef}}{c_{ac}}$ if $c_{a,min} < c_{ac}$

c_{ac} = Critical edge distance (in)

= $4.0h_{ef}$

N_b = Concrete breakout strength of a single anchor in tension in uncracked concrete, **ACI [17.4.2.2]**

= $0.538\sqrt{f'_c}(h_{ef})^{1.5}$ (kips)

N_{pn} = Nominal pullout strength of a single anchor in tension, **ACI [17.4.3.1]**

= $\psi_{c,P}N_p$

$\psi_{c,P}$ = Modification factor for pullout strength of anchors based on the presence or absence of cracks in concrete, **ACI [17.4.3.6]**

= 1.4 where analysis indicates no cracking at service load levels

= 1.0 where analysis indicates cracking at service load levels

N_p = Nominal pullout strength of a single anchor in tension based on the 5 percent fractile of results of tests performed and evaluated according to ICC-ES AC193 / ACI 355.2

For adhesive anchors:

$$N_r = \phi_{ts}N_{sa} \leq \phi_{tc}N_{cb} \leq \phi_{tc}N_a$$

In which:

N_{cb} = Nominal concrete breakout strength in tension, **ACI [17.4.2.1]**

= $\frac{A_{Nc}}{9(h_{ef})^2}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$

h_{ef} = Effective embedment depth of anchor. May be reduced per **ACI [17.4.2.3]** when anchor is located near three or more edges.

≤ $20d_a$ (in)

d_a = Outside diameter of anchor (in)

$\psi_{cp,N}$ = Modification factor for post-installed anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, **ACI [17.4.2.7]**

= 1.0 if $c_{a,min} \geq c_{ac}$

= $\frac{c_{a,min}}{c_{ac}} \geq \frac{1.5h_{ef}}{c_{ac}}$ if $c_{a,min} < c_{ac}$

$c_{a,min}$ = Minimum edge distance from center of anchor shaft to the edge of concrete, see Figure 40.16-1 or Figure 40.16-2 (in)

c_{ac} = Critical edge distance (in)

= $2.0h_{ef}$

N_a = Nominal bond strength of a single anchor in tension, **ACI [17.4.5.1]**

= $\frac{A_{Na}}{4c_{Na}^{2}} \psi_{ed,Na} \psi_{cp,Na} N_{ba}$

A_{Na} = Projected influence area of a single adhesive anchor, see Figure 40.16-2

= $(S_1 + S_2)(S_3 + S_4)$

$\psi_{ed,Na}$ = Modification factor for tensile strength of adhesive anchors based on the proximity to edges of concrete member, **ACI [17.4.5.4]**

= 1.0 if $c_{a,min} \geq c_{Na}$

= $0.7 + 0.3\frac{c_{a,min}}{c_{Na}}$ if $c_{a,min} < c_{Na}$

c_{Na} = Projected distance from center of anchor shaft on one side of the anchor required to develop the full bond strength of a single adhesive anchor

= $10d_a\sqrt{\frac{\tau_{uncr}}{1100}}$ (in)

τ_{uncr} = Characteristic bond stress of adhesive anchor in uncracked concrete, see Table 40.16-1

$\psi_{cp,Na}$ = Modification factor for pullout strength of adhesive anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, **ACI [17.4.5.5]**

= 1.0 if $c_{a,min} \geq c_{ac}$

$$= \frac{c_{a,min}}{c_{ac}} \geq \frac{c_{Na}}{c_{ac}} \text{ if } c_{a,min} < c_{ac}$$

N_{ba} = Bond strength in tension of a single adhesive anchor, **ACI [17.4.5.2]**
= $\tau_{cr} \pi d_a h_{ef}$

τ_{cr} = Characteristic bond stress of adhesive anchor in cracked concrete, see Table 40.16-1

Note: Where analysis indicates cracking at service load levels, adhesive anchors shall be qualified for use in cracked concrete in accordance with ICC-ES AC308 / ACI 355.4. For adhesive anchors located in a region of a concrete member where analysis indicates no cracking at service load levels, τ_{uncr} shall be permitted to be used in place of τ_{cr}.

In addition to the checks listed above for all adhesive anchors, the factored sustained tensile force must be less than or equal to the factored sustained tensile resistance per **ACI [17.3.1.2]**:

$$0.50\ \phi_{tc} N_{ba} \geq N_{ua,s}$$

40.16.4 Concrete Anchor Shear Capacity

Concrete anchors in shear fail in one of three ways: steel shear rupture, concrete breakout, or concrete pryout. Figure 40.16-3 shows the concrete breakout failure mechanism for anchors in shear.

The projected concrete breakout area, A_{Vc}, shown in Figure 40.16-3 is limited vertically by H, and in both horizontal directions by S_i:

H = Minimum of:

1. The member depth (h_a) or
2. 1.5 times the edge distance (c_{a1}) (in).

S_i = Minimum of:

1. Half the anchor spacing (S),
2. The perpendicular edge distance (c_{a2}), or
3. 1.5 times the edge distance (c_{a1}) (in).

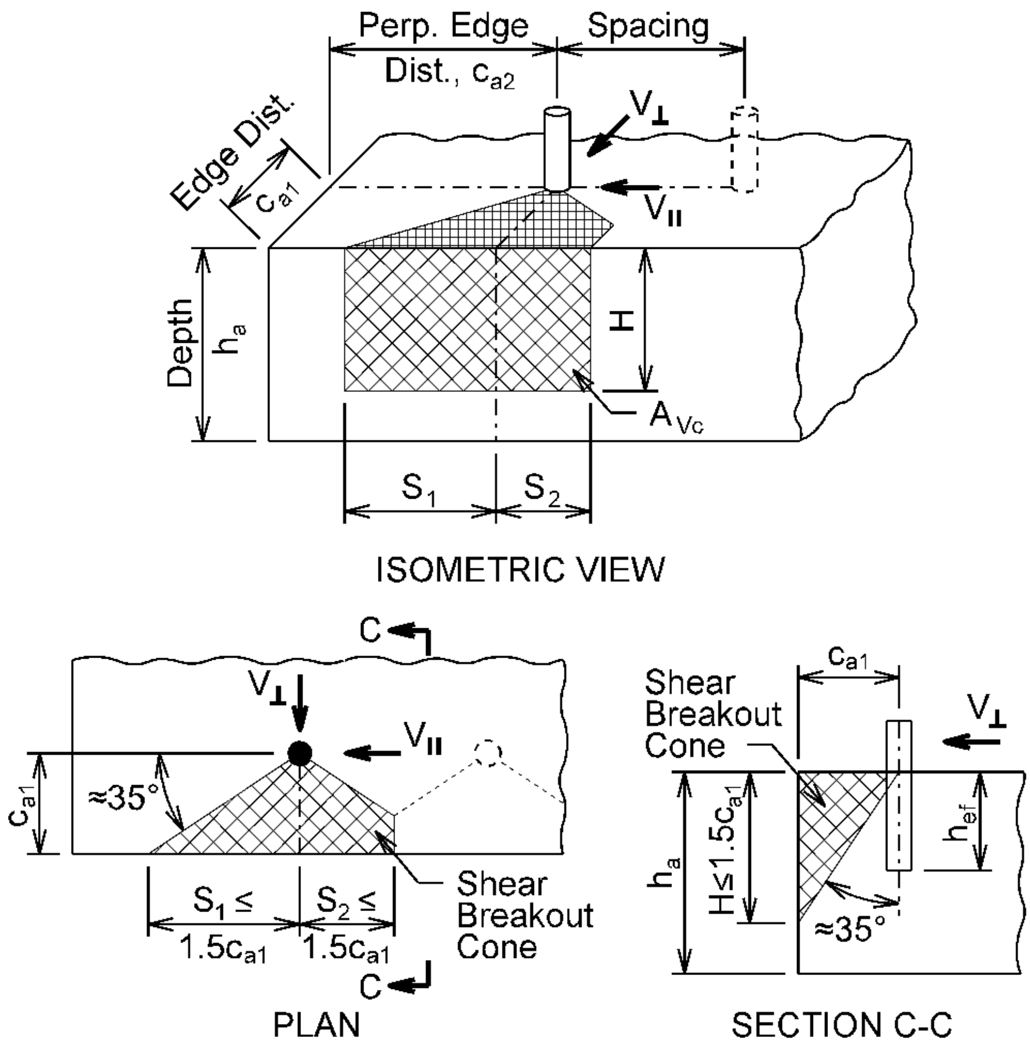

Figure 40.16-3
Concrete Breakout of Concrete Anchors in Shear

If the shear is applied to more than one row of anchors as shown in Figure 40.16-4, the shear capacity must be checked for the worst of the three cases. If the row spacing, SP, is at least equal to the distance from the concrete edge to the front anchor, E1, check both Case 1 and Case 2. In Case 1, the front anchor is checked with the shear load evenly distributed between the rows of anchors. In Case 2, the back anchor is checked for the full shear load. If the row spacing, SP, is less than the distance from the concrete edge to the front anchor, E1, then check Case 3. In case 3, the front anchor is checked for the full shear load. If the anchors are welded to an attachment to evenly distribute the force to all anchors, only Case 2 needs to be checked.

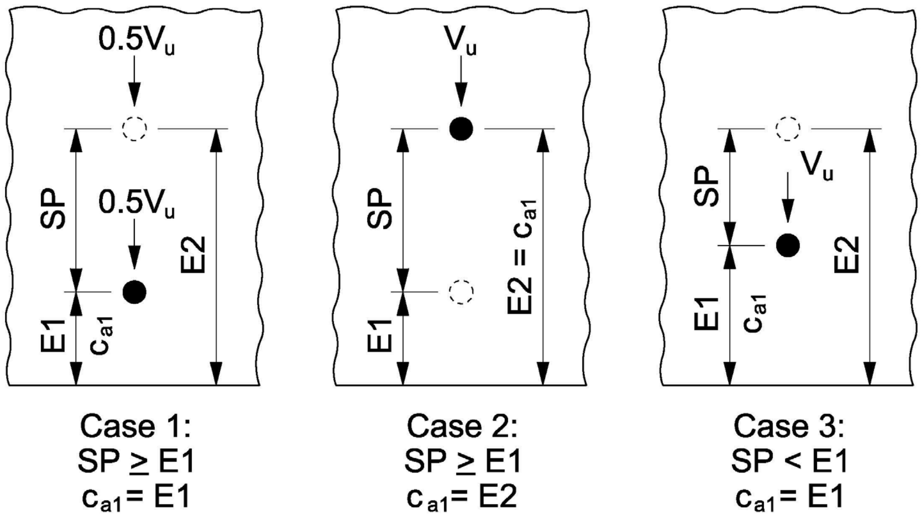

Figure 40.16-4
Concrete Anchor Shear Force Cases

The factored shear force on each anchor, V_u, must be less than or equal to the factored shear resistance, V_r. For mechanical and adhesive anchors:

$$V_r = \phi_{vs} V_{sa} \leq \phi_{vc} V_{cb} \leq \phi_{vp} V_{cp}$$

In which:

ϕ_{vs} = Strength reduction factor for anchors in concrete, **ACI [17.3.3]**
= 0.60 for brittle steel as defined in 40.16.1.1
= 0.65 for ductile steel as defined in 40.16.1.1

V_{sa} = Nominal steel strength of anchor in shear, **ACI [17.5.1.2]**
= $0.6 A_{se,V} f_{uta}$

$A_{se,V}$ = Effective cross-sectional area of anchor in shear (in^2)

ϕ_{vc} = Strength reduction factor for anchors in concrete, **ACI [17.3.3]**
= 0.70 for anchors without supplementary reinforcement per 40.16.2
= 0.75 for anchors with supplementary reinforcement per 40.16.2

V_{cb} = Nominal concrete breakout strength in shear, **ACI [17.5.2.1]**
= $\dfrac{A_{Vc}}{4.5(c_{a1})^2} \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{p,V} V_b$

A_{Vc} = Projected area of the concrete failure surface on the side of the concrete member at its edge for a single anchor, see Figure 40.16-3
= $H(S_1 + S_2)$

c_{a1} = Distance from the center of anchor shaft to the edge of concrete in the direction of the applied shear, see Figure 40.16-3 and Figure 40.16-4 (in)

$\psi_{ed,V}$ = Modification factor for shear strength of anchors based on proximity to edges of concrete member, **ACI [17.5.2.6]**
= 1.0 if $c_{a2} \geq 1.5c_{a1}$ (perpendicular shear)
= $0.7 + 0.3\dfrac{c_{a2}}{1.5c_{a1}}$ if $c_{a2} < 1.5c_{a1}$ (perpendicular shear)
= 1.0 (parallel shear)

c_{a2} = Distance from the center of anchor shaft to the edge of concrete in the direction perpendicular to c_{a1}, see Figure 40.16-3 (in)

$\psi_{c,V}$ = Modification factor for shear strength of anchors based on the presence or absence of cracks in concrete and the presence or absence of supplementary reinforcement, **ACI [17.5.2.7]**
= 1.4 for anchors located in a region of a concrete member where analysis indicates no cracking at service load levels
= 1.0 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels without supplementary reinforcement per 40.16.2 or with edge reinforcement smaller than a No. 4 bar
= 1.2 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels with reinforcement of a No. 4 bar or greater between the anchor and the edge
= 1.4 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels with reinforcement of a No. 4 bar or greater between the anchor and the edge, and with the reinforcement enclosed within stirrups spaced at no more than 4 inches

$\psi_{h,V}$ = Modification factor for shear strength of anchors located in concrete members with $h_a < 1.5_{ca1}$, **ACI [17.5.2.8]**
= $\sqrt{\dfrac{1.5c_{a1}}{h_a}} \geq 1.0$

h_a = Concrete member thickness in which anchor is located measured parallel to anchor axis, see Figure 40.16-3 (in)

$\psi_{p,V}$ = Modification factor for shear strength of anchors based on loading direction, **ACI [17.5]**
= 1.0 for shear perpendicular to the concrete edge, see Figure 40.16-3
= 2.0 for shear parallel to the concrete edge, see Figure 40.16-3

V_b = Concrete breakout strength of a single anchor in shear in cracked concrete, per **ACI [17.5.2.2]** , shall be the smaller of:

$$[7(\frac{l_e}{d_a})^{0.2}\sqrt{d_a}]\sqrt{f'_c}(c_{a1})^{1.5} \text{ (lb)}$$

Where:
$l_e = h_{ef} \leq 8d_a$
d_a = Outside diameter of anchor (in)
f'_c = Specified compressive strength of concrete (psi)

and

$$9\sqrt{f'_c}(c_{a1})^{1.5}$$

ϕ_{vp} = Strength reduction factor for anchors in concrete
= 0.65 for anchors without supplementary reinforcement per 40.16.2
= 0.75 for anchors with supplementary reinforcement per 40.16.2

V_{cp} = Nominal concrete pryout strength of a single anchor, **ACI [17.5.3.1]**
= $2.0N_{cp}$

Note: The equation above is based on $h_{ef} \geq 2.5$ in. All concrete anchors must meet this requirement.

N_{cp} = Nominal concrete pryout strength of an anchor taken as the lesser of:

mechanical anchors: $\frac{A_{Nc}}{9(h_{ef})^2}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$

adhesive anchors: $\frac{A_{Na}}{4(c_{Na})^2}\Psi_{ed,Na}\Psi_{cp,Na}N_{ba}$

and

$$\frac{A_{Nc}}{9(h_{ef})^2}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$$

For shear in two directions, check both the parallel and the perpendicular shear capacity. For shear on an anchor near a corner, check the shear capacity for both edges and use the minimum.

40.16.5 Interaction of Tension and Shear

For anchors that are subjected to tension and shear, interaction equations must be checked per **ACI [17.6]**.

If $\frac{V_{ua}}{\phi V_n} \leq 0.2$ for the governing strength in shear, then the full strength in tension is permitted: $\phi N_n \geq N_{ua}$. If $\frac{N_{ua}}{\phi N_n} \leq 0.2$ for the governing strength in tension, then the full strength in shear is permitted: $\phi V_n \geq V_{ua}$. If $\frac{V_{ua}}{\phi V_n} > 0.2$ for the governing strength in shear and $\frac{N_{ua}}{\phi N_n} > 0.2$ for the governing strength in tension, then:

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2$$

40.16.6 Plan Preparation

The required minimum pullout capacity (as stated on the plans) is equal to the Nominal Tensile Resistance as determined in 40.16.3.

Typical notes for bridge plans (shown in all capital letters):

Adhesive anchors located in uncracked concrete:

ADHESIVE ANCHORS X/X-INCH (or No. X BAR). EMBED XX" IN CONCRETE. *(Illustrative only, values must be calculated depending on the specific situation).*

Adhesive anchors located in cracked concrete:

ADHESIVE ANCHORS X/X-INCH (or No. X BAR). EMBED XX" IN CONCRETE. ANCHORS SHALL BE APPROVED FOR USE IN CRACKED CONCRETE. *(Illustrative only, values must be calculated depending on the specific situation).*

When using anchors to anchor bolts or studs, the bolt, nut, and washer or the stud as detailed on the plans is included in the bid item “Adhesive Anchors _-Inch”.

For anchors using rebar, the rebar should be listed in the “Bill of Bars” and paid for under the bid item “Bar Steel Reinforcement HS Coated Structures”.

When adhesive anchors are used as an alternative anchorage the following note should be included in the plans:

> ADHESIVE ANCHORS SHALL CONFORM TO SECTION 502.2.12 OF THE STANDARD SPECIFICATION. *(Note only applicable when the bid item Adhesive Anchor is not used).*

It should be noted that AASHTO is considering adding specifications pertaining to concrete anchors. This chapter will be updated once that information is available.

40.17 Plan Details

1. Excavation for Structures on Overlays

   There is considerable confusion on when to use or not use the bid item “Excavation for Structures” on overlay projects. In order to remove the confusion, the following note is to be added to all overlay projects that only involve removal of the paving block (or less).

   > Any excavation required to complete the overlay or paving block at the abutments is to be considered incidental to the bid item “(insert applicable bid item)”.

   If the overlay project has any excavation other than the need to replace a paving block or prepare the end of the deck for the overlay, the “Excavation for Structures” bid item should be used and the above note left off the plan.

2. For steel girder bridge deck replacements, show the existing exterior girder size including the lower flange size on the plans to assist the contractor in ordering falsework brackets.

3. On structure deck overlay projects:

   Verify desired transverse or roadway cross slopes with Regions considering the changes in bridge rating and approach pavement grade. Although relatively flat by current standard of a 0.02 ft/ft cross slope, a cross slope of 0.01 ft/ft or 0.015 ft/ft may be the most desirable.

   The designer should evaluate 3 types of repairs. “Preparation Decks Type 1” is concrete removal to the top of the bar steel. “Preparation Decks Type 2” is concrete removal below the bar steel. “Full Depth Deck Repair” is full depth concrete removal and repair. The designer should compute the quantity and cost for each type and a total cost. Compare the total cost to the estimated cost of a deck replacement and proceed accordingly. Show the location of “Full Depth Deck Repair” on the plan sheet.

   Contractors have gotten projects where full depth concrete removal was excessive and no locations designated. It would have been a better decision to replace the deck.

4. When detailing two stage concrete deck construction, consider providing pier cap construction joints to coincide with the longitudinal deck construction joint. Also, transverse deck bar steel lap splicing is preferred over the use of bar couplers. This is applicable where there is extra bridge width and the falsework can be left for both deck pours.

5. Total Estimated Quantities

   The Region should provide the designer with a Rehabilitation Structure Survey Report that provides a complete description of the rehabilitation and estimated quantities. Contact the Region for clarifications on the scope of work.

   Additional items:

- Provide deck survey outlining areas of distress (if available). These plans will serve as documentation for future rehabilitations.

- Distressed areas should be representative of the surveyed areas of distress. Actual repairs will likely be larger than the reported values while removing all unsound materials.

- Provide Preparation Deck Type 1 & 2 and Full-Depth Repair estimates for areas of distress.

- Coordinate asphaltic materials with the Region and roadway designers.

See Chapter 6-Plan Preparation and Chapter 40 Standards for additional guidance.

40.18 Retrofit of Steel Bridges

Out of plane bending stresses may occur in steel bridges that introduce early fatigue cracks or worse yet brittle fractures. Most, if not all, problems are caused at connections that are either too flexible or too rigid. It is important to recognize the correct condition as retrofitting for the wrong condition will probably make it worse.

40.18.1 Flexible Connections

A connection is too flexible when minor movement can occur in a connection that is designed to be rigid. Examples are transverse or bearing stiffeners fitted to a tension flange, floorbeams attached to the web only and not both flanges.

The solution for stiffeners and transverse connection plates is to shop weld the stiffener to the web and flanges. This becomes a Category C weld but is no different than the terminated web to stiffener weld. If the condition exists in the field, welding is not recommended. Retrofit is to attach a T-section with four bolts to the stiffener and four bolts to the flange. Similar details should be used in attaching floorbeams to the girder.

40.18.2 Rigid Connections

A connection is too rigid when it is fitted into place and allowed to move but the movement can only occur in a refined area which introduces high stresses in the affected area. Examples are welded gusset connection plates for lower lateral bracing that are fitted around transverse or bearing stiffeners.

Other partial constraint details are:

1. Intersecting welds
2. Gap size-allowing local yielding
3. Weld size
4. Partial penetration welds versus fillet welds
5. Touching and intersecting welds

The solution is to create spaces large enough (approximately 1/4” or more) for more material to flex thus reducing the concentration of stress. For gusset connection plates provide a larger gap than 1/4” and no intersecting welds. For existing conditions it may be necessary to drill holes at high stress concentrations. For new conditions it would be better to design a rigid connection and attach to the flange rather than the web. For certain situations a fillet weld should be used over a partial penetration weld to allow slight movement.

40.19 Reinforcing Steel for Deck Slabs on Girders for Deck Replacements

Effective Span Ft-In	T=Slab Thickness Inches	Transverse Bars & Spacing	Longitudinal Bars & Spacing	Longitudinal* Continuity Bars & Spacing
4-0	6.5	#5 @ 8"	#4 @ 8.5"	#5 @ 7.5"
4-3	6.5	#5 @ 7.5"	#4 @ 7.5"	#5 @ 7.5"
4-6	6.5	#5 @ 7.5"	#4 @ 7.5"	#5 @ 7.5"
4-9	6.5	#5 @ 7"	#4 @ 7.5"	#5 @ 7.5"
5-0	6.5	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
5-3	6.5	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
5-6	6.5	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
5-9	6.5	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
2-3	7	#4 @ 9"	#4 @ 11"	#5 @ 6.5"
2-6	7	#4 @ 8.5"	#4 @ 11"	#5 @ 6.5"
2-9	7	#4 @ 8"	#4 @ 11"	#5 @ 6.5"
3-0	7	#4 @ 7.5"	#4 @ 11"	#5 @ 6.5"
3-3	7	#4 @ 7"	#4 @ 11"	#5 @ 6.5"
3-6	7	#4 @ 6.5"	#4 @ 11"	#5 @ 6.5"
3-9	7	#4 @ 6.5"	#4 @ 10.5"	#5 @ 6.5"
4-0	7	#4 @ 6"	#4 @ 10"	#5 @ 6.5"
4-3	7	#5 @ 9"	#4 @ 9.5"	#5 @ 7"
4-6	7	#5 @ 8.5"	#4 @ 9"	#5 @ 7"
4-9	7	#5 @ 8"	#4 @ 8.5"	#5 @ 7"
5-0	7	#5 @ 8"	#4 @ 8.5"	#5 @ 7"
4-3	7	#5 @ 7.5"	#4 @ 8"	#5 @ 7"
5-6	7	#5 @ 7'	#4 @ 7"	#5 @ 7"
5-9	7	#5 @ 7"	#4 @ 7"	#5 @ 7"
6-0	7	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
6-3	7	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
6-6	7	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
6-9	7	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
7-0	7	#5 @ 6"	#4 @ 6"	#5 @ 6"
4-0	7.5	#4 @ 7"	#4 @ 10.5"	#5 @ 6"
4-3	7.5	#4 @ 6.5"	#4 @ 10.5"	#5 @ 6"
4-6	7.5	#4 @ 6.5"	#4 @ 10"	#5 @ 6"
4-9	7.5	#4 @ 6"	#4 @ 10"	#5 @ 6"
5-0	7.5	#5 @ 9"	#4 @ 9.5"	#5 @ 6"
5-3	7.5	#5 @ 8.5"	#4 @ 9"	#5 @ 6.5"

5-6	7.5	#5 @ 8"	#4 @ 8.5"	#5 @ 6.5"
5-9	7.5	#5 @ 8"	#4 @ 8.5"	#5 @ 6.5"
6-0	7.5	#5 @ 7.5"	#4 @ 8"	#5 @ 6.5"
6-3	7.5	#5 @ 7.5"	#4 @ 7.5"	#5 @ 6.5"
6-6	7.5	#5 @ 7"	#4 @ 7.5"	#5 @ 6.5"
6-9	7.5	#5 @ 7"	#4 @ 6.5"	#5 @ 6.5"
7-0	7.5	#5 @ 7"	#4 @ 6.5"	#5 @ 6.5"
7-3	7.5	#5 @ 6.5"	#4 @ 6.5"	#5 @ 6.5"
7-6	7.5	#5 @ 6.5"	#5 @ 10"	#5 @ 6.5"
7-9	7.5	#5 @ 6"	#5 @ 10"	#5 @ 6.5"
8-0	7.5	#5 @ 6"	#5 @ 10"	#5 @ 6.5"
8-3	7.5	#5 @ 6"	#5 @ 9.5"	#5 @ 6.5"

Table 40.19-1

Reinforcing Steel for Deck Slabs on Girders for Deck Replacements – HS20 Loading

Max. Allowable Design Stresses: f_c' = 4000 psi, f_y = 60 ksi, Top Steel 2-1/2" Clear, Bottom Steel 1-1/2" Clear, Future Wearing Surface = 20 lbs/ft. Transverse and longitudinal bars shown in table are for one layer only. Place identical steel in both top and bottom layer, except in negative moment region. Use in top layer for slab on steel girders in negative moment region when not designed for negative moment composite action.

40.20 Fiber Reinforced Polymer (FRP)

40.20.1 Introduction

Fiber reinforced polymer (FRP) material is a composite composed of fibers encased in a polymer matrix. The fibers provide tensile strength while the resin protects the fibers and transfers load between them. FRP can be used to repair or to retrofit bridges. Repair is often defined as returning a member to its original condition after damage or deterioration while retrofitting refers to increasing the capacity of a member beyond its original capacity.

For plan preparations, FRP repairs and retrofits are categorized as either structural strengthening or non-structural protection. Contact the Bureau of Structures Design Section for current Special Provisions and for other FRP considerations.

40.20.2 Design Guidelines

While there is no code document for the design of FRP repairs and retrofits, there are two nationally recognized design guidelines: the *Guide Specification for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements* (14.) hereinafter referred to as the AASHTO FRP Guide, and the *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures ACI 440.2R-08 (15.)* hereinafter referred to as the ACI FRP Guide.

Note: BOS has been evaluating the design methodologies found in the AASHTO FRP Guide and ACI RFP Guide. Noticeable differences between the guides warrants further investigation, with input from industry representation. FRP repairs and retrofits shall be in accordance with the applicable Special Provisions.

40.20.3 Applicability

Not all concrete structures can be retrofitted or repaired using FRP. Most FRP research has been conducted on normal sized members, therefore many of the design equations cannot be used with exceptionally large or deep members. Additionally, members with disturbed regions (D-regions) such as deep beams and corbels are outside of the scope of many design equations.

The structure must have some amount of load carrying capacity prior to the installation of the FRP. Due to the potential for premature debonding, FRP cannot be counted on to carry service loads; it may only be used increase the ultimate capacity of the structure for strength and extreme event load cases. The unrepaired or unretrofitted structure be able to carry the service dead and live loads:

$$R_r \geq \eta_i[(DC + DW) + (LL + IM)]$$

Where:

R_r = factored resistance computed in accordance with AASHTO LRFD Section 5

η_i = load modifier = 1.0

DC = force effects due to components and attachments

DW = force effects due to wear surfaces and utilities

LL = force effects due to live load

IM = force effects due to dynamic load allowance

If capacity is added in flexure to accommodate increased loads, the shear capacity of the member must be checked to ensure that it is still sufficient for the new loading. For non-structural FRP applications, applicability checks may not be required.

40.20.4 Materials

A typical FRP system consists of a primer, fibers, resin, bonding material (either additional resin or an adhesive), and a protective coating. FRP is specified in terms of the types of fiber and resin, the number of layers, the fiber orientation and the geometry. FRP is sold as a system and all materials used should be from the same system.

40.20.4.1 Fibers

The most common types of fiber used for bridge repairs are glass and carbon. Glass fibers are not as stiff or as strong as carbon, but they are much less expensive. Unless there is reason to do otherwise, it is recommended that glass fibers be used for corrosion protection and spall control. Carbon fibers should be used for strengthening and crack control.

Carbon fibers cannot be used where the FRP comes into contact with steel out of concerns for galvanic corrosion due to the highly conductive nature of carbon fibers. For applications where galvanic corrosion is a concern, glass fibers may be used, even in structural applications.

Often, FRP is requested by the region to provide column confinement. The engineer must determine if the requested confinement is true confinement where the FRP puts the column into triaxial compression to increase the capacity and ductility, or if the FRP is confining a patch from spalling off. In the case of true confinement (which is very rare in Wisconsin), carbon fibers should be used and the repair requires structural design. For spall control, glass fibers are acceptable and the repair is considered non-structural.

40.20.4.2 Coatings

After the FRP has been installed and fully cured, a protective coating is applied to the entire system. A protective coating is needed to protect against ultraviolet degradation and can also provide resistance to abrasion, wear, and chemicals. In situations where the FRP is submerged in water, inert protective coatings can help prevent compounds in the FRP from leaching into the water, mitigating environmental impacts.

Protective coatings can be made from different materials depending on the desired coating characteristics. Common coating types include vinyl ester, urethane, epoxy, cementitious, and acrylic. Acrylic coatings are generally the least expensive and easiest to apply, though they may also be less durable. If no coating type is specified, it is likely that the manufacturer will provide an acrylic coating.

For shorter term repairs, acrylic coatings are sufficient, but longer repairs should consider other coating types such as epoxy. Any coating used must be compatible with the FRP system selected by the contractor.

40.20.4.3 Anchors

The bond between the FRP and the concrete is the most critical component of an FRP installation and debonding is the most common FRP failure mode. Certain FRP configurations use anchors to increase the attachment of the FRP and attempt to delay or prevent debonding. These anchors can consist of near surface mounted bars, fiber anchors, additional FRP strips, or mechanical anchors such as bolts. It is permitted to use additional U-wrap strips to anchor flexural FRP, but the use of additional longitudinal strips to anchor shear FRP is prohibited. The use of additional U-wrap strips for flexural anchorage is required in some instances.

Because neither design guide requires anchorage or provides information as to what constitutes anchorage, it is left to the discretion of the designer to determine if anchorage should be used and in what quantities. The use of anchors is highly encouraged, particularly for shear applications and in situations where there is increased potential for debonding such as reentrant corners.

Specifying anchors will add cost to the repair, but these costs may be offset by increased capacity accorded to anchored systems in shear. The additional costs can also be justified if debonding is a concern. If the designer chooses to use anchors, anchors should be shown on plans, but the design of the anchors is left to the manufacturer.

40.20.5 Flexure

Flexural FRP is applied along the tension face of the member, where it acts as additional tension reinforcement. The fibers should be oriented along the length of the member.

40.20.5.1 Pre-Design Checks

If the design of the FRP will be provided by the contractor or their consultant, the engineer must still perform certain checks before specifying FRP in the plans. For flexure, the engineer must check that the structure has sufficient capacity to carry service loads without additional capacity from the FRP, as discussed in 40.20.3.

40.20.5.2 Composite Action

Composite action of the deck slab can be considered when designing flexural FRP repairs for girders, provided that the deck was designed to be composite. If composite action is considered, composite section properties must be computed. These properties should be substituted into the design equations presented in this section. Accounting for composite action will increase the capacity provided by the FRP.

40.20.5.3 Pre-Existing Substrate Strain

Unless all loads are removed from the member receiving FRP (including self-weight), there will be strain present in the concrete when the FRP is applied. This initial or pre-existing substrate strain ε_{bi} is computed through elastic analysis. All loads supported by the member during FRP installation should be considered and cracked section properties should be considered if necessary.

40.20.5.4 Deflection and Crack Control

Conduct standard LRFD serviceability checks for deflection and crack control while accounting for the contribution of the FRP. Because both the FRP and the concrete will be in the elastic zone at service levels, standard elastic analysis can be used to determine stresses and strains. Transformed section analysis can be used to transform the FRP into an equivalent area of concrete for the purposes of analysis. The condition of the member determines if the cracked or uncracked section properties should be used in computations.

40.20.6 Shear

In shear repair/retrofitting applications, the FRP acts essentially as external stirrups. The FRP wrap is applied with the fibers running transverse to the member.

40.20.6.1 Pre-Design Checks

If the design of the FRP will be provided by the contractor or their consultant, the engineer must still perform certain checks before specifying FRP in the plans. For shear, the engineer must check that the structure has sufficient capacity to carry service loads without additional capacity from the FRP, as discussed in 40.20.3.

Additionally, the engineer must ensure that the amount of FRP capacity required does not exceed the maximum allowable shear reinforcement. <u>It is important to note that the FRP capacity listed on the plans will be a factored capacity</u>, while the maximum allowable shear reinforcement check is for an unfactored capacity. Strength reduction factors must be incorporated to make a proper comparison.

If the FRP capacity is close to the maximum allowed, the designer must take care to ensure that a design is feasible. The capacity provided by FRP depends on the number of FRP layers, with each additional layer providing a discrete increase in capacity. There may be a situation where n layers does not provide enough capacity, but n+1 layers provides too much capacity and violates the maximum allowable shear reinforcement criteria. Changes in spacing of the wraps may help decrease the capacity provided by the FRP.

Example problems in shear can be found in the appendices of NCHRP Report 655 (16) and potential shear wrapping configurations can be found in NCHRP Report 678 (17).

40.21 References

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18. Agrawal, A.K., Jalinoos, F., Davis, N., Hoomaan, E., Sanayei, M. (2018). *Foundation Reuse for Highway Bridges,* FHWA-HIF-18-055, Federal Highway Administration, Washington, D.C.

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Table of Contents

43.1 Overview

This chapter is one part of a larger structures asset management program.

Chapter 42 – Bridge Preservation: Establishes program-level goals, objectives, measures, and strategies for the preservation and maintenance of bridges in Wisconsin and serves as the policy foundation for this chapter. Work actions and strategies detailed in Chapter 42 are incorporated in both Chapter 41 and 43.

Chapter 41 – Structures Asset Management: Focuses on implementing the philosophy outlined in Chapter 42. More specifically, Chapter 41 details the process to deliver preservation, rehabilitation, and replacement projects through the improvement program.

Chapter 43 – Structures Asset Management; Maintenance Work: Similar to Chapter 41, as this chapter also focuses on implementing the philosophy outlined in Chapter 42. However, the chapter provides the policy, procedure, and workflow for those bridge preservation and bridge maintenance actions most often performed through the annual Highway Maintenance Work Plan (HMWP). These actions complement work performed through the improvement program.

Work identified in this chapter is critical to a fully-functioning bridge asset management program. A given bridge will not achieve its maximum potential lifespan without the type of work detailed in this chapter. This is illustrated in Figure 43.1-1.

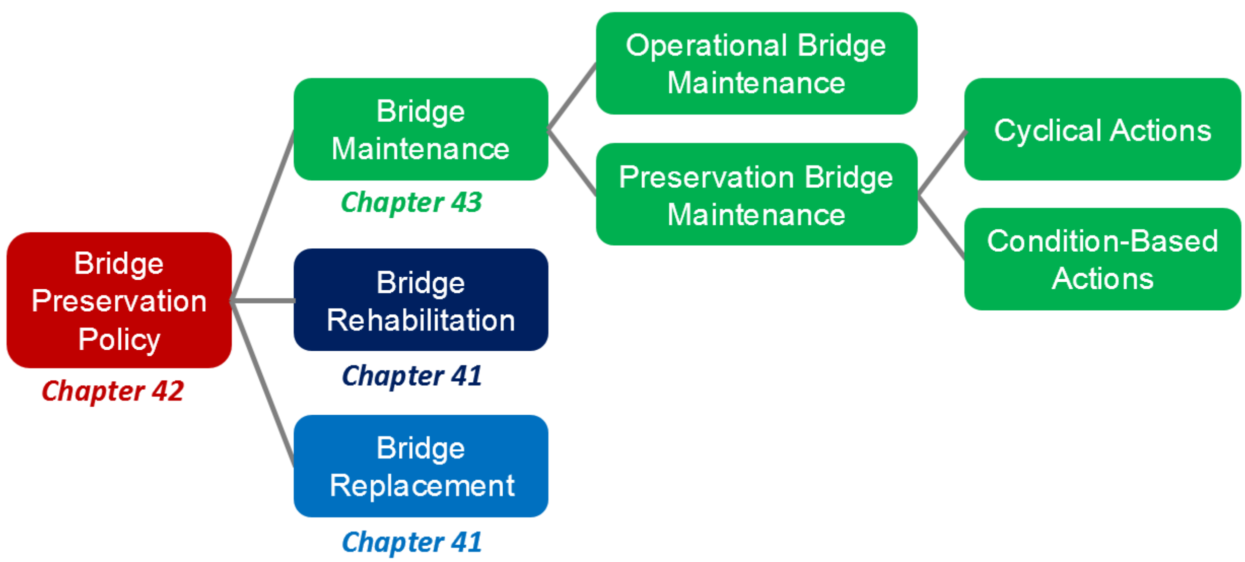

Figure 43.1-1
Bridge Asset Management Work Activities

43.1.1 Highway and Bridge Maintenance Work Plan

The Highway Maintenance Work Plan is coordinated by the Bureau of Highway Maintenance (BHM) and incorporates many different activities and subject areas. The Bridge Maintenance Work Plan is one piece of the overall Highway Maintenance Work Plan, as shown in Figure 43.1-2. The Bureau of Structures is responsible for the technical direction associated with the Bridge Maintenance Work Plan.

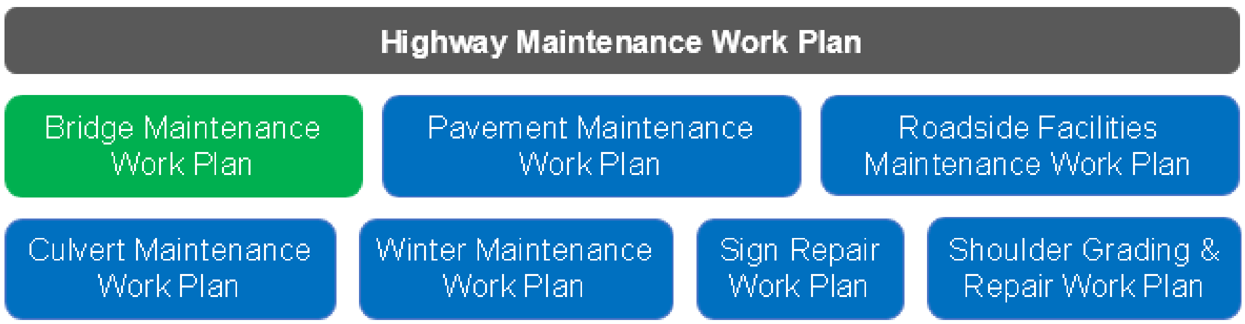

Figure 43.1-2
Components of the Highway Maintenance Work Plan

43.1.2 WisDOT Roles and Responsibilities for Bridge Maintenance

A well-defined bridge asset management program helps the Department to direct its available types of funding in the most optimal ways to achieve maximum bridge life. Bridge maintenance actions (including bridge preservation actions) are critical to maximizing the effectiveness of the Department's bridge asset management program, and thus it is critical that roles and responsibilities in bridge maintenance are clearly defined and optimally applied.

Bureau of Structures

The Bureau of Structures (BOS) maintains and updates the comprehensive preservation policy for structures (Chapter 42 – Bridge Preservation). BOS develops and maintains the Highway Structures Information System (HSIS), a database of structures information, including condition information (inspection reports). BOS also develops and maintains the Wisconsin Structures Asset Management System (WiSAMS -see 41.2.1), a software tool used to forecast needed structures work. Together, these tools facilitate identification of structure work for both the improvement program and the highway maintenance work plan.

For bridge preservation, cyclical actions along with some limited condition-based actions are the work types that have traditionally fallen within the funding authority and workforce ability of the WisDOT highway maintenance work plan.

Bureau of Highway Maintenance

The Bureau of Highway Maintenance (BHM) is the lead for allocating available funding across program and asset types, including bridge maintenance work. After Region allocations are determined, BHM is responsible for ensuring that each region is setting up a workplan that is in alignment with those allocations and Department priorities.

Regional Bridge Maintenance

Regional Bridge Maintenance engineers are the primary contact between WisDOT and the country service providers that perform the actions detailed in the Highway Maintenance Work Plan. Regional Bridge Maintenance works with BOS to develop and prioritize the work plan (see 43.3.2 and 43.3.3). Regional Bridge Maintenance engineers are also the primary contact for documentation of work performed by the county service providers.

Regional Programing

Regional Programing engineers work with BOS and Regional Bridge Maintenance engineers to pull bridge maintenance work into the let improvement program as appropriate.

Local Service Providers

Local service providers (primarily country work crews) are the labor force that performs the work detailed in the Highway Maintenance Work Plan. They are responsible for completing work and providing proper documentation to WisDOT after it is complete.

43.2 Bridge Maintenance Actions for Asset Management

Through strategic use of structure inventory data (stored in HSIS), well-documented preservation policies (see Chapter 42), and WISAMS asset management algorithms, WisDOT has the ability to optimally align bridge work activities with the appropriate maintenance and improvement programs to coordinate appropriate treatment actions throughout the lifecycle of a structure.

43.2.1 Operational Bridge Maintenance Actions

Operational bridge maintenance actions are those actions necessary for the regular operation of a bridge. These actions are expected and are necessary to maintain a bridge in serviceable condition. Bridge preservation activities such as those described in Chapter 42 may lessen the amount or frequency of operational maintenance, but it will not be eliminated. Examples include:

- Cutting brush
- Patching/filling spalls
- Hot-rubbering end joints
- Joint gland replacements
- Channel debris removal
- Washout/erosion repair
- Retrofitting fatigue cracks

These actions are performed on an "as-needed" basis; some may require immediate or near-immediate action to maintain a safe and serviceable structure. That being the case, operational bridge maintenance items may take priority over all other maintenance in terms of timing and funding. These items are most often captured in the "maintenance items" recorded by the bridge inspector in the inspection report. These items are collected and stored in HSIS.

It should also be noted that time-critical repairs (deck patching, bridge hit response) are also considered operational bridge maintenance actions. Because of their nature, they are not identified in advance, but rather addressed immediately as the need arises.

Operational bridge maintenance actions are identified by the bridge inspector, except for time-critical repairs.

Operational bridge maintenance actions are most typically funded via Routine Maintenance Agreements (RMA).

43.2.2 Preservation Bridge Maintenance Actions

Preservation bridge maintenance actions are those aimed at extending the usable life of the given bridge. This work generally falls into two categories; cyclical and condition-based work actions.

43.2.2.1 Cyclical Work Actions

Cyclical maintenance occurs on a regular schedule and thus are a regular component of the annual Bridge Maintenance Work Plan. Cyclical work actions are performed as a preventative measure to attempt to slow deterioration and extend structure life. One example of a cyclical work action is deck washing; the intent is to remove chlorides (salts) from the deck, which accelerate deck deterioration. See Chapter 42 for more information.

Cyclical work actions are identified by BOS and verified/modified by Regional Bridge Maintenance.

Cyclical work actions are most typically funded via RMA.

43.2.2.2 Condition-Based Work Actions

Condition-based maintenance occurs irregularly based on the specific condition of an individual structure. The work action is only performed when a specific need is identified, and the work is performed to address the deficiency. One example of a condition-based work action is crack sealing.

Condition-based work actions are most commonly identified based on specific condition data (inspection reports). The work is typically identified by BOS and included in the unconstrained needs list (see 43.3.1), See Chapter 42 for more information.

Though not as common, condition-based work actions can be identified by the bridge inspector. In general, BOS and Region Bridge Maintenance engineers collaborate to determine the appropriate condition-based actions.

Condition-based work actions are funded by either Routine Maintenance Agreements (RMA), Discretionary Maintenance Agreements (DMA), and Performance-Based Maintenance (PBM); RMA is most typical.

43.2.3 Delivery Mechanisms for Bridge Maintenance Work

The general delivery mechanism for the overall structures program is shown in Figure 43.2-1.

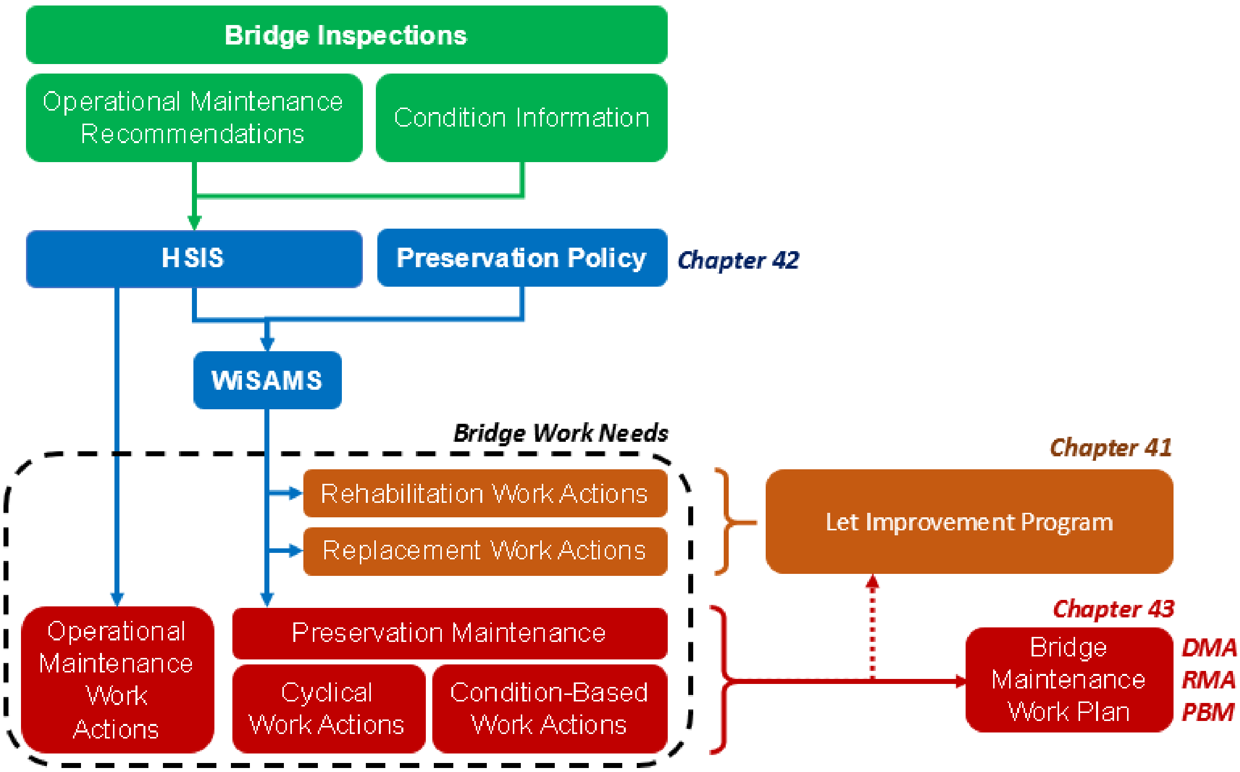

Figure 43.2-1
Overall Structures Program Delivery

Highway maintenance program funding and work force resources represent the bottom (red) portion of this diagram. The focus is on bridge maintenance, including both preservation and operational maintenance actions. Funding for bridge maintenance work comes from three primary sources; Routine Maintenance Agreements (RMA), Discretionary Maintenance Agreements (DMA), and Performance-Based Maintenance (PBM). This is illustrated in Figure 43.2-2.

Federal and state rules prohibit use of federal funding on certain preservation and maintenance activities and use of state maintenance funding on certain activities. The direction for eligibility of federal funds is outlined in FDM 3-1 Exhibit 5.2 - *Agreement for the Use of Federal Funds for Preventive Maintenance of Structures.*

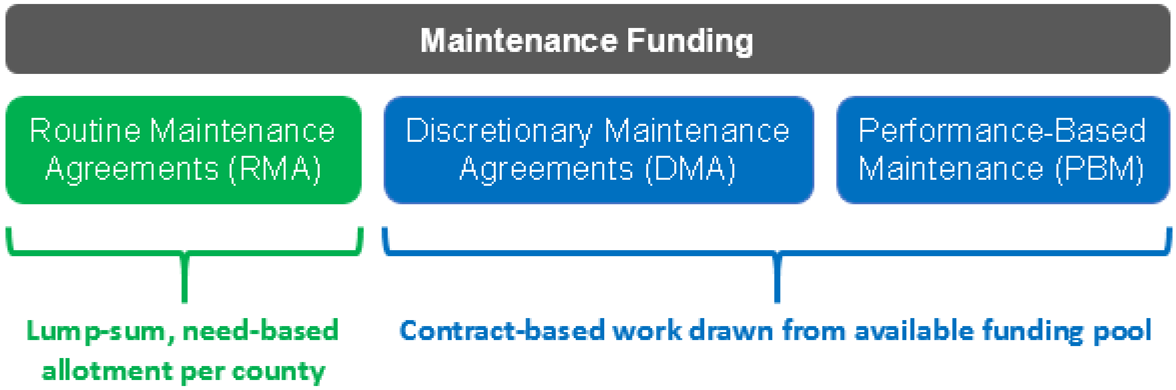

Figure 43.2-2
Maintenance Funding Mechanisms

43.3 Bridge Maintenance Work Plan Development

This section details how the Bridge Maintenance Work Plan is developed, including the parties involved and interim milestone deadlines. It's important to note that the process described here is based on the calendar year (CY), not fiscal year (FY).

43.3.1 Unconstrained Needs Identification

Using data in HSIS, WiSAMS, and maintenance items identified in inspection reports, BOS will generate a list of unconstrained maintenance needs; both operational and preservation maintenance work actions, without consideration of any fiscal constraints. This will be referred to as the Unconstrained Bridge Maintenance Needs List.

- Timeline: The Unconstrained Bridge Maintenance Needs list for the upcoming calendar year is distributed to Regional Bridge Maintenance no later than January 15.

The format of the Unconstrained Bridge Maintenance Needs List will remain intact through the entire annual cycle of the Bridge Maintenance Work Plan. This list will be used to track the changing status of the work identified and provide the data to update HSIS and produce annual maintenance program reports.

43.3.2 Draft Bridge Maintenance Work Plan

Regions use the Unconstrained Bridge Maintenance Needs List as the starting point to develop the Draft Bridge Maintenance Work Plan. Region Bridge Maintenance Engineers review the list and use on-site knowledge to edit the list, adding or deleting as needed. The Draft Bridge Maintenance Work Plan is subject to approval from BOS to ensure compliance with asset management philosophies. Prioritization and evaluation for funding that follow are primarily the responsibility of Region personnel, with input from BOS as appropriate.

- Timeline: The Draft Bridge Maintenance Work Plan is completed by February 1 of the calendar year (CY) for the work plan.

43.3.3 Bridge Maintenance Work Plan Prioritization

BHM is responsible for managing funding for the overall Highway Maintenance Work Plan. This includes funding for the Bridge Maintenance Work Plan, but also includes pavement work, winter maintenance, etc. BHM assembles all proposed work for the Highway Maintenance Work Plan and works with Region Bridge Maintenance to determine the appropriate funding level for the Bridge Maintenance Work Plan.

With the funding level determined, Region Bridge Maintenance will take the Draft Bridge Maintenance Work Plan and prioritize the list. Prioritization is done taking into account items such as (but not exclusively):

- Safety of the travelling public
- Planned future improvement work

- Required traffic control / impacts on traffic
- Criticality of the individual bridge (type, size, condition, ADT, etc.)
- Capabilities of the local service provider in question

Regional Bridge Maintenance should provide a brief explanation for work actions that did not prioritize. This information will be used in needs identification and prioritization (see 43.5).

- Timeline: The Prioritized Bridge Maintenance Work Plan is completed by February 15 of the CY for the work plan.

43.3.4 Evaluation for Appropriate Funding Source

Region Bridge Maintenance engineers and Region Program engineers review the Prioritized Bridge Maintenance Work Plan to determine the most appropriate funding type to address each identified work action. This splits the Prioritized Bridge Maintenance Work Plan into either the Improvement Program or Highway Maintenance Work Plan (RMA, PBM, DMA). The work remaining after items are pulled into the improvement program represents the Final Bridge Maintenance Work Plan.

- Timeline: Evaluation for funding source and development of the Final Bridge Maintenance Work Plan is completed no later than March 1 of CY for the work plan.

Figure 42.3-1 shows the Bridge Maintenance Work Plan development timeline.

Deliverable	Responsible Party	Timeline
Unconstrained Bridge Maintenance Needs List	• Bureau of Structures	January 15
Draft Bridge Maintenance Work Plan	• Region Maintenance • Bureau of Structures	February 1
Prioritized Bridge Maintenance Work Plan	• Bureau of Highway Maintenance • Region Maintenance	February 15
Final Bridge Maintenance Work Plan	• Region Maintenance • Region Planning	March 1

Figure 42.3-1
Bridge Maintenance Work Plan Development Timeline

43.3.5 Delivering the Bridge Maintenance Work Plan

Following prioritization, additional fields are added to the Prioritized Bridge Maintenance Work Plan for the local service providers to document work completed. This document represents the Final Maintenance Work Plan. The contracts developed with the county service providers for bridge preservation work should include the Final Bridge Maintenance Work Plan as an attachment to the "Scope of Work". This will help ensure:

- Accuracy in specific work requests to the county.
- A mechanism to track the progress and completions of work.
- A method to support invoicing by the county for work completed.
- A method to document specific bridge maintenance work performed in HSIS.

The Bureau of Highway Maintenance is the responsible party for program management for invoicing and payment. Region Bridge Maintenance is the lead for contracting with the local service providers and approving the actual work performed. Region Bridge Maintenance also acts as the technical lead, providing direction and feedback as required. Region Maintenance is also the point-of-contact for collecting and verifying documentation (as needed); see 43.4 below for more information.

- Timeline: The Bridge Maintenance Work Plan is delivered to the county service providers no later than March 15 of the CY for the work plan.

43.4 Documentation of Completed Bridge Maintenance Work

Local service providers shall submit a copy of the Final Bridge Maintenance Work Plan to Region Bridge Maintenance. The work plan includes areas to document information related to completed work items. It should be noted that the Final Bridge Maintenance Work Plan includes columns to capture cost data. This information is critical. As WisDOT refines the structures asset management program, this cost data will be a parameter in cost-benefit analysis and resource allocation decisions.

Region Bridge Maintenance is the lead for working with the local service providers to collect the completed Final Bridge Maintenance Work Plan. Region Maintenance will review and verify (as necessary) and then submit to BOS for inclusion in HSIS.

Timeline: Documentation is complete and back to Region Bridge Maintenance by November 15. Regions review, verify (as needed) and deliver documentation to BOS by December 15.

An overview of the entire process is shown below in Figure 43.4-1.

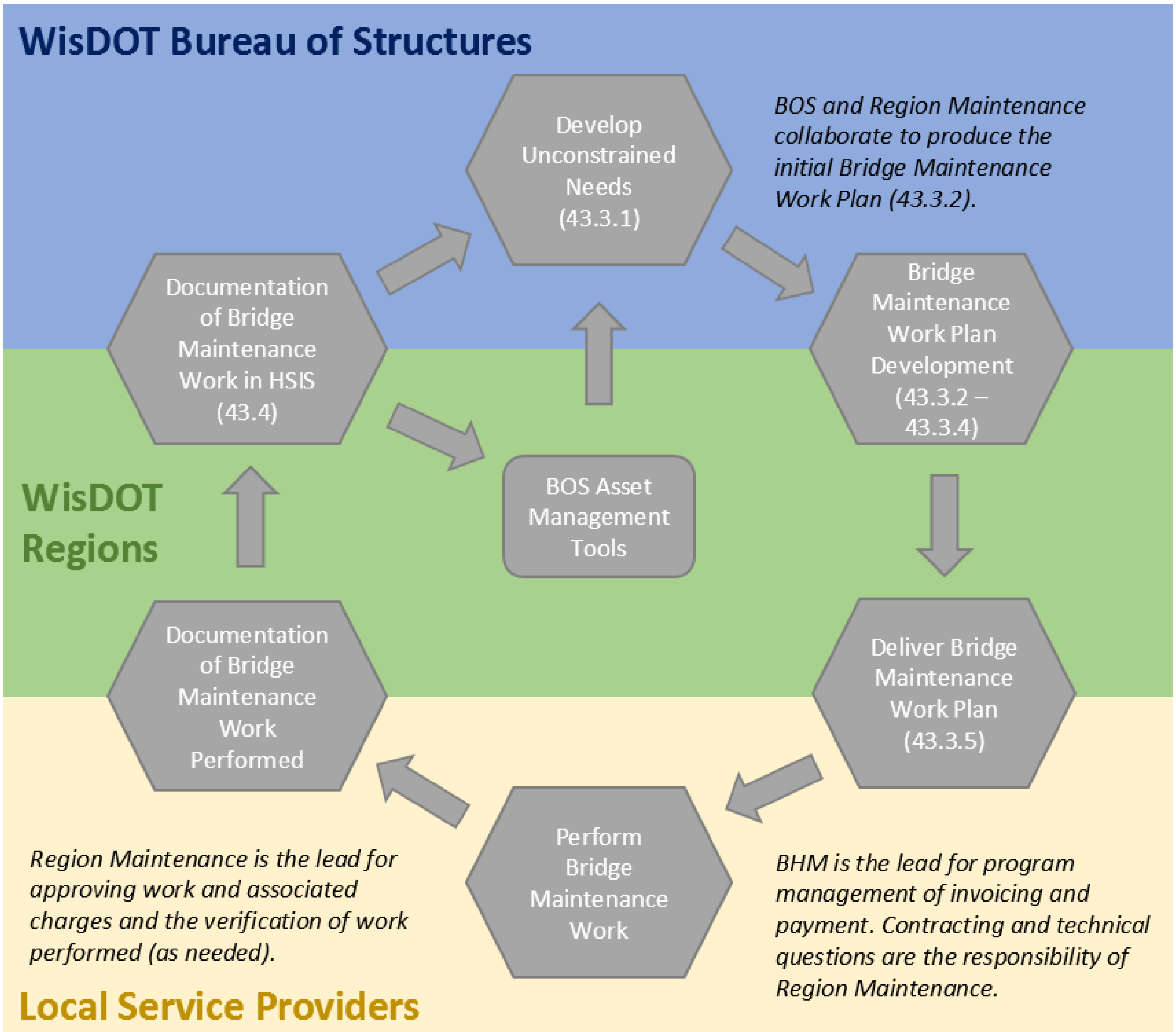

Figure 43.4-1
Bridge Maintenance Workflow and Responsibilities

43.5 Bridge Maintenance Work Reporting

Analyzing data collected from the Final Bridge Maintenance Work Plan is critical to understanding the cost-benefit of performing bridge maintenance activities. BOS will determine the appropriate program reports and share with affected stakeholders, including Region Bridge Maintenance.

It must be noted that there are no goals or target levels associated with these reports at this time. This is currently an information-gathering and analysis exercise to determine the impacts of past work to help shape the direction of future work. BOS will analyze and present the data in a manner to best inform WisDOT and DTSD management on the optimal level of funding for bridge maintenance work and how those funds might best be spent.

- Timeline: BOS will produce bridge maintenance work reports by February 1 for the previous calendar year.

43.6 Definitions

Highway Maintenance Program: The funding mechanism or collection of funding mechanisms by which WisDOT contracts with local service providers to perform maintenance work. The Highway Maintenance Program is inclusive of all transportation infrastructure maintenance including bridge maintenance, but also pavement maintenance and more.

Highway Maintenance Work Plan (HMWP): The list of specific work actions to be performed through the Highway Maintenance Program as described above. It includes work actions on bridges, but also pavements and more. See 43.1.1 and Figure 43.1-2.

Bridge Maintenance Work Plan: This plan addresses bridge maintenance work and is appropriate work for local service providers. The Bridge Maintenance Work Plan is a subset of the larger Highway Maintenance Work Plan.

Bridge Maintenance Actions: This term encompasses both Operational and Preservation bridge maintenance actions.

Operational Bridge Maintenance Actions: Actions necessary for the regular operation of a bridge; actions necessary to maintain the bridge in a serviceable condition.

Preservation Bridge Maintenance Actions: Bridge Preservation is defined as actions or strategies that prevent, delay, or reduce deterioration of bridges or bridge elements, restore the function of existing bridges, keep bridges in good or fair condition and extend their service life. Preservation actions may be cyclic or condition-driven.

Highway Structures Information System: Highway Structures Information System (HSIS) is the system developed by WisDOT for managing the inventory and inspection data of all highway structures. The inspection data is collected in accordance with the NBIS and *2019 AASHTO Manual for Bridge Element Inspection.*

Wisconsin Structures Asset Management System (WiSAMS): Automated application to determine optimal work candidates for improving the condition of structures. This application serves as a programming and planning tool for structures improvements, rehabilitations, maintenance, and preservation. This application coupled with the Highways Structures Information System (HSIS) serves as a comprehensive Structures (Bridge) Management system.

43.7 References

1. FDM 3-1 Exhibit 5.2 Agreement for the Use of Federal Funds for Preventive Maintenance of Structures. (May 2016). (https://wisconsindot.gov/rdwy/fdm/fd-03-05-e0502.pdf#fd3-5e5.2)

2. Highway Maintenance Manual (https://wisconsindot.gov/Pages/doing-bus/local-gov/hwy-mnt/mntc-manual/default.aspx)

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45.3.5 Highway Structure Information System (HSIS)

The Highway Structure Information System (HSIS) is an on-line database used to store a wide variety of bridge information. Data stored in HSIS is used to create the National Bridge Inventory (NBI) file that is submitted annually to FHWA. Much of this data can be useful for the load rating engineer when performing a rating. HSIS is also the central source for documents such as plans and maintenance records. Other information, such as design calculations, rating calculations, fabrication drawings, and items mentioned in 45.3.4.4 may also be found in HSIS. For more information on HSIS, see the WisDOT Bureau of Structures web page or contact the Bureau of Structures Bridge Management Unit.

45.3.6 Load Rating Methodologies – Overview

There are two primary methods of load rating bridge structures that are currently utilized by WisDOT. Both methods are detailed in the AASHTO MBE. They are as follows:

- Load and Resistance Factor Rating (LRFR)
- Load Factor Rating (LFR)

Load and Resistance Factor Rating is the most current rating methodology and has been the standard for new bridges in Wisconsin since approximately 2007. LRFR employs the same basic principles as LFR for the load factors, but also utilizes multipliers on the capacity side of the rating equation, called resistance factors, to account for uncertainties in member condition, material properties, etc. This method is covered in 45.3.7, and a detailed description of this method can also be found in **MBE [6A]**.

Load Factor Rating (LFR) has been used since the early 1990s to load rate bridges in Wisconsin. The factor of safety for LFR-based rating comes from assigning multipliers, called load factors, to both dead and live loads. A detailed description of this method can be found in 45.3.8 and also in **MBE [6B]**.

Allowable Stress Rating (ASR) is a third method of load rating structures. ASR was the predominant load rating methodology prior to the implementation of LFR. It is not commonly used for modern load rating, though it is still permitted to be used for select superstructure types (See 45.3.2). The basic philosophy behind this method assigns an appropriate factor of safety to the limiting stress of the material being analyzed. The maximum stress in the member produced by actual loadings is then checked for sufficiency. A more detailed description of this method can be found in 45.3.9 below and also in **MBE [6B]**.

45.3.7 Load and Resistance Factor Rating (LRFR)

The basic rating equation for LRFR, per **MBE [Equation 6A.4.2.1-1]**, is:

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_{P})(P)}{(\gamma_{LL})(LL + IM)}$$

For the Strength Limit States (primary limit state when load rating using LRFR):

$$C = \phi_C \phi_S \phi R_n$$

Where the following lower limit shall apply:

$$\phi_C \phi_S \geq 0.85$$

Where:

RF	=	Rating factor
C	=	Capacity
R_n	=	Nominal member resistance
DC	=	Dead-load effect due to structural components and attachments
DW	=	Dead-load effect due to the wearing surface and utilities
P	=	Permanent loads other than dead loads
LL	=	Live load effects
IM	=	Dynamic load allowance
γ_{DC}	=	LRFR load factor for structural components and attachments
γ_{DW}	=	LRFR load factor for wearing surfaces and utilities
γ_P	=	LRFR load factor for permanent loads other than dead loads = 1.0
γ_{LL}	=	LRFR evaluation live load factor
ϕ_c	=	Condition factor
ϕ_s	=	System factor
ϕ	=	LRFR resistance factor

The LRFR methodology is comprised of three distinct procedures:

- Design Load Rating (first level evaluation) – Used for verification during the design phase, a design load rating is performed on both new and existing structures alike. See 45.3.7.6 for more information.
- Legal Load Rating (second level evaluation) – If required, the legal load rating is used to determine whether or not the bridge in question can safely carry legal-weight traffic; whether or not a load posting is required. See 45.3.7.7 for more information.

- Emergency Vehicle Load Rating – the Legal Load Rating also includes a separate analysis of FAST Act emergency vehicles (EVs), which may exceed weight limits in place for other vehicles but are considered “legal” because they do not require a permit. The emergency vehicle load rating is used to determine whether or not the bridge in question can safely carry emergency vehicles; whether or not an emergency vehicle-specific weight restriction is required.

- Permit Load Rating (third level evaluation) – The permit load rating is used to determine whether or not over-legal weight vehicles may travel across a bridge. See 45.3.7.8 for more information.

The results of each procedure serve specific uses (as noted above) and also guide the need for further evaluations to verify bridge safety or serviceability. A flow chart outlining this approach is shown in Figure 45.3-1. The procedures are structured to be performed in a sequential manner, as needed, starting with the design load rating. Load rating for emergency vehicles is only required when a bridge fails the design load rating (RF < 0.9) at the inventory level. Load rating for AASHTO legal loads is only required when a bridge fails the design load rating (RF < 1.0) at the operating level.

Note that when designing a new structure, it is required that the rating factor be greater than one for the HL-93 vehicle at the inventory level (note also that new designs shall include a dead load allotment for a future wearing surface); therefore, a legal load rating will never be required on a newly designed structure.

Similarly, only bridges that pass the legal load rating at the operating level (RF ≥ 1.0) can be evaluated utilizing the permit load rating procedures. See 45.11 for more information on over-weight permitting.

45.3.7.1 Limit States

The concept of limit states is discussed in detail in the AASHTO LRFD design code (**LRFD [3.4.1]**). The application of limit states to the design of Wisconsin bridges is discussed in 17.2.3.

Service limit states are utilized to limit stresses, deformations, and crack widths under regular service conditions. Satisfying service limits during the design-phase is critical in order for the structure in question to realize its full intended design-life. WisDOT policy regarding load rating using service limit states is as follows:

<u>Steel Superstructures</u>

- The Service II limit state shall be satisfied (inventory rating > 1.0) during design.

- For design or legal load ratings for in-service bridges, the Service II rating shall be checked at the inventory and operating level.

- The Service II limit state should be considered for permit load rating at the discretion of the load rating engineer.

Reinforced Concrete Superstructures

- WisDOT does not consider the Service I limit state during design.
- For design or legal load ratings of new or in-service bridges, the Service I rating is not required.
- The Service I limit state should be considered for permit load rating at the discretion of the load rating engineer.

Prestressed Concrete Superstructures

- The Service III limit state shall be satisfied (inventory rating > 1.0) during the design phase for a new bridge.
- For rehabilitation design load ratings of an in-service bridge, the Service III limit state should be considered for legal load rating at the discretion of the load rating engineer, but in general, it is not required for prestressed girders that do not show signs of distress. The Service III limit state is not required for a permit load rating.
- For design or legal load ratings of new or in-service bridges, the Service I limit state is not required. The Service I limit state should be considered for permit load rating at the discretion of the load rating engineer.

See Table 45.3-1 for live load factors to use for each limit state. Service limit states checks that are considered optional are shaded.

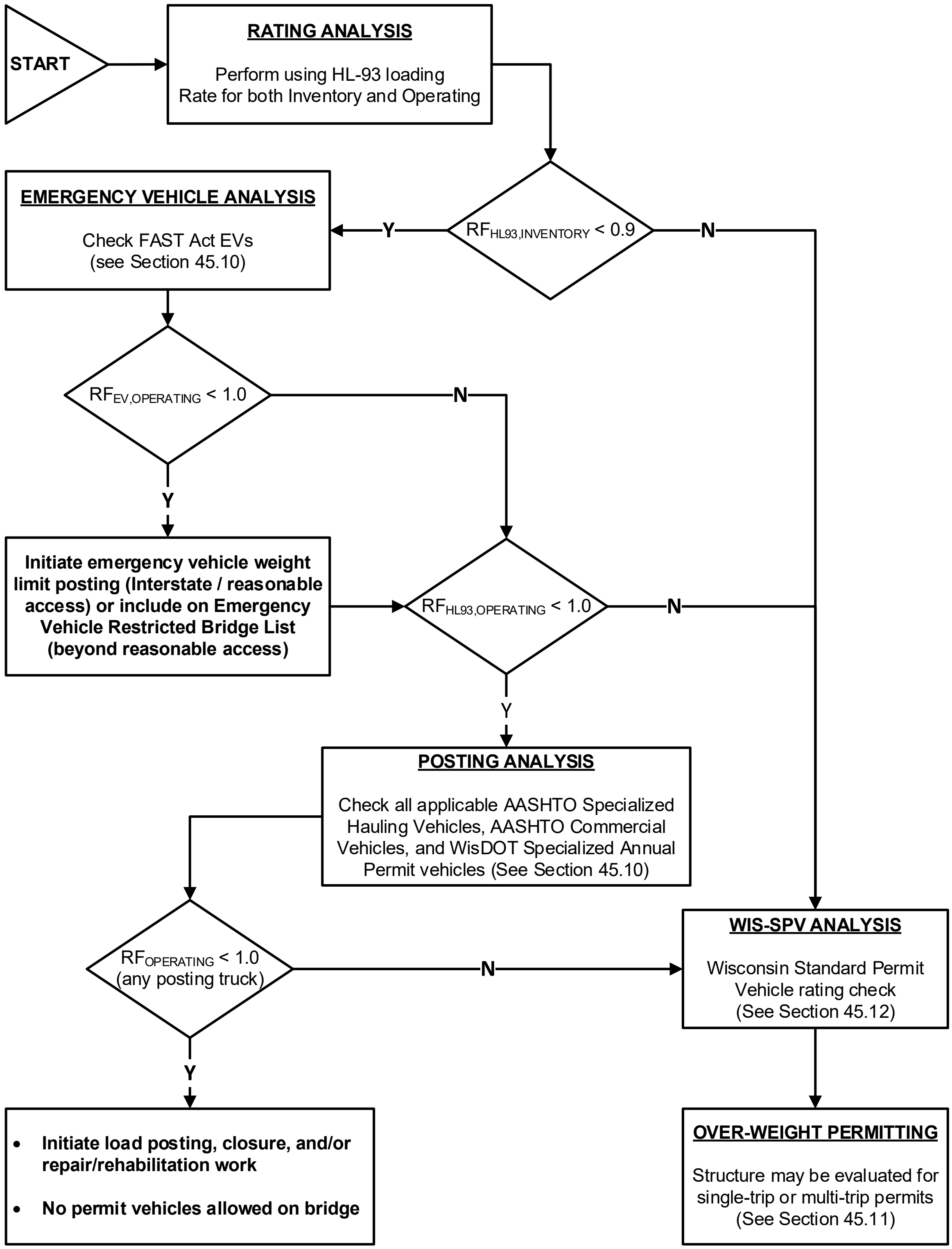


Figure 45.3-1

Load and Resistance Factor Rating Flow Chart

45.3.7.2 Load Factors

The load factors for the Design Load Rating shall be taken as shown in Table 45.3-1. The load factors for the Legal Load Rating shall be taken as shown in Table 45.3-1 and Table 45.3-2.

For emergency vehicles, alternate live load factors determined in accordance with NCHRP Project 20-07 / Task 410 may be used. If alternate live load factors are used, this shall be noted in the Load Rating Summary Form, along with assumptions of one-way ADTT and emergency vehicle crossings per day.

The load factors for the Permit Load Rating shall be taken as shown in Table 45.3-1 and Table 45.3-3 . Again, note that the shaded values in Table 45.3-1 indicate optional checks that are currently not required by WisDOT.

Bridge Type	Limit State	Dead Load DC	Dead Load DW	Design Load		Legal Load	Permit Load
				Inventory	Operating		
				LL	LL	LL	LL
Steel	Strength I, II	1.25	1.50	1.75	1.35	Table 45.3-2	Table 45.3-3
	Service II	1.00	1.00	1.30	1.00	1.30	1.00
Reinforced Concrete	Strength I, II	1.25	1.50	1.75	1.35	Table 45.3-2	Table 45.3-3
	Service I	1.00	1.00	--	--	--	1.00
Prestressed Concrete	Strength I, II	1.25	1.50	1.75	1.35	Table 45.3-2	Table 45.3-3
	Service III	1.00	1.00	0.80	--	1.00	--
	Service I	1.00	1.00	--	--	--	1.00
Timber	Strength I, II	1.25	1.50	1.75	1.35	Table 45.3-2	Table 45.3-3

Table 45.3-1
Limit States and Live Load Factors (γ_{LL}) for LRFR

Loading Type	Live Load Factor
AASHTO Legal Vehicles, State Specific Vehicles, and Lane Type Legal Load Models	1.45
Specialized Haul Vehicles (SU4, SU5, SU6, SU7)	1.45
FAST Act Emergency Vehicles (EV2, EV3) **Alternate load factors per NCHRP Project 20-07/Task 410 are allowed.*	1.30*

Table 45.3-2
Live Load Factors (γ_{LL}) for Legal Loads in LRFR

Permit Type	Loading Condition	Distribution Factor	Live Load Factor
Annual	Mixed with Normal Traffic	Two or more lanes	1.30
Single Trip	Mixed with Normal Traffic	One Lane	1.20
Single Trip	Escorted with no other vehicles on the bridge	One Lane	1.10

Table 45.3-3
Live Load Factors (γ_{LL}) for Permit Loads in LRFR

45.3.7.3 Resistance Factors

The resistance factor, ϕ, is used to reduce the computed nominal resistance of a structural element. This factor accounts for variability of material properties, structural dimensions and workmanship, and uncertainty in prediction of resistance. Resistance factors for concrete and steel structures are presented in Section 17.2.6, and resistance factors for timber structures are presented in **MBE [6A.7.3]**.

45.3.7.4 Condition Factor: ϕ_C

The condition factor provides a reduction to account for the increased uncertainty in the resistance of deteriorated members and the likely increased future deterioration of these members during the period between inspection cycles.

WisDOT policy items:

Current WisDOT policy is to set the condition factor equal to 1.0.

45.3.7.5 System Factor: ϕ_S

System factors are multipliers applied to the nominal resistance to reflect the level of redundancy of the complete superstructure system. Bridges that are less redundant will have their factor member capacities reduced, and, accordingly, will have lower ratings. The aim of the system factor is to provide reserve capacity for safety of the traveling public. See Table 45.3-4 for WisDOT system factors.

Superstructure Type	ϕ_S
Welded Members in Two-Girder/Truss/Arch Bridges	0.85
Riveted Members in Two-Girder/Truss/Arch Bridges	0.90
Multiple Eyebar Members in Truss Bridges	0.90
Three-Girder Bridges with Girder Spacing ≤ 6.0 ft	0.85
Four-Girder Bridges with Girder Spacing ≤ 4.0 ft	0.95
All Other Girder and Slab Bridges	1.00
Floorbeam Spacings > 12.0 ft and Non-Continuous Stringers	0.85
Redundant Stringer Subsystems Between Floorbeams	1.00

Table 45.3-4
System Factors for WisDOT

45.3.7.6 Design Load Rating

The design load rating assesses the performance of bridges utilizing the LRFD design loading, producing an inventory and operating rating. Note that when designing a new structure, it is required that the RF > 1.0 at the inventory level. In addition to providing a relative measure of bridge capacity, the design load rating also serves as a screening process to identify bridges that should be load rated for legal loads. If a structure has an inventory RF < 0.9, it may not be able to safely carry emergency vehicles, and if it has an operating RF < 1.0, it may not be able to safely carry other legal-weight traffic and therefore a legal load rating must be performed. If a structure has rating factors above these thesholds, , proceeding to the legal load rating is not required. However, the load rating engineer is still required to rate the Wisconsin Standard Permit Vehicle (Wis-SPV) as shown in 45.12.

45.3.7.6.1 Design Load Rating Live Load

The LRFD design live load, HL-93, shall be utilized as the rating vehicle(s). The components of the HL-93 loading are described in 17.2.4.2.

45.3.7.7 Legal Load Rating

Bridges that do not satisfy the HL-93 design load rating check (RF < 1.0 at operating level) shall be evaluated for legal loads to determine if legal-weight traffic should be restricted; whether a load posting is required. Additionally, bridges that do not satisfy the HL-93 design load rating check (RF < 0.9 at inventory level) shall be evaluated for FAST Act emergency vehicle loads to determine if emergency vehicle-specific weight limits are required. If the load rating engineer determines that a load posting is required, please notify the Bureau of Structures Rating Unit. For more information on the load posting of bridges, see 45.10.

45.3.7.7.1 Legal Load Rating Live Load

The live loads used for legal load rating calculations are a combination of AASHTO-prescribed vehicles, Wisconsin-specific vehicles, and FAST Act emergency vehicles. The vehicles to be used for the legal load rating are described in 45.10.

45.3.7.8 Permit Load Rating

Permit load rating is the level of load rating analysis required for all structures when performing the Wisconsin Standard Permit Vehicle (Wis-SPV) design check as illustrated in 45.12. The results of the Wis-SPV analysis are used in the regulation of multi-trip permits. The actual permitting process for State-owned bridges is internal to the WisDOT Bureau of Structures.

Permit load rating is also used for issuance of single trip permits. For each single trip permit, the actual truck configuration is analyzed for every structure it will cross. The single trip permitting process for State-owned bridges is internal to WisDOT Bureau of Structures.

For more information on over-weight truck permitting, see 45.11.

45.3.7.8.1 Permit Load Rating Live Load

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed (Figure 45.3-1). Specifics on this analysis can be found in 45.12.

For specific single trip permit applications, the actual truck configuration described in the permit shall be the live load used to analyze all pertinent structures. Permit analysis for State-owned bridges is internal to the WisDOT Bureau of Structures.

WisDOT policy items:

WisDOT interpretation of **MBE [6A.4.5.4.1]** is for spans up to 200'-0", only the permit vehicle shall be considered present in a given lane. For spans 200'-0" in length or greater an additional lane load shall be applied to simulate closely following vehicles. The lane load shall be taken as 0.2 klf in each lane and shall only be applied to those portions of the span(s) where the loading effects add to the permit load effects.

Also note, as stated in the footnote of **MBE [Table 6A.4.5.4.2a-1]**, when using a single-lane LRFD distribution factor, the 1.2 multiple presence factor should be divided out from the distribution factor equations.

45.3.7.9 Load Distribution for Load and Resistance Factor Rating

In general, live load distribution factors should be calculated based on the guidance of the current AASHTO LRFR Standard Design specifications. For WisDOT-specific guidance on the placement and distribution of live loads, see 17.2.7 or 18.4.5.1 for concrete slab superstructures and 17.2.8 for concrete deck on girder superstructures.

See also 45.5.1.2 for specific direction on the placement of live loads for rating and posting.

Dead loads shall be distributed as described in 17.2.7 for concrete slab superstructures and 17.2.8 for concrete deck on girder superstructures.

45.3.8 Load Factor Rating (LFR)

The basic rating equation for Load Factor Rating can be found in **MBE [Equation 6B.4.1-1]** and is:

$$RF = \frac{C - A_1 D}{A_2 L(1 + I)}$$

Where:

RF	=	Rating factor for the live load carrying capacity
C	=	Capacity of the member
D	=	Dead load effect on the member
L	=	Live load effect on the member
I	=	Impact factor to be used with the live load effect
A_1	=	Factor for dead load
A_2	=	Factor for live load

Unlike LRFR, load factor rating does not have three prescribed levels of rating analysis. However, in practice, the process is similar for both LRFR and LFR.

The first step is to perform a rating analysis to determine inventory and operating ratings. Based on the results of the rating analysis, a posting analysis should be performed when:

- The inventory rating factor is less than or equal to 1.0 (HS-20) – Emergency Vehicles (EVs) only, see Figure 45.10-5; or

- The operating rating factor is less than or equal to 1.2 (HS-24) – Specialized Hauling Vehicles (SHVs) only, see Figure 45.10-2; or

- The operating rating factor is less than or equal to 1.0 (HS-20) for all other posting vehicles.

An emergency vehicle analysis is performed to determine whether a bridge can safely carry emergency vehicles, which may exceed legal weight limits in place for other vehicles. A posting analysis is performed to determine whether a bridge can safely carry other legal-weight traffic. Both analyses are performed at the operating level. See 45.10 for more information.

Permit analysis is used to determine whether or not over legal-weight vehicles may travel across a bridge. See 45.11 for more information on over-weight vehicle permitting.

A flow chart outlining this approach is shown in Figure 45.3-2. The procedures are structured to be performed in a sequential manner, as needed.

45.3.8.1 Load Factors for Load Factor Rating

See Table 45.3-5 for load factors to be used when rating with the LFR method. The nominal capacity, C, is the same regardless of the rating level desired.

For emergency vehicles, alternate live load factors determined in accordance with NCHRP Project 20-07 / Task 410 may be used. If alternate live load factors are used, this shall be noted in the Load Rating Summary Form, along with assumptions of one-way ADTT and emergency vehicle crossings per day.

LFR Live Load Factors		
Rating Level	A_1	A_2
Inventory	1.3	2.17
Operating	1.3	1.3

Table 45.3-5
LFR Live Load Factors

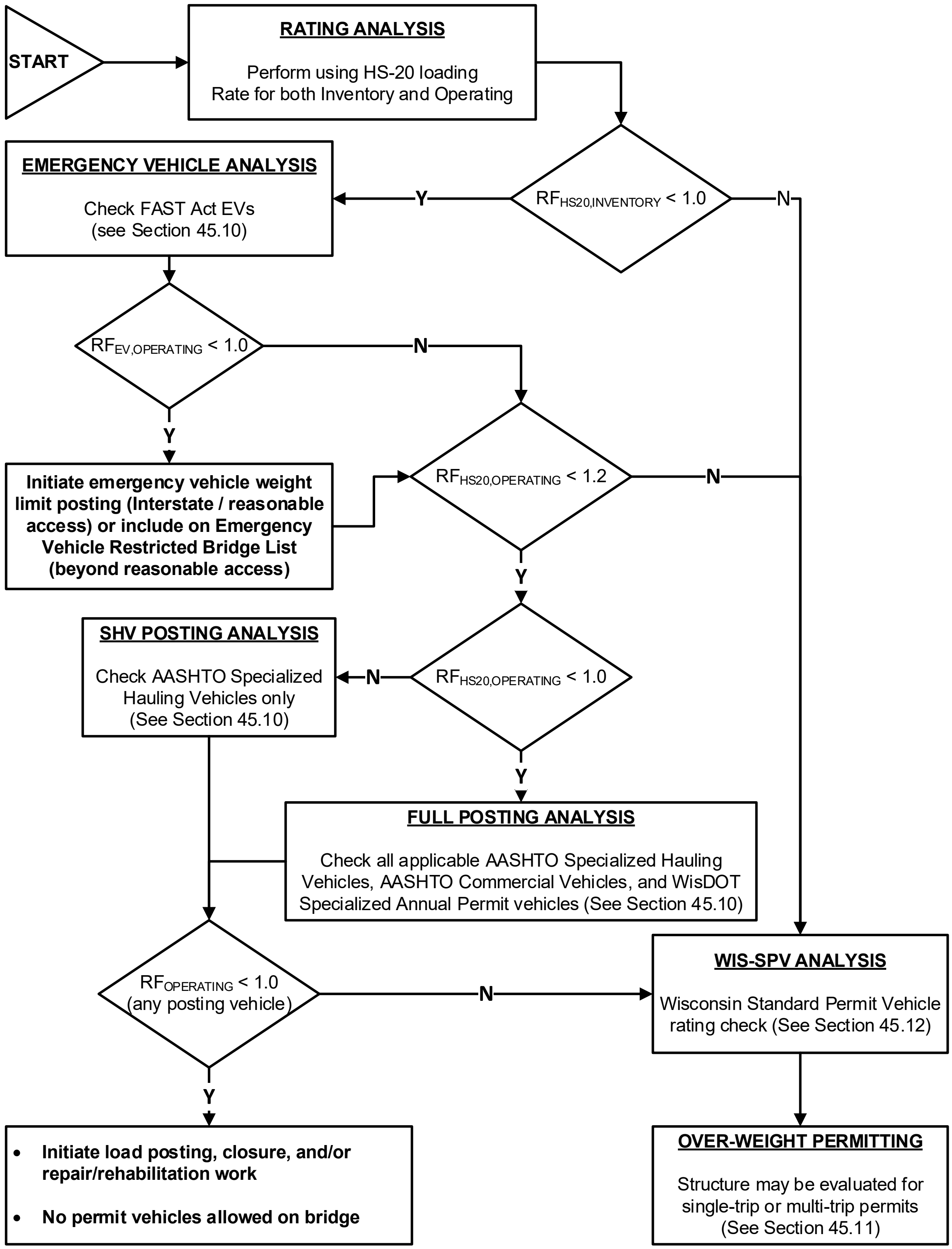


Figure 45.3-2

Load Factor Rating and Allowable Stress Rating Flow Chart

45.3.8.2 Live Loads for Load Factor Rating

Similar to LRFR, there are three potential checks to be made in LFR that are detailed in the flow chart shown in Figure 45.3-2.

- For purposes of calculating the inventory and operating rating of the structure, the live load to be used should be the HS20 truck or lane loading as shown in Figures 17.2-1 and 17.2-3.
- The live load(s) to be used for analysis are a combination of AASHTO-prescribed vehicles, Wisconsin-specific vehicles, and FAST Act emergency vehicles. For more information on load posting analysis, refer to 45.10.2.
- For conducting the Wisconsin Standard Permit Vehicle analysis, use the loading shown in Figure 45.12-1.

45.3.8.3 Load Distribution for Load Factor Rating

In general, distribution factors should be calculated based on the guidance of the *AASHTO Standard Design Specifications, 17th Edition*.

See 45.5.1.2 for specific direction on the placement of live loads for rating and posting.

Dead loads shall be distributed as described in 17.2.7 for concrete slab superstructures and 17.2.8 for concrete deck on girder superstructures.

45.3.9 Allowable Stress Rating (ASR)

The basic rating equation can be found in **MBE [Equation 6B.4.1-1]** and is:

$$RF = \frac{C - D}{L(1 + I)}$$

Where:

RF	=	Rating factor for the live load carrying capacity
C	=	Capacity of the member
D	=	Dead load effect on the member
L	=	Live load effect on the member
I	=	Impact factor to be used with the live load effect

Unlike LRFR, allowable stress rating does not have three prescribed levels of rating analysis. However, in practice, the process is similar for both LRFR and ASR.

The first step is to perform a rating analysis to determine inventory and operating ratings. Based on the results of the rating analysis, a posting analysis should be performed when:

- The inventory rating factor is less than or equal to 1.0 (HS-20) – Emergency Vehicles (EVs) only, see Figure 45.10-5; or

- The operating rating factor is less than or equal to 1.2 (HS-24) – Specialized Hauling Vehicles (SHVs) only, see Figure 45.10-2; or

- The operating rating factor is less than or equal to 1.0 (HS-20) for all other posting vehicles.

An emergency vehicle analysis is performed to determine whether a bridge can safely carry emergency vehicles, which may exceed legal weight limits in place for other vehicles. A posting analysis is performed to determine whether a bridge can safely carry other legal-weight traffic. Both analyses are performed at the operating level. See 45.10 for more information.

Permit analysis is used to determine whether or not over legal-weight vehicles may travel across a bridge. See 45.11 for more information on over-weight vehicle permitting.

A flow chart outlining this approach is shown in Figure 45.3-2. The procedures are structured to be performed in a sequential manner, as needed.

45.3.9.1 Stress Limits for Allowable Stress Rating

The inventory and operating stress limits used in ASR vary by material. See **MBE [6B]** for more information.

45.3.9.2 Live Loads for Allowable Stress Rating

Similar to LRFR and LFR, there are three potential checks to be made in ASR.

- For purposes of calculating the inventory and operating rating of the structure, the live load to be used should be the HS-20 truck or lane loading as shown in Figures 17.2-1 and 17.2-3.

- The live load(s) to be used for analysis are a combination of AASHTO-prescribed vehicles, Wisconsin-specific vehicles, and FAST Act emergency vehicles. For more information on load posting analysis, refer to 45.10.2.

- For conducting the Wisconsin Standard Permit Vehicle analysis, use the loading shown in Figure 45.12-1.

45.3.9.3 Load Distribution for Allowable Stress Rating

In general, distribution factors should be calculated based on the guidance of the *AASHTO Standard Design Specifications, 17th Edition*.

See 45.5.1.2 for specific direction on the placement of live loads for rating and posting.

Dead loads shall be distributed as described in 17.2.7 for concrete slab superstructures and 17.2.8 for concrete deck on girder superstructures.

45.3.10 Engineering Judgment, Condition-Based Ratings, and Load Testing

Engineering judgment or condition-based ratings alone shall not be used to determine the capacity of a bridge when sufficient structural information is available to perform a calculation-based method of analysis.

Ratings determined by the method of field evaluation and documented engineering judgment may be considered when the capacity cannot be calculated due to one or more of the following reasons:

- No bridge plans available
- Concrete bridges with unknown reinforcement

The engineer shall consider all available information, including:

- Condition of load carrying elements (inspection reports – current and historic)
- Year of construction
- Material properties of members (known or assumed per 45.5.2)
- Type of construction
- Redundancy of load path
- Field measurements
- Comparable structures with known construction details
- Changes since original construction
- Loading (past, present, and future)
- Other information that may contribute to making a more-informed decision

If the engineer of record is considering using a judgment- or inspection-based load rating, a thorough visual observation of the bridge should be conducted, including observing actual traffic patterns for the in-service bridge.

The criteria applied to determine a rating by field evaluation and documented engineering judgment shall be documented via the Load Rating Summary Form (see 45.9) accompanied by any and all related inspection reports, any calculation performed to assist in the rating and assumptions used for those calculations, a written description of the observed traffic patterns for the bridge, relevant correspondences, and any available, relevant photographs of the bridge or bridge condition.

Bridge owners may also consider nondestructive proof load tests in order to establish a safe capacity for bridges in which a load rating cannot be calculated.

WisDOT policy items:

Consult the Bureau of Structures Rating Unit before moving forward with an engineering judgment-based, inspection-based load rating, or with a load testing procedure on either the State or Local system.

45.3.11 Refined Analysis

Methods of refined analysis are discussed in **LRFD [4.6.3]**. These include the use of 2D and 3D finite element modeling of bridge structures, which preclude the use of live load distribution factor equations and instead rely on the relative stiffness of elements in the analytical model for distribution of applied loads. As such, a 2D or 3D model requires the inclusion of elements contributing to the transverse distribution of loads, such as deck and cross frame elements that are otherwise not directly considered in a line girder or strip width analysis. Additional guidance on refined analysis can be found in the AASHTO/NSBA publication “G13.1 Guidelines for Steel Girder Bridge Analysis, 2nd Edition” and the FHWA “Manual of Refined Analysis” (anticipated 2017).

WisDOT policy items:

Prior to using refined analysis, consult the Bureau of Structures Rating Unit. Additional documentation is required when performing a refined analysis; see 45.9 for these requirements.

The Bureau of Structures does not require a specific piece of software be used by consultant engineers when performing a refined load rating analysis. See 45.4 for information on load rating computer software.

Refined analysis for load rating purposes is required for certain structure types, and/or structures with certain geometric characteristics. In other instances a refined analysis may be utilized to improve the structure rating for the purpose of avoiding load posting or to improve the capacity for permitting.

A refined analysis is required for the following structure types:

- Steel rigid frames
- Bascule-type movable bridges
- Tied arches
- Cable stayed or suspension bridges

- Steel box (tub) girder bridges

A refined analysis is require if any of the following geometric characteristics are present within the structural system to be load rated:

- Steel girder structure curved in plan, not meeting the criteria discussed in 45.6.3.2.1.
- Steel girder structure skewed 40 degrees or more, with cross framing type discussed in 45.6.3.2.2.
- Skew varies between adjacent supports by more than 20 degrees.

A refined analysis *may* be required if any of the following geometric characteristics are present within the structural system to be load rated. Contact the Bureau of Structures Rating Unit prior to determine the level of effort to rate the structure.

- Steel girder structures with flared girder spacing, such that the change in girder spacing over the span length is greater or equal to 0.015 ($\Delta S/L \geq 0.015$).
- Structures with complex framing plans; those having discontinuous girders utilizing transfer girders in-span.
- Superstructure supported by flexible supports (e.g. straddle bent with integral pier cap). *Note: These "flexible" supports are considered primary members and are to be included in a load rating.*

45.4 Load Rating Computer Software

Though not required, computer software is a common tool used for load rating. WisDOT BOS encourages the use of software for its benefits in increased efficiency and accuracy. However, the load rating engineer must be aware that software is a tool; the engineer maintains responsibility for understanding and verifying any load rating obtained from computer software and should have a full understanding of all underlying assumptions. The load rating engineer is responsible for ensuring that any software used to develop a rating performs that rating in accordance with relevant AASHTO specifications and taking into account specific WisDOT policy noted in this chapter.

45.4.1 Rating Software Utilized by WisDOT

The Bureau of Structures currently uses a mix of software developed in-house and software available commercially. For a list of software currently used by WisDOT for each primary structure type, see the Bureau of Structures website:

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

WisDOT does not currently mandate the use of any particular software for load ratings.

45.4.2 Computer Software File Submittal Requirements

When load rating software is used as a tool to derive the load rating for a bridge project (new or rehabilitation), the electronic input file shall be included with the project submittal.

Some superstructure types may require advanced modeling techniques in order to fully and accurately capture the structural response. See 45.3.11 for more information on refined analysis.

See 45.9 (Documentation and Submittals) for more information.

WisDOT policy items:

For in-service concrete pipe culverts in poor condition, contact the Bureau of Structures Rating Unit for direction on what is required for a load rating.

45.8.2.3 In-Service Steel Pipe Culverts

An in-service steel pipe culvert in fair or better condition does not require a calculated load rating. Ratings shall be reported as:

- Inventory rating factor: 1.0
- Operating rating factor: 1.67
- Wisconsin Standard Permit Vehicle (Wis-SPV): 190 kips

WisDOT policy items:

For in-service steel pipe culverts in poor condition, contact the Bureau of Structures Rating Unit for direction on what is required for a load rating.

45.9 Load Rating Documentation and Submittals

The Bridge Rating and Management Unit is responsible for maintaining information for every structure in the Wisconsin inventory, including load ratings. This information is used throughout the life of the structure to help inform decisions on potential load postings, repairs, rehabilitation, and eventual structure replacement. That being the case, it is critical that WisDOT collect and store complete and accurate documentation regarding load ratings.

45.9.1 Load Rating Calculations

The rating engineer is required to submit load rating calculations. Calculations should be comprehensive and presented in a logical, organized manner. The submitted calculations should include a summary of all assumptions used (if any) to derive the load rating.

45.9.2 Load Rating Summary Forms

After the structure has been load rated, the WisDOT Bridge Load Rating Summary Form shall be completed and utilized as a cover sheet for the load rating calculations (see Figure 45.9-1). This form may be obtained from the Bureau of Structures or is available on the following website:

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

If required, the Refined Analysis Rating Form (see 45.9.5 and Figure 45.9-2) is available at the same location.

Instructions for completing the forms are as follows:

Load Rating Summary Form

1. Fill out applicable Bridge Data, Structure Type, and Construction History information using HSIS as reference.

2. Check what rating method and rating vehicle was used to rate the bridge in the spaces provided.

3. Enter the inventory/operating ratings, controlling element, controlling force effect, and live load distribution factor for the rating vehicle.

    a. If the load distribution was determined through refined methods (i.e., finite element analysis), it is not necessary to record the live load distribution factor. Instead, enter “REFINED” in the space provided and use the “Remarks/Recommendations” section to describe the methods used to determine live load distribution.

4. The rating for the Wisconsin Special Permit vehicle (Wis-SPV) is always required and shall be given on the rating sheet for both a multi-lane distribution and a single-lane distribution. Make sure not to include the future wearing surface in these calculations.

All reported ratings are based on current conditions and do not reflect future wearing surfaces. Enter the Maximum Vehicle Weight (MVW) for the Wis-SPV analysis, controlling element, controlling force effect, and live load distribution factor.

5. When necessary, AASHTO legal and WisDOT Specialized annual Permit vehicles shall be analyzed and load postings determined per the requirements of 45.10.

   a. Enter the lowest operating rating in kips for each appropriate vehicle type, along with corresponding controlling element and force effect, as well as live load distribution factor.

   b. If a posting vehicle analysis was performed, check the box indicating if a load posting is required or not required. The weight limit in tons is automatically calculated when posting vehicle rating factors are below 1.0. If analysis shows that a load posting is required, specify the level of posting and contact the Bureau of Structures Rating Unit immediately.

6. When necessary, emergency vehicles shall be analyzed and weight limit restrictions determined per the requirements of 45.10.

   a. Enter the lowest operating rating factor for each emergency vehicle, along with corresponding controlling element and force effect, as well as live load distribution factor.

   b. Check the box indicating if an emergency vehicle weight limit is required or not required. The single axle, tandem axle, and gross vehicle weight limits are automatically calculated when emergency vehicle rating factors are below 1.0. If analysis shows that an emergency vehicle weight limit is required, specify the level of the limit and contact the Bureau of Structures Rating Unit immediately.

7. Enter all additional remarks as required to clarify the load capacity calculations.

8. It is necessary for the responsible engineer to sign and seal the form in the space provided. This is true even for rehabilitation projects with no change to the ratings.

45.9.3 Load Rating on Plans

The plans shall contain the following rating information:

- Inventory Load Rating – The plans shall have either the HS value of the inventory rating if using LFR or the rating factor for the HL-93 if using LRFR. For LFR ratings, the rating should be rounded down to the nearest whole number. <u>This rating shall be based on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface.</u> See 6.2.2.3.4 for more information on reporting ratings on plans.

- Operating Load Rating – The plans shall have either the HS value of the operating rating if using LFR or the rating factor for the HL-93 if using LRFR. For LFR ratings, the rating should be rounded down to the nearest whole number. <u>This rating shall be based</u>

on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface. See 6.2.2.3.4 for more information.

- Wisconsin Special Permit Vehicle – The plans shall also contain the results of the Wis-SPV analysis utilizing single-lane distribution and assuming that the vehicle is mixing with normal traffic and that the full dynamic load allowance is utilized. This rating shall be based on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface. The recorded rating for this is the total allowable vehicle weight rounded down to the nearest 10 kips. If the value exceeds 250 kips, limit the plan value to 250 kips. See 6.2.2.3.4 for more information.

45.9.4 Computer Software File Submittals

If analysis software is used to determine the load rating, the software input file shall be provided as a part of the submittal. The name of the analysis software and version should be noted on the Load Rating Summary form in the location provided.

45.9.5 Submittals for Bridges Rated Using Refined Analysis

Additional pages of documentation are required when performing a refined analysis. In addition to the Load Rating Summary Form, also submit the Refined Analysis Rating Form as shown in Figure 45.9-2.

45.9.6 Other Documentation Topics

Structures with Two Different Rating Methods

There may be situations where a given superstructure contains elements that were constructed at different times. In these situations, two different rating methods are used during the design/rating process. For example, a girder replacement or widening. In this case, the new girder(s) would be designed/rated using LRFR, while the existing girders would be rated using LFR. A Load Rating Summary Form shall be submitted for both new & existing structure analysis methods; controlling LRFR rating of the new superstructure components, and controlling LFR rating of the existing superstructure. Both sets of controlling rating values (new & existing) shall be noted on the plan set, as noted in 6.2.2.3.4.

Wisconsin Department of Transportation

Bridge Load Rating Summary

v. 07-2020

Bridge Data

Bridge Number:		Traffic Count:		Truck Traffic %:	
Owner:		Overburden Depth (in):			
Municipality:		Inspection Date:			
Feature On:		NBI Condition Ratings:			
Feature Under:		Deck	Superstructure	Substructure	Culvert
Design Loading:					

Structure Type

Span #	Material	Configuration	Length (ft)
1			

Construction History

Year	Work Performed

Load Rating Summary

			Controlling Element	Controlling Force Effect	LL Distribution Factor
Rating Method:		Ratings			
		Inventory			
Rating Vehicle:		Operating			
Wisconsin SPV		MVW (k)	Controlling Element	Controlling Force Effect	LL Distribution Factor
Single Lane (w/o FWS)					
Multi Lane (w/o FWS)					

Load Posting Analysis (when required per Wisconsin Bridge Manual, Chapter 45)

Posting Vehicle		Vehicle GVW (k)	Rating Factor	Weight Limit (T)	Controlling Element	Controlling Force Effect	LL Distribution Factor
AASHTO Legal Vehicles	Type 3	50		N/A			
	Type 3S2	72		N/A			
	Type 3-3	80		N/A			
	SU4	54		N/A			
	SU5	62		N/A			
	SU6	69.5		N/A			
	SU7	77.5		N/A			
WisDOT Spec.	PUP	98		N/A			
	Semi	98		N/A			
FAST Act EVs	EV2	57.5		N/A			
	EV3	86		N/A			

Posting for Legal/Specialized Permit Vehicles:

X Not Required

Required T

Weight Limits for Emergency Vehicles:

X Not Required

Required T Single Axle// T Tandem // T Gross

Computer Software and Version Used:		**Load Rating Engineer**
Additional Remarks:		Name:
		Date:

Figure 45.9-1

Bridge Load Rating Summary Form

In Addition to this form, submit electronic analysis files (eg. .MDX, .bdb)

ANALYSIS FILE SUMMARY (FILL OUT FOR EACH ANALYSIS FILE SUBMITTED)

Analysis Type:	☐ Grid/Grillage ☐ Plate & Ecc. Beam ☐ 3D FEM ☐ Other *(describe below)*
Analysis Program:	☐ MDX ☐ AASHTOWare ☐ CSI Bridge ☐ LARSA ☐ Other
Program Version:	
File Name:	
File Description:	Describe the purpose of the file. Example: This file is used for the Wis-SPV rating using single lane distribution.
Analysis Assumptions:	Highlight key assumptions in modeling. (This section may be omitted if submitting MDX or AASHTOWare analysis files. This section may also be omitted if submitting separate document containing analysis assumptions and results). Example of things to include: a description of the finite element model, simplifications made to model, exceptions to original design plans, loads applied, how loads are applied (e.g. equally distributed to all girders), support conditions, composite/non-composite sections.
Summary of Results:	Summarize results. (This section may be omitted if submitting MDX or AASHTOWare analysis files. This section may also be omitted if submitting separate document containing analysis assumptions and results). Provide table of results for service load reactions, moment, shear, and/or stress output for members at 10th points (minimum) for the appropriate load cases. Provide a table of capacities at each 10th point, such that load ratings can be directly computed with appropriate load and/or resistance and impact factors. Provide example or typical calculations.

Figure 45.9-2

Refined Analysis Rating Form

45.10 Load Postings

45.10.1 Overview

Legal-weight for vehicles travelling over bridges is determined by state-specific statutes, which are based in part on the Federal Bridge Formula. The Federal Bridge Formula is discussed in 45.2.5. When a bridge does not have the capacity to carry legal-weight traffic, more stringent load limits are placed on the bridge – a load posting. Currently in Wisconsin, load postings are based on gross vehicle weight; there is no additional consideration for number of axles or axle spacing. Load posting signage is discussed further in 45.10.4.

A separate analysis is conducted for emergency vehicles (EVs). As a result of the 2015 Fixing America's Surface Transportation Act (FAST Act), FHWA requires bridges to be load rated for emergency vehicles where they are exempt from regular weight limits, and restricted if necessary. When a bridge does not have the capacity to carry the FAST Act EVs, emergency vehicle-specific load postings are required for bridges on the Interstate and within reasonable access to the Interstate. Because Wisconsin statutes also exempt emergency vehicles from state laws governing weight provisions, bridges located beyond reasonable access with insufficient capacity will be placed on the Emergency Vehicles Restricted Bridge List (under development). Weight limit restrictions for emergency vehicles are based on a combination of the single axle, tandem axle, and gross vehicle weight limits, discussed further in 45.10.3. Additional information on FAST Act EV load rating requirements may be found in FHWA's memorandum, "Action: Load Rating for the FAST Act's Emergency Vehicles" (November 2016) and the technical guidance, "Questions and Answers: Load Rating for the FAST Act's Emergency Vehicles, Revision R01" (March 2018).

In order to remain open to traffic, a bridge should be capable of carrying a minimum gross live load weight of three tons at the Operating level. Bridges not capable of carrying a minimum gross live load weight of three tons at the Operating level must be closed. As stated in the **MBE [6A.8.1]** and **[6B.7.1]**, when deciding whether to close or post a bridge, the Owner should consider the character of traffic, the volume of traffic, the likelihood of overweight vehicles, and the enforceability of weight posting.

The owner of a bridge has the responsibility and authority to load post a bridge as required. The State Bridge Maintenance Engineer has the authority to post a bridge and must issue the approval to post any State bridge.

WisDOT policy items:

Consult the Bureau of Structures Rating Unit as soon as possible with any analysis that results in a load posting or emergency vehicle weight limit for any structure on the State or Local system.

45.10.2 Load Posting Live Loads

The live loads to be used in the rating formula for posting considerations are any of the three typical AASHTO Commercial Vehicles (Type 3, Type 3S2, Type 3-3) shown in Figure 45.10-1, any of the four AASHTO Specialized Hauling Vehicles (SHVs - SU4, SU5, SU6, SU7) shown in Figure 45.10-2, the WisDOT Specialized Annual Permit Vehicles shown in Figure 45.10-3, and the Wisconsin Standard Permit Vehicle shown in Figure 45.12-1.

The AASHTO Commercial Vehicles and Specialized Hauling Vehicles are modeled on actual in-service vehicle configurations. These vehicles comply with the provisions of the Federal Bridge Formula and can thus operate freely without permit; they are legal weight/configuration.

The WisDOT Specialized Annual Permit Vehicles are Wisconsin-specific vehicles. They represent vehicle configurations made legal in Wisconsin through the legislative process and current Wisconsin state statutes.

The Wisconsin Standard Permit Vehicle (Wis-SPV) is a configuration used internally by WisDOT to assist in the regulation of multi-trip (annual) permits. Multi-trip permits and the Wis-SPV are discussed in more detail in 45.11.2 and 45.12.

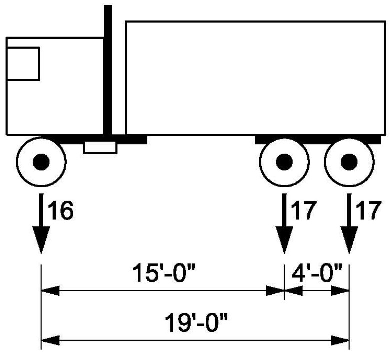
As stated in **MBE [6A.4.4.2.1a]**, for spans up to 200', only the vehicle shall be considered present in the lane for positive moments. It is unnecessary to place more than one vehicle in a lane for spans up to 200' because the load factors provided have been modeled for this possibility. For spans 200' in length or greater, the AASHTO Type 3-3 truck multiplied by 0.75 shall be analyzed combined with a lane load as shown in Figure 45.10-4. The lane load shall be taken as 0.2 klf in each lane and shall only be applied to those portions of the span(s) where the loading effects add to the vehicle load effects.

Also, for negative moments and reactions at interior supports, a lane load of 0.2 klf combined with two AASHTO Type 3-3 trucks multiplied by 0.75 shall be used. The trucks should be heading in the same direction and should be separated by 30 feet as shown in Figure 45.10-4. There are no span length limitations for this negative moment requirement.

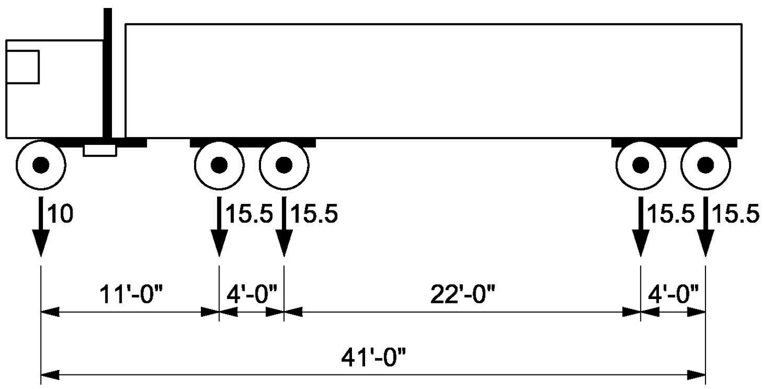
When the lane-type load model (see Figure 45.10-4) governs the load rating, the equivalent truck weight for use in calculating a safe load capacity for the bridge shall be taken as 80 kips as is specified in **MBE [6A.4.4.4]**.

For emergency vehicle weight limits, FHWA has determined that, for the purpose of load rating, two emergency vehicle configurations (EV2 and EV3) produce effects in typical bridges that envelop the effects resulting from the family of typical emergency vehicles covered by the FAST Act. The EV2 and EV3 are shown in Figure 45.10-5.

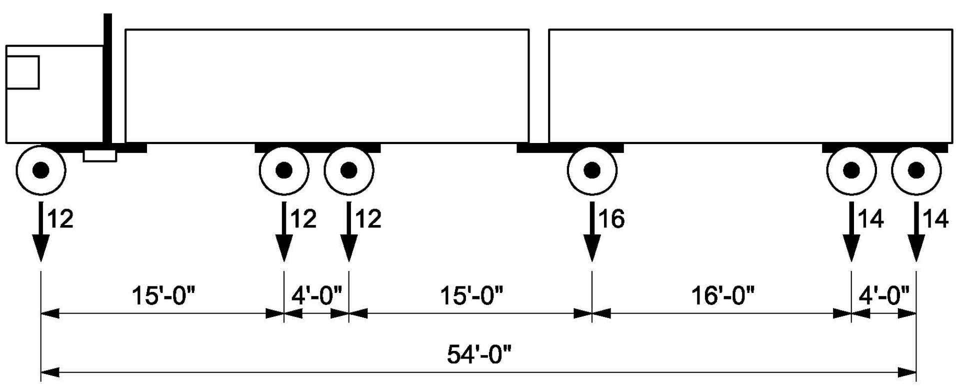
Indicated concentrations are axle loads in kips.



Type 3 Unit Weight = 50 Kips (25 tons)



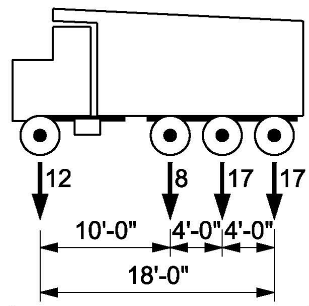
Type 3S2 Unit Weight = 72 Kips (36 tons)



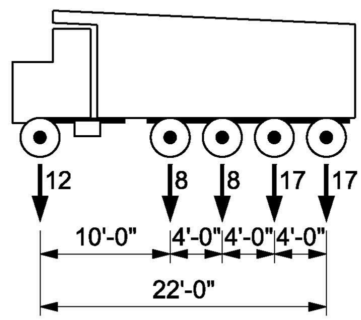
Type 3-3 Unit Weight = 80 Kips (40 tons)

Figure 45.10-1
AASHTO Commercial Vehicles

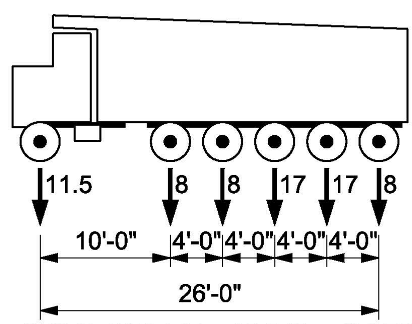
Indicated concentrations are axle loads in kips.



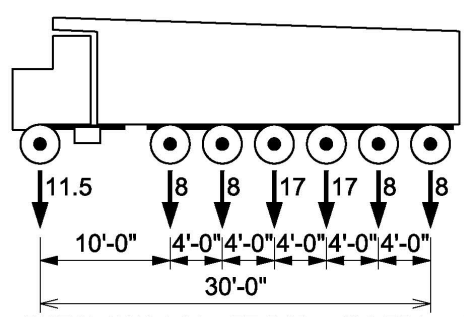
Type SU4 Unit Weight = 54 Kips (27 tons)



Type SU5 Unit Weight = 62 Kips (31 tons)



Type SU6 Unit Weight = 69.5 Kips (34.75 tons)



Type SU7 Unit Weight = 77.5 Kips (38.75 tons)

Figure 45.10-2
AASHTO Specialized Hauling Vehicles (SHVs)

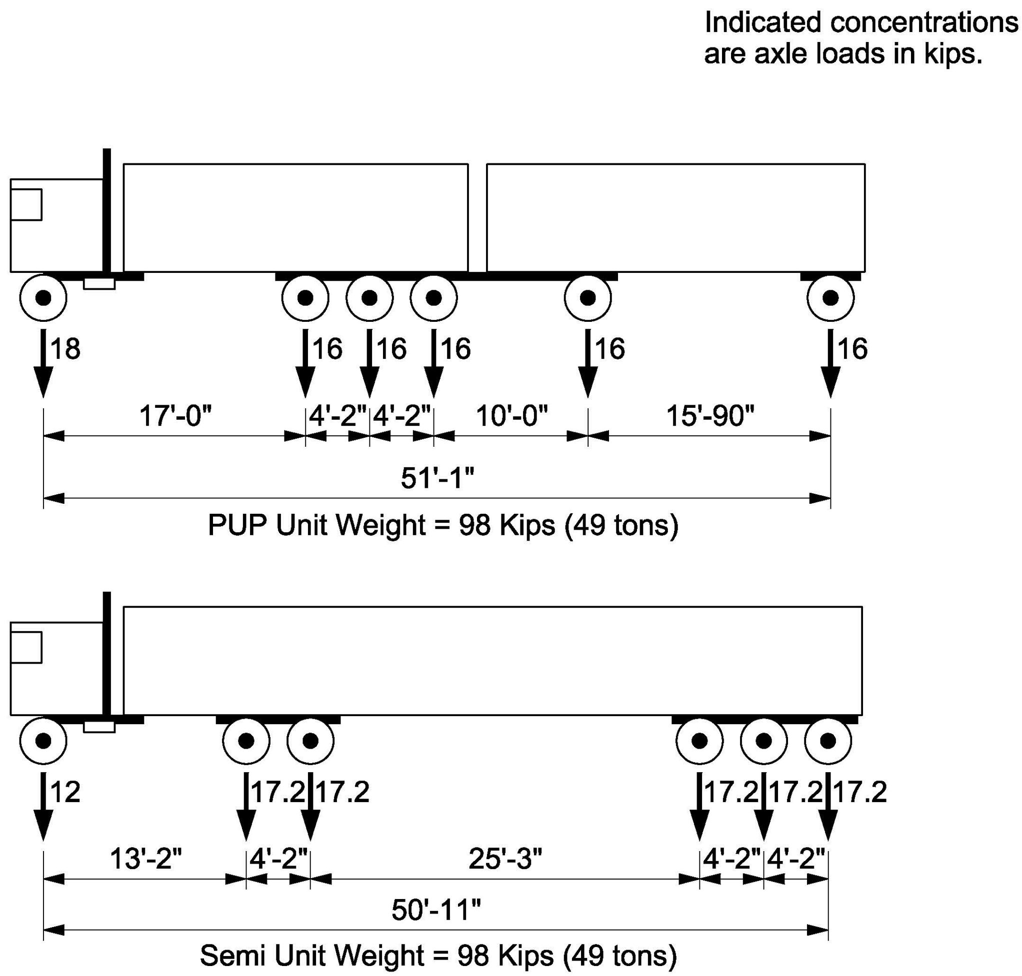
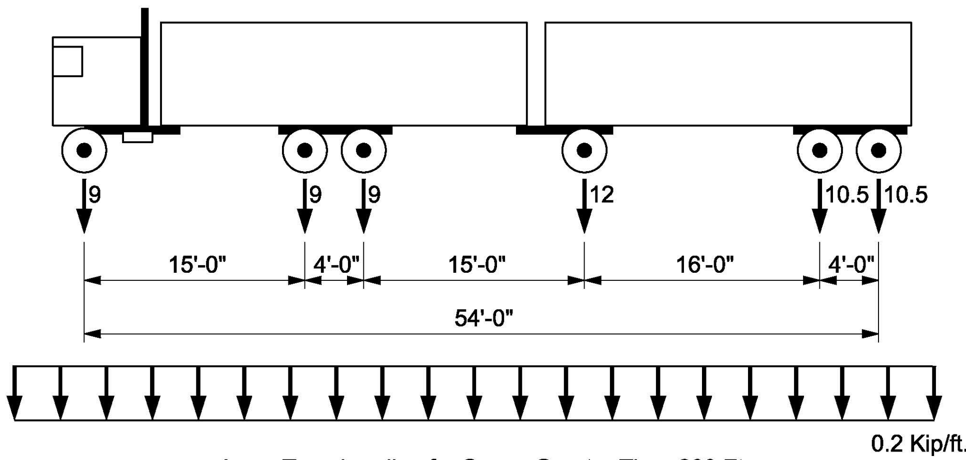
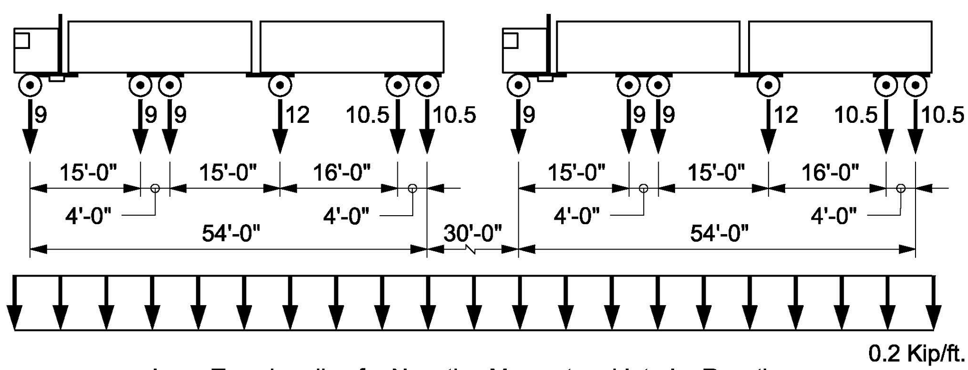


Figure 45.10-3
WisDOT Specialized Annual Permit Vehicles

Indicated concentrations are axle loads in kips (75% of type 3-3).



Lane-Type Loading for Spans Greater Than 200 Ft.



Lane-Type Loading for Negative Moment and Interior Reaction.

Figure 45.10-4

Lane Type Legal Load Models

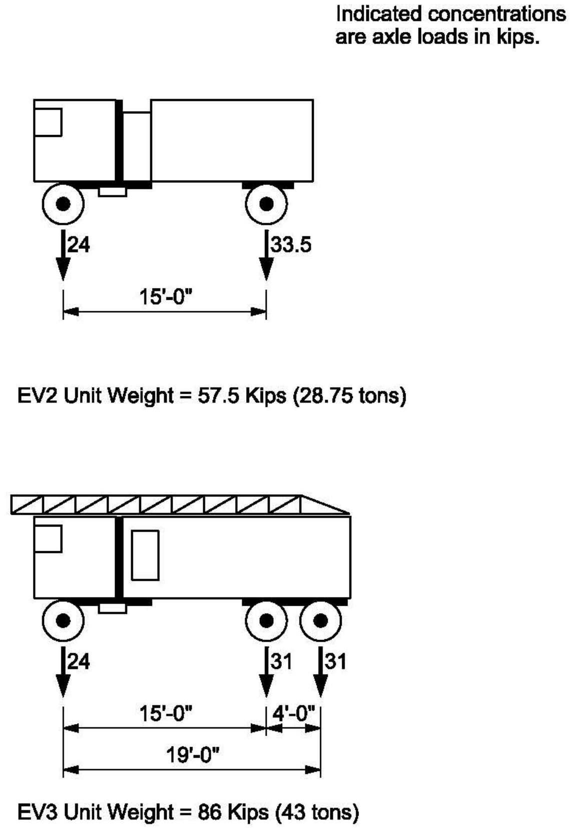


Figure 45.10-5
Emergency Vehicle Load Models

45.10.3 Load Posting Analysis

All posting vehicles shall be analyzed at the operating level. A load posting analysis is required when the calculated rating factor at operating level for a bridge is:

- Less than 1.0 for HL-93 loading using LRFR methodology.
- Less than 1.0 for HS-20 loading using LFR/ASR methodology; or
- Less than or equal to 1.2 for LFR/ASR methodology (SHV analysis only)

A load posting analysis is very similar to a load rating analysis, except the posting live loads noted in 45.10.2 are used instead of typical LFR or LRFR live loading.

If the calculated rating factor at operating is less than 1.0 for a given load posting vehicle, then the bridge shall be posted, with the exception of the Wis-SPV. For State Trunk Highway Bridges, current WisDOT policy is to post structures with a Wisconsin Standard Permit Vehicle (Wis-SPV) rating of 120 kips or less. If the RF ≥ 1.0 for a given vehicle at the operating level, then a posting is not required for that particular vehicle.

A bridge is posted for the lowest restricted weight limit of any of the standard posting vehicles. To calculate the capacity, in tons, on a bridge for a given posting vehicle utilizing LFR, multiply the rating factor by the gross vehicle weight in tons. To calculate the posting load for a bridge analyzed with LRFR, refer to 45.10.3.2.

Posting or weight limit analysis for emergency vehicles occurs separately; it is required when the calculated rating factor at inventory level for a bridge is:

- Less than 0.9 for HL-93 loading using LRFR methodology; or
- Less than 1.0 for HS-20 loading using LFR/ASR methodology.

If the calculated rating factor at operating rating is less than 1.0 for a given emergency vehicle, then the bridge shall have an emergency vehicle-specific weight limit restriction, as follows:

- If RF_{EV2} < 1.0 and RF_{EV3} < 1.0
  - Single Axle = Minimum (RF_{EV2} x 16.75 tons, RF_{EV3} x 31 tons)
  - Tandem = Minimum (RF_{EV2} x 28.75 tons, RF_{EV3} x 31 tons)
  - Gross = Minimum (RF_{EV2} x 28.75 tons, RF_{EV3} x 43 tons)
- If only RF_{EV2} < 1.0
  - Single Axle = RF_{EV2} x 16.75 tons
  - Tandem = RF_{EV2} x 28.75 tons
  - Gross = RF_{EV2} x 28.75 tons
- If RF_{EV2} < 1.0 and RF_{EV3} < 1.0
  - Single Axle = Minimum (16 tons, RF_{EV3} x 31 tons)
  - Tandem = RF_{EV3} x 31 tons
  - Gross = RF_{EV3} x 43 tons

Sign postings may or may not be required for emergency vehicles, depending on their location. Refer to 45.10.4.

45.10.3.1 Limit States for Load Posting Analysis

For LFR methodology, load posting analysis should consider strength-based limit states only.

For LRFR methodology, load posting analysis should consider strength-based limit states, but also some service-based limit states, per Table 45.3-1.

45.10.3.2 Legal Load Rating Load Posting Equation (LRFR)

When using the LRFR method and the operating rating factor (RF) calculated for each legal truck described above is greater than 1.0, the bridge does not need to be posted. When for any legal truck the RF is between 0.3 and 1.0, then the following equation should be used to establish the safe posting load for that vehicle (see **MBE [Equation 6A8.3-1]**):

$$\text{Posting} = \frac{W}{0.7}\left[(RF) - 0.3\right]$$

Where:

RF = Legal load rating factor

W = Weight of the rating vehicle

When the rating factor for any vehicle type falls below 0.3, then that vehicle type should not be allowed on the bridge. If necessary, the structure may need to be closed until it can be repaired, strengthened, or replaced. This formula is only valid for LRFR load posting calculations.

45.10.3.3 Distribution Factors for Load Posting Analysis

WisDOT policy items:

The AASHTO Commercial Vehicles and Specialized Hauling Vehicles shall be analyzed using a multi-lane distribution factor for bridge widths 18'-0" or larger. Single lane distribution factors are used for bridge widths less than 18'-0".

The WisDOT Specialized Annual Permit Vehicles shown in Figure 45.10-3 shall be analyzed using a single-lane distribution factor, regardless of bridge width.

The Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed for load postings using a multi-lane distribution factor for bridge widths 18'-0" or larger. Single lane distribution factors are used for bridge widths less than 18'-0".

45.10.4 Load Posting Signage

Current WisDOT policy is to post State bridges for a single gross weight, in tons. Bridges that cannot carry the maximum weight for the vehicles described in 45.10.2 at the operating level are posted with the standard sign shown in Figure 45.10-6. This sign shows the bridge capacity for the governing load posting vehicle, in tons. The sign should conform to the requirements of the *Wisconsin Manual for Uniform Traffic Control Devices (WMUTCD)*.

In the past, local bridges were occasionally posted with the signs shown in Figure 45.10-7 using the H20, Type 3 and Type 3S2 vehicles. The H20 represented the two-axle vehicle, the Type 3 represented the three-axle vehicle and the Type 3S2 represented the combination vehicle. This practice is not encouraged by WisDOT and is generally not allowed for State-owned structures, except with permission from the State Bridge Maintenance Engineer.

Emergency vehicle posting signs, however, are based on a combination of the single axle, tandem axle, and gross vehicle weight limits, as shown in Figure 45.10-8. Emergency vehicle posting signs are only required for bridges on the Interstate and within reasonable access (one road mile) to or from an Interstate interchange.

WEIGHT LIMIT 10 TONS	BRIDGE CLOSED

Figure 45.10-6
Standard Signs Used for Posting Bridges

WEIGHT LIMIT 2 AXLE VEHICLES 15 TONS 3 AXLE VEHICLES 20 TONS COMBINATION VEHICLES 30 TONS	WEIGHT LIMIT 2 AXLE VEHICLES 14 TONS 3 AXLE VEHICLES 18 TONS COMBINATION VEHICLES 28 TONS	WEIGHT LIMIT 2 AXLE VEHICLES 14 TONS 3 AXLE VEHICLES 18 TONS COMBINATION VEHICLES 28 TONS

Figure 45.10-7
Historic Load Posting Signs

EMERGENCY
VEHICLE
WEIGHT LIMIT

SINGLE AXLE	15 TONS
TANDEM	25 TONS
GROSS	35 TONS

Figure 45.10-8
Emergency Vehicle Load Posting Signs

45.13 References

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