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2.1 Organizational Charts

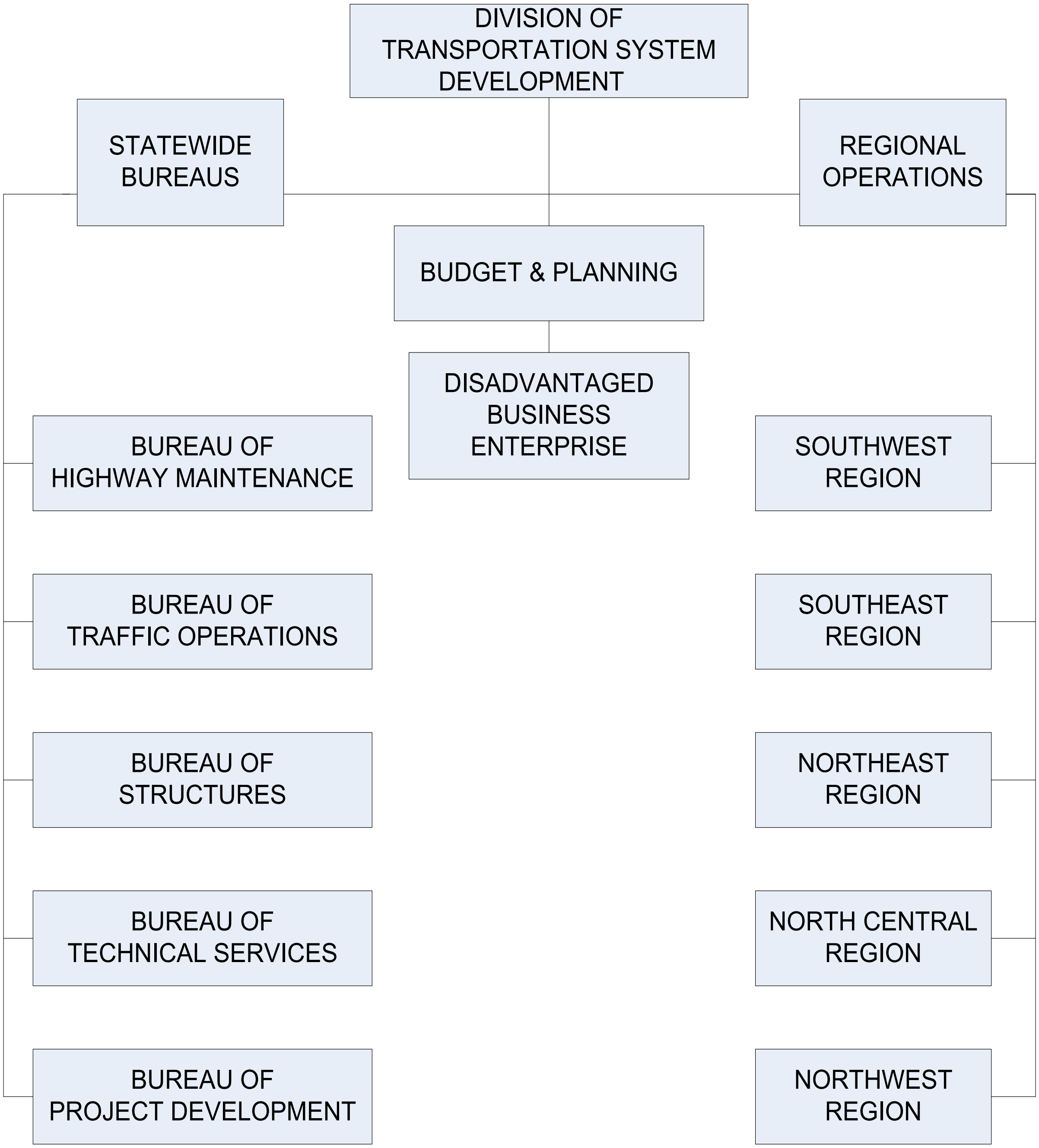

Figure 2.1-1
Division of Transportation System Development

2.5 Structure Numbers

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

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

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

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

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

  - Assign one bridge number to any bridge with a closed median, where the area between the two roadways on the bridge is bridged over and can support traffic sharing a common substructure unit or units. Closed medians may have either mountable or non-mountable curbs or barriers. Refer to Figure 2.5-2.
  - Assign two bridge numbers to separate superstructures with an open median (not meeting the closed median criteria above) sharing a common substructure unit or units. Refer to Figure 2.5-2.
  - Separate bridge numbers be reported for each mainline bridge and the ramp that connects to the mainline bridge, when the ramp has at least one distinct abutment and is greater than 20 feet in length. Separate bridge numbers are to be assigned for a bridge that divides into two or more separate bridges, or two or more bridges that merge into one single bridge. In both cases, the separating point between bridges should be the closest deck joint, or substructure unit to the separating point, or other logical and reasonable location as determined by BOS.
  - Assign a bridge number to any temporary bridge open to highway traffic more than 24 months.

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

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

WisDOT Policy Item:

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

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

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

- R is assigned to permanent retaining walls. For a continuous wall consisting of various wall types, such as a secant pile wall followed by a soldier pile wall, unit numbers should be assigned to each wall type segment. Wall facing discontinuities (e.g. stairwells, staged construction, tiers, or changes to external loads) do not require unique wall numbers if the leveling pad or footing is continuous between the completed wall segments. For soldier pile walls with anchored and non-anchored segments, unique wall numbers are not required for each segment.

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

  Criteria for assigning R numbers based on wall heights:

  - Proprietary retaining walls (e.g., modular block MSE walls)

    - MSE walls - R is assigned to structures with a maximum height greater than or equal to 5.5 ft measured from the bottom of wall or top of leveling pad to top of wall.

    - Modular block gravity walls – R is assigned to structures with a maximum height greater than or equal to 4.0 ft measured from the bottom of wall or top of leveling pad to top of wall.

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

    - R is assigned to structures with a maximum height greater than or equal to 5.5 ft measured from the plan ground line to top of wall.

  In addition to the wall height criteria, walls may warrant a R number if intended to support vehicle loading, support slopes that exceed 2.5:1, or require a geotechnical analysis. For example, a sheet pile wall typically will be assigned a R number when supporting vehicle loading with a 4.8 ft exposed wall height. Note: roadway Standard Detail Drawings (SDD) for walls (e.g., SDD 14B32) may include vehicle loading. In those cases, details following roadway SDD's do not require R numbers when design limitations are not exceeded.

  Walls that do not meet the above criteria for assigning R numbers are deemed "minor retaining walls" and do not require an R number. Refer to FDM 11-55-5.2 for more information. Contact the Bureau of Structures region liaison for more information on assigning R numbers.

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

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

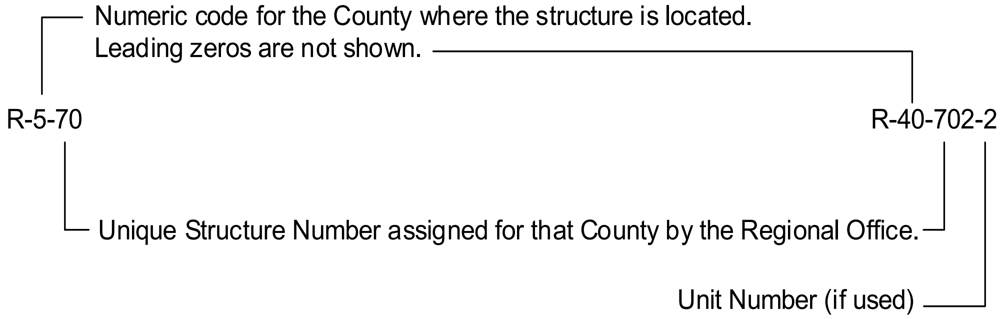

Figure 2.5-1
Structure Number Detail

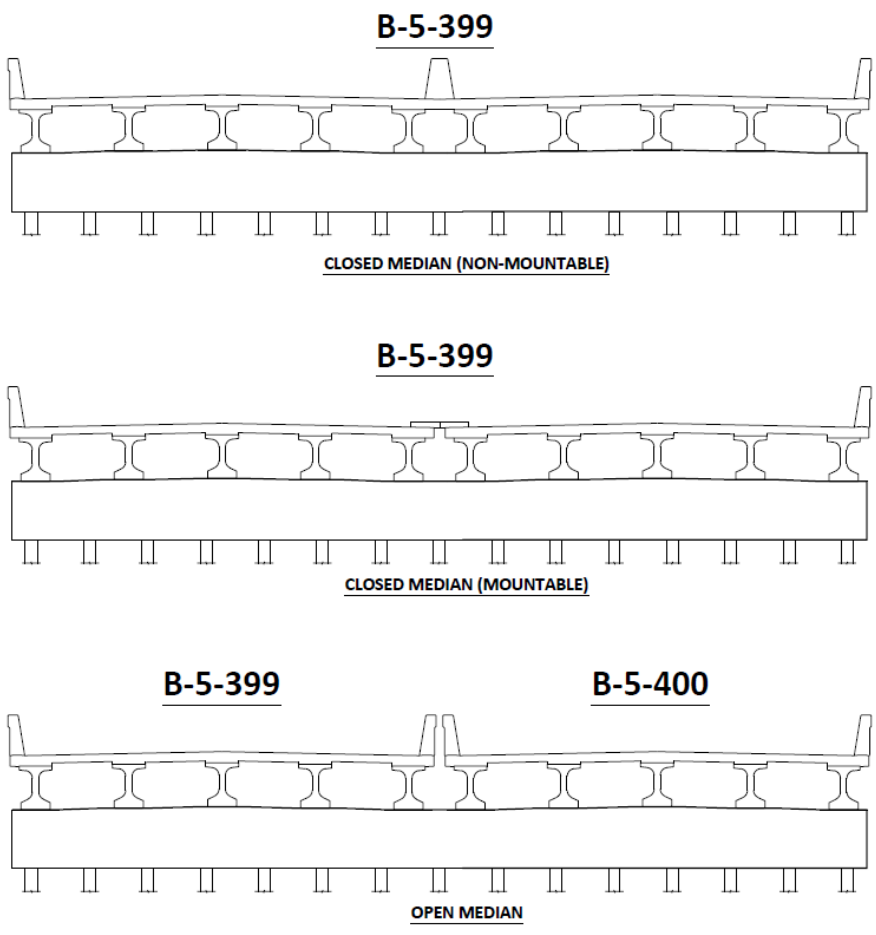

Figure 2.5-2
Structure Numbers for Closed and Open Medians

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4.1 Introduction

Transportation structures, such as bridges and retaining walls, have a strong influence on the appearance of transportation projects, as well as the overall appearance of the general vicinity of the project. In locations where there is an opportunity to appreciate such structures, it is often desirable to add aesthetic enhancements to fit the project site.

Desirable bridge aesthetics do not necessarily need to cost much, if any, additional money. Aesthetic enhancements can be made in a number of ways. Primary features such as structure type and shape have the most influence on appearance, with color and texture playing secondary roles. Formliners, especially when used in conjunction with a multi-colored stain, are more expensive than one or two single color stains on smooth concrete, and have on a number of occasions not fit the context of the project. It is the responsibility of the design team to identify aesthetic treatments that are consistent with the environment and goals of the project, are maintainable over the life of the structures, and are cost effective. See 4.5 for current policy regarding structure aesthetics.

While initial cost for aesthetic enhancements is a concern, it has become apparent that maintenance costs can be considerably more than initial costs. Stain, which acts more like paint, must be periodically redone. Such reapplication oftentimes requires lane closures which are both an undesirable inconvenience to users and come with a significant cost associated with maintenance-of-traffic.

4.3 Primary Features

Superstructure Type and Shape

At highway speeds, highway structures are viewed from 300-500 feet away. The general shape of the bridge, with an emphasis on thinness, produces the most appealing structure. Given that there are realistic physical limitations on thinness (without resorting to anchored end spans or other costly measures), the designer has other options available to achieve the appearance of thinness such as:

- Larger overhangs to create better shadow lines.
- Horizontal recess on the backside of the parapet, which could be stained or left as plain concrete. Unless noted otherwise, any parapet that is non-standard (either side) is considered Community Sensitive Design (CSD).
- Eliminate or minimize pedestals along the parapet. Such pedestals tend to break up the horizontal flow and make the superstructure appear top heavy. Pedestals, if desired, are better left on the wings to delineate the beginning or end of the bridge or to frame the bridge when viewed from below. If used on the superstructure, keep the pedestal size smaller and space apart far enough to avoid a top heavy appearance. See Chapter 30 – Railings for further guidance.
- Minimize vertical or patterned elements on the backside of the parapet as such elements tend to break up the horizontal flow. Rock formliner has become an overused aesthetic enhancement for the backside of parapets, as its use oftentimes does not fit the surroundings. Unless noted otherwise, any parapet that is non-standard (either side) is considered CSD. Rustications or recessed panels may be allowed on the backside of the parapet and are considered non-CSD, provided they meet certain criteria. See Chapter 30 – Railings and 4.9 for further guidance.
- Structure type should be based on economics, not aesthetics. Additional costs associated with a preferred structure type are considered CSD. Add-ons, such as false arches, etc. are considered CSD.

Abutment Type and Shape

Wing walls are the most visible portion of the abutment. Unless pedestrians are beneath a bridge, formliners or other aesthetic enhancements are not very visible and should be left off of the abutment front face, as these *treatments provide no additional aesthetic value.*

Pier Type and Shape

Pier shapes should be kept relatively simple and uncluttered. For highway grade separations, the end elevation of the pier is the view most often seen by the traveling public. For slower speed roads or where pedestrians travel beneath a bridge, the front pier elevation is also seen. For taller piers, such as those used for multi-level interchanges or water crossings, the entire 3D-view of the pier is readily seen and the pier shape is very important. For such piers, a clean, smooth flowing slender shape that clearly demonstrates the flow of forces from the superstructure to the ground is essential. External and internal (reentrant) corners on the pier/column shaft should be kept to a reasonable number. (Approximately 8 external, 4 internal maximum).

Grade and/or Skew

While grade and skew cannot be controlled by the bridge design engineer, these geometric features do affect bridge appearance. For example, a steep grade or pronounced vertical curve makes the use of a block type rustication an awkward choice. Horizontal blocks are typically associated with buildings and block buildings tend to have level roof lines. Cut stone form liners used on steep grades or pronounced vertical curves require excessive cutting of forms, which drives up price. Consideration of abutment height warrants more consideration when bridges are on steep grades, with a more exposed abutment face on the high end of the bridge producing a more balanced look.

Large skews tend to make piers longer as well as making the front elevation of the pier more visible to properties adjacent to the bridge. With larger skews, having more than one multi-columned pier can create a ‘forest’ of pier columns if the columns are too numerous. Try to maximize column spacing or use multiple hammerhead piers to help alleviate this effect. Abutment wings tend to be longer on the acute corners of bridges. Whatever aesthetic treatment is used needs to be appropriate for both the longer and shorter wings.

The design engineer should keep in mind that a bridge is never entirely seen at a 90-degree angle as depicted in a side elevation view. As the person viewing the bridge moves closer to the bridge the pier directly in front of them will be seen nearly as an end elevation of the pier, while adjacent piers will start to be viewed more as a pier side elevation. The ‘forest’ of columns starts to take effect, again, especially for wider bridges.

4.5 Aesthetics Process

The structural design engineer needs to be involved early in the aesthetic decision making process. BOS should have early representation on projects with considerable aesthetic concerns. Throughout this process it is important to remember that aesthetics is a concept, not a commodity – it is about a look, not about what can be added to a structure.

WisDOT policy item:

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

Aesthetic and/or Decorative Amenities (non-Participating, or CSD Amenities)

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

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

Participating (non-CSD) Amenities

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

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

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

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

4.8 Resources on Aesthetics

The *Bridge Aesthetic Sourcebook* from AASHTO is a very good source of practical ideas for short and medium span bridges. The Transportation Research Board (TRB) Subcommittee on Bridge Aesthetics authored this document and it can be found on the following website: The final printing of this guide (noted in the References) is available through the AASHTO publication website:

4.9 Non-CSD Aesthetic Concepts

Standards 4.02-4.05 provide details for acceptable non-CSD funded aesthetic concepts. The three types (Type I, Type II and Type III) show a plain wing, a wing with a rustication trim line and a wing with a recessed panel, respectively. For each given wing type, one or two acceptable parapet and/or pier details are shown.

- Type I: Simple features utilizing a plain wing, standard parapet and minimal pier rustications. Type I is ideal for most rural and some urban applications.

- Type II: The wings utilize the same rustication trim line as the columns. The columns can have single or paired rustication trim lines. Single rustication lines can be used for 32-inch parapets and double rustication lines can be used for 42-inch parapets. Type II can be used in urban applications and other limited areas.

- Type III: Recessed panel wings and recessed panel columns, along with standard parapets (with or without recessed panels), are to be used in urban settings, only.

Within a given corridor, only one Type should be chosen so as not to create a disharmonious experience for those driving the corridor.

The following pages show renderings of the various non-CSD aesthetic concepts.

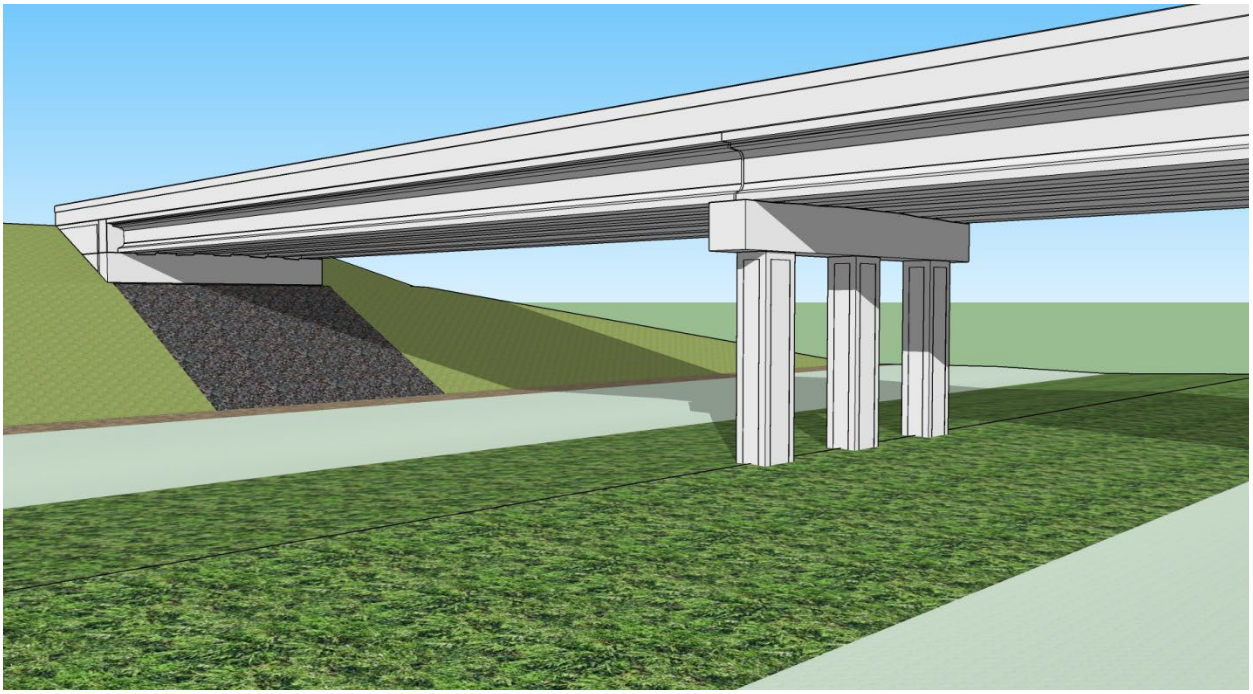

Figure 4.9-2
Aesthetic Concept Type III

- Recessed panel abutment wings
- Recessed panel columns
- Standard parapet (shown) or modified parapet with recessed panels on backside of parapet (not shown)
- Urban applications

4.10 References

1. AASHTO, *Bridge Aesthetics Sourcebook,* 2010.

2. Gottemoeller, Frederick, *Bridgescape: The Art of Designing Bridges*, John Wiley & Sons, Inc., 2004.

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8.1 Introduction

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

8.1.1 Objectives of Highway Drainage

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

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

8.1.2 Basic Policy

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

8.1.3 Design Frequency

Federal and State governments have placed increasing emphasis on environmental protection over the last several years. Consequently, the administrative rules established by regulatory agencies have made past practice of designing structures to accommodate flood frequencies of 25 and 50 years obsolete and unworkable. Thus, the design discharge for all bridges and box culverts covered under this chapter shall be the 100-year (Q100) or 1% chance frequency flood. In floodplain management this is also referred to as the Regional or Base flood. Design frequency is determined from requirements in Federal Highway Administration (FHWA) directives and the co-operative agreement between Wisconsin Department of Transportation (DOT) and Wisconsin Department of Natural Resources (DNR). The following publications are suggested for guidance.

8.3 Hydraulic Design of Bridges

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

8.3.1 Hydraulic Design Factors

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

8.3.1.1 Velocity

Velocity through the bridge opening is a major design factor. Velocity relates to the scour potential in the bridge opening and the development of scour areas adjacent to the bridge. Examination of the “existing conditions” model, existing site conditions, soil conditions, and flooding history will give good insight to acceptable design velocity. Velocities with potential to compromise slope or streambed stability are not acceptable and should be avoided. This threshold will vary depending on site geometry and local stream geomorphology.

8.3.1.2 Roadway Overflow

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

8.3.1.3 Bridge Skew

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

8.3.1.4 Backwater and High-water Elevation

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

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

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

8.3.1.5 Freeboard

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

It has become common practice that if debris and ice are a potential problem, or adequate freeboard cannot be provided, a concrete slab superstructure is preferred. However, a cast-in-place concrete slab superstructure will need sufficient clearance from the normal water elevation for temporary falsework. Refer to Chapter 18.1.2 for additional information. A girder superstructure may be susceptible to damage when ice and/or debris is a significant problem. Girder structures are more susceptible to damage associated with buoyancy and lateral hydrostatic forces. In situations where the superstructure may be inundated during major flood events, it is recommended that the girders be anchored, tied or blocked so they cannot be pushed or lifted off the substructure units by hydraulic forces. In addition, air vents near the top of the girder webs can allow entrapped air to escape and thus may reduce buoyancy forces. The use of Precast Pretensioned Slab and Box Sections is allowed where desirable freeboard cannot be provided and conventional cast in place slabs cannot be employed. The following requirements should be met:

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

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

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

2. HY8

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

8.3.2.6 Develop Hydraulic Model

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

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

The designer shall determine whether the proposed site is located in a FEMA Special Flood Hazard Area (Zone AE, A, etc). If so, a determination shall be made whether an effective hydraulic model (HEC-RAS, HEC-2, WSPRO, etc) exists for the waterway. If an effective model exists, it shall be used to evaluate the impact of the proposed stream crossing structure on mapped floodplain elevations. Areas mapped as Zone AE should always have an effective model. Effective models can be acquired from the DNR or the FEMA Engineering Library. Contact a DNR regional floodplain engineer with any questions related to existing effective models.

The designer should verify that the results of the existing hydraulic model match the flood profile listed in the corresponding Flood Insurance Study (FIS) report. This is called the ‘duplicate effective’ model. The duplicate effective model should then be updated to include geometry based on any recent project survey information. This is called the ‘corrected effective’ model and will serve as the existing condition for the bridge hydraulic analysis.

The Project Engineer shall ensure the appropriate local zoning authority is notified of the results of the hydraulic analysis.

Official bridge hydraulic models and supporting documentation are available for download from the Highway Structures Information System (HSIS).

8.3.2.6.1 Bridge Hydraulics

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

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

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

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

8.3.2.6.2 Roadway Overflow

One potential element in developing a hydraulic model for a stream crossing is roadway overflow. It is sometimes necessary to compute flow over highway embankments in combination with flow through structure openings. Most automated methodologies will incorporate the division of flow through a structure and over the road in determination of the solution. HEC-RAS relies on user defined coefficients for both the structure and roadway flow solutions. The discharge equation and coefficients for flow over a highway embankment are given in this section.

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

The weir discharge equation is:

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

Where:

Computing Contraction Scour.

1. Live-Bed Contraction Scour

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{\frac{6}{7}} \left(\frac{W_1}{W_2}\right)^{k_1}$$

Where:

y_s	=	y_2-y_0 = Average scour depth, ft
y_1	=	Average depth in the upstream main Channel, ft
y_2	=	Average depth in the contracted section, ft
y_0	=	Existing depth in the contracted section before scour, ft
Q_1	=	Flow in upstream channel transporting sediment, ft^3/s
Q_2	=	Flow in contracted channel, ft^3/s
W_1	=	Bottom Width of upstream main channel, ft
W_2	=	Net bottom Width of channel at contracted section, ft
k_1	=	Exponent for mode of bed material transport, 0.59-0.69 see 8.5 ref. (14)

2. Clear-Water Contraction Scour

$$y_2 = \left[\frac{Q^2}{130 \cdot D_m^{\frac{3}{2}} \cdot W^2}\right]^{\frac{3}{7}}$$

Where:

y_s	=	y_2-y_0 = Average scour depth, ft
y_2	=	Average depth in the contracted section, ft
y_0	=	Existing depth in the contracted section before scouring, ft
Q	=	Discharge through the bridge associated with W, ft^3/s
D_m	=	Diameter of the smallest nontransportable particle ($1.25D_{50}$), ft
D_{50}	=	Median Diameter of the bed material (50% smaller than), ft
W	=	Net bottom Width of channel at contracted section, ft

8.3.2.7.4 Local Scour

Local scour is the removal of material from around a pier, abutment, spur dike, or the embankment. It is caused by an acceleration of the flow and/or resulting vortices induced by obstructions to flow.

1. Pier Scour & Colorado State University's (CSU) Equation

   The recommended equation for determination of pier scour is the CSU's equation. Velocity is a factor in calculating the Froude Number. Therefore it is applicable where a hydraulic model of the bridge is available. The equation and appropriate charts and tables are shown below in Table 8.3-1, Table 8.3-2, Table 8.3-3 and Figure 8.3-5. See 8.5 reference (14) for a complete discussion of the CSU Equation.

   The CSU equation for pier scour is:

$$\frac{y_s}{a} = 2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot K_4 \cdot \left(\frac{y_1}{a}\right)^{0.35} \cdot Fr_1^{0.43}$$

Where:

y_s	=	Scour depth, ft
y_1	=	Flow depth directly upstream of the pier, ft
a	=	Pier width, ft
Fr_1	=	Froude number directly upstream of the pier = $V_1/(gy_1)^{1/2}$
V_1	=	Mean Velocity of flow directly upstream of the pier, ft/s
g	=	Acceleration of gravity, 32.2 ft/s^2
K_1	=	Correction Factor for pier nose shape (see Table 8.3-1 and Figure 8.3-5)
K_2	=	Correction Factor for angle of attack of flow (see Table 8.3-2)
K_3	=	Correction Factor for bed condition (see Table 8.3-3)
K_4	=	Correction Factor for armoring by bed material 0.7 - 1.0 (see 8.5 reference 14)

Correction Factor, K_1, for Pier Nose Shape (HEC-18 Table 7.1)	
Shape of Pier Nose	K_1
(a) Square Nose	1.1
(b) Round Nose	1.0
(c) Circular Cylinder	1.0
(d) Group of Cylinders	1.0
(e) Sharp Nose	0.9

Table 8.3-1
Correction Factor, K_1, for Pier Nose Shape

Correction Factor, K_2, for Angle of Attack, Θ, of the Flow (HEC-18 Table 7.2)			
Angle	L/a = 4	L/a = 8	L/a = 12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0
Angle = skew angle of flow L = length of pier, ft a = pier width, ft			

Table 8.3-2
Correction Factor, K_2, for Angle of Attack, θ, of the Flow

Increase in Equilibrium Pier Scour Depths, K_3, for Bed Conditions (HEC-18 Table 7.3)		
Bed Condition	Dune Height, ft	K_3
Clear – water Scour	N/A	1.1
Plane Bed and Antidune Flow	N/A	1.1
Small Dunes	$10 > H \geq 2$	1.1
Medium Dunes	$30 > H \geq 10$	1.2 to 1.1
Large Dunes	$H \geq 30$	1.3

Table 8.3-3
Increase in Equilibrium Pier Scour Depths, K_3, for Bed Condition

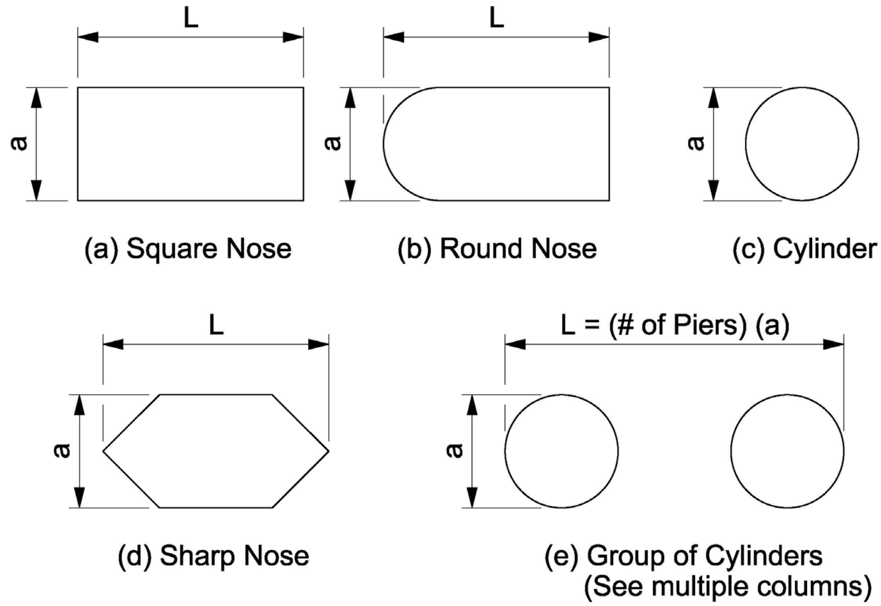

Figure 8.3-5
Common Pier Shapes

2. Abutment Scour Equations

Abutment scour analysis is dependent on equations that relate the degree of projection of encroachment (embankment) into the flood plain.

FHWA publication HEC-18 "Evaluating Scour at Bridges" strongly recommends using the NCHRP Project 24-20 methodology to assess abutment scour. This method includes equations that encompass a range of abutment types and locations, as well as flow conditions. The primary advantage of this approach is that the equations are more physically representative of the abutment scour process, but it also avoids using the effective embankment length, which can be difficult to determine accurately. This approach computes total scour, rather than just local scour, at the abutment. Reference HEC-18 for a detailed description of the NCHRP approach and equations.

Common hydraulic modeling programs used for bridge design typically provide the required hydraulic parameters needed to calculate abutment scour. Designers are cautioned to closely examine how the parameters that are used in these automated routines are defined. FHWA's Hydraulic Toolbox software is commonly used to calculate abutment scour using the NCHRP 24-20 methodology.

The other two methods presented in HEC-18 are the Froehlich and HIRE equations. These methods often predict excessively conservative abutment scour depths. This is due to the fact that these equations were developed based on results of experiments in laboratory flumes and did not reflect the typical geometry or flow distribution associated with roadway encroachments on floodplains. However, since the NCHRP

8.4.2 Design Procedure

8.4.2.1 Determine Design Discharge

See 8.2 for procedures.

8.4.2.2 Determine Hydraulic Stream Slope

See 8.3.2.2 for procedures.

8.4.2.3 Determine Tailwater Elevation

The tailwater elevation is the depth of water in the natural channel computed at the outlet of the culvert. In situations of steeper slopes and small culverts, the tailwater is not a critical design factor. However, for mild slopes and larger culverts, the tailwater is a critical design factor. It may control the outlet velocity and depth of flow in the culvert.

The tailwater elevation is calculated using a typical section downstream of the outlet and performing a “normal depth” analysis. Most hydraulic engineering textbooks and handbooks include discussion of methods to calculate “normal depth” for symmetrical and irregular cross-sections in an open channel.

8.4.2.4 Design Methodology

The most prevalent design methodology for culverts is the procedure in the FHWA publication HDS No. 5, see 8.5 reference (13). It is highly recommended the designer first thoroughly study the methodologies presented in that publication.

Several computer software programs are available from public and private sources which use the same technique and methodology presented in HDS No. 5. One public domain computer program developed by FHWA entitled “HY8” is based on the HDS No. 5 manual. This program and documentation are available from the FHWA web site (see 8.7 Appendix 8-B). HEC-RAS also has culvert options using the same methodology. HEC-RAS has the capability of allowing the user to calculate the tailwater based on a downstream section and to calculate a combination of culvert and roadway overflow.

8.4.2.5 Develop Hydraulic Model

There are two major types of culvert flow: (1) flow with inlet control, and (2) flow with outlet control. For each type of control, different factors and formulas are used to compute the hydraulic capacity of a culvert. Under inlet control, the cross-sectional area, and the inlet geometry at the entrance are of primary importance. Outlet control involves the consideration of the tailwater in the outlet channel, the culvert slope, the culvert roughness, and the length of the culvert barrel, as well as inlet geometry and cross-sectional area.

Another design of Inlet control which is used frequently is “Tapered Inlets” or improved inlets. The slope-tapered and side-tapered inlets are more efficient hydraulically, and can be a more economical design for long culverts in flow with inlet control.

In all culvert design, headwater depth (HW) or depth of water at the entrance to a culvert is an important factor in culvert capacity. The headwater depth is the vertical height from the culvert invert elevation at the entrance to the total energy elevation of the headwater pool (depth plus velocity head). Because of the low velocities at the entrance in most cases and difficulty in determining the velocity head for all flows, the water surface elevation and the total energy elevation at the entrance are assumed to be coincident.

The box culvert charts presented here are inlet and outlet control nomographs Figure 8.4-3 and Figure 8.4-4, and a critical depth chart Figure 8.4-6. Note the “Inlet Type” over the HW/D scales on Figure 8.4-3 and entrance loss coefficients “K_e” for inlet types on Figure 8.4-4. The following illustrative problems are examples of their use. Forms similar to Figure 8.4-2 are used for computation.

1. Outlet Control Problem.

   The information necessary to solve this problem is given in Figure 8.4-2.

   Check for Inlet Control: For a Q/B value of 36 and a twin 10 x 5 box with type “C” inlet; HW/D=1.08 from Figure 8.4-3.

   The HW = 1.08 (5 ft) = 5.4 ft.

   Check for Outlet Control: For Q = 720/2 = 360 cfs. Length = 180 ft. and type “C” inlet; H = 1.5 ft. from Figure 8.4-4, TW = 5.2 ft. = ho

   Then HW = H + h_o - LS_o = 1.5 ft. + 5.2 ft. - .2 ft. = 6.5 ft.

   Design HW is 6.5 ft. (outlet controls) and the outlet velocity is 7.2 f.p.s. No heavy riprap is needed at the discharge apron.

2. Inlet Control Problem.

   The information necessary to solve this problem is given in Figure 8.4-5.

   Check for Inlet Control: For a Q/B value of 36 and a twin 10 x 5 box with type “C” inlet; HW/D = 1.08 from Figure 8.4-3.

   Then HW = 1.08 (5 ft.) = 5.4 ft.

   Check for Outlet Control: For Q = 720/2 = 360 cfs. Length = 132 ft. and type “C” inlet; H = 1.3 ft. from Figure 8.4-4. From Figure 8.4-6 critical depth = 3.4 ft. ho = (3.4 ft. + 5 ft.)/2 = 4.2 ft.

   Then HW = H + h_o - LS_o = 1.3 ft. + 4.2 ft. - .7 ft. = 4.8 ft.

   Design HW = 5.4 ft. (inlet control) and the outlet velocity is 11.0 f.p.s. Heavy riprap is needed at the discharge apron.

$$B = \left(\frac{0.315 \cdot Q^{2/3}}{d_c}\right)^{3/2} = \left(\frac{0.315 \cdot 800^{2/3}}{5}\right)^{3/2} = 13'$$

Flare from B = 9 ft to B = 13 ft at an angle of 150/13.5 = 11^0

$$\text{Length} = \frac{\left(\frac{13-8}{2}\right)}{\tan 11^0} = 13'$$

$$\text{Specific Head, } H_A = 7.5 + \frac{V_A{}^2}{2g} = \frac{13.5^2}{2 \cdot 32.2} = 10.33'$$

By trial and error; assume $\frac{V_B{}^2}{2g} = 7.5'$

$$V_B = (2 \cdot 32.2 \cdot 7.5)^{1/2} = 22\text{fps}$$

Elevation head (depth) = 10.33-7.2 = 2.83'

Check trial; Q = AV = (13x2.83)x22 = 809 cfs, Q_{actual} = 800 cfs, OK

$$d_c = 0.315 \cdot \sqrt[3]{\left(\frac{Q}{B}\right)^2} = 0.315 \cdot \left(\frac{800}{13}\right)^{2/3} = 0.315 \cdot 15.6 = 4.91'$$

$$\frac{h_v}{H} = \frac{\left(\frac{V_B{}^2}{2g}\right)}{10.33} = \frac{7.5}{10.33} = 0.725 > \frac{1}{3} \qquad \therefore X_a{}^2 = \frac{2V^2}{g} y_1$$

$$X_a = \left[\frac{2 \cdot 22^2 \cdot (5 + 2.83)}{32.2}\right]^{1/2} = 15.35' \qquad \text{Use } X_a = 15'\text{-}6''$$

Dimensions:

Height of floor blocks	=	0.8 x 4.91 = 4'-0"
Height of end sill	=	0.4 x 4.91 = 2'-0"
Length of Basin	=	15.5+2.55 d_c = 28'
Floor Blocks	=	2'-0" square

Height of Sidewalls = $(2.15 + 0.85)d_c$ = 14.48' above basin floor. Use 13'-0"

8.4.2.7.3 Hydraulic Jump Stilling Basins

The simplest form of a hydraulic jump stilling basin has a straight centerline and is of uniform width. A sloping apron or a chute spillway is typically used to increase the Froude number as the water flows from the culvert to the stilling basin. The outlet barrel of the culvert is also sometimes flared to decrease y_1 so that the tailwater elevation necessary to cause a hydraulic jump need not be so high. This is done using the 150/V relationship as in the drop outlet sample problem. y_1 is usually kept in the 2-3 foot range.

Referring to Figure 8.4-12, the required tailwater is computed by the formula:

$$y_2/y_1 = \tfrac{1}{2}\ [(1+8F_1^2)^{1/2} - 1]$$

Where:

y_2 = tailwater depth required to cause the hydraulic jump

y_1 = water depth at beginning of hydraulic jump

F_1 = Froude number = $v_1 / (gy_1)^{1/2}$

g = acceleration of gravity

v_1 = velocity at beginning of hydraulic jump

End sill height (ΔZ_0) is determined graphically from Figure 8.4-13

Length of jump is assumed to be 6 times the depth change (y_2-y_1).

In many cases the tailwater height isn't deep enough to cause the hydraulic jump. To remedy this, the slope of the culvert may be increased to greater than the slope of the streambed. This will result in an apron depressed such that normal tailwater is of sufficient depth.

The problem of scour on the downstream side of the end sill can be overcome by providing riprap in the stream bottom. If riprap is used, it starts from the top of the sill at a maximum slope of 6:1 up from end sill to original streambed. If no riprap is used, the streambed begins at the top of the end sill.

More detailed discussion about the various types of hydraulic jump stilling basins and their design can be found in 8.5 reference (20).

Sample computations are shown in 8.4.2.7.3.1.

Assume:

$y_1 = 2.2$ and $\frac{V_1^2}{2 \cdot g} = 9.775 - 2.2 = 7.575'$

$V_1 = (2 \times 32.2 \times 7.575)^{1/2} = 22.1$ fps

$Q = 600 = AV = 2.2 \times \text{width} \times 22.1$, width = 12.36

$$\text{Length of flare} = \frac{\frac{(12.36-7)}{2}}{\tan 9^0} = 17'$$

$Y_1 = 2.20$

$V_1 = 22.1$

$$F_1 = \frac{V_1}{\sqrt{g \cdot y_1}} = \frac{22.1}{\sqrt{32.2 \times 2.2}} = 2.63$$

$$y_2 = y_1 \cdot \frac{1}{2} \cdot \left(\sqrt{1 + 8 \times 2.63^2} - 1\right) = 7.15$$

$L = 6(y_2 – y_1) = 6\ (7.15 – 2.20) = 29.7'$ use L = 30 ft.

Assume $y_3 = 5'$

$y_3/y_1 = 5/2.2 = 2.27$

From Figure 8.4-13, $\Delta Z_o/y_1 = 0.5$

$\Delta Z_o = 1.1$, use 1'-6"

8.4.2.7.4 Riprap Stilling Basins

The riprap stilling basins, in many cases, is a very economical approach to dissipate energy at culvert outlets and avoid damaging scour. A good treatise on riprap stilling basin is given in the FHWA Hydraulic Design of Energy Dissipators for Culverts and Channels, see 8.5 reference (20).

8.4.2.8 Select Culvert Design Alternatives

The "proposed culvert" design shall be based on several design factors. In most design situations, the pertinent hydraulic factors discussed in 8.4.1 will dictate the final selection of culvert size, length, scour protection, as well as the approach roadway design.

8.5 References

1. Wisconsin Department of Natural Resources, *Wisconsin's Floodplain Management Program, Chapter NR116*, Register, August 2004, No. 584.

2. Levin, S.B., and Sanocki, C.A., 2023, Estimating Flood Magnitude and Frequency for Unregulated Streams in Wisconsin: U.S. Geological Survey Scientific Investigations Report 2022–5118, 25 p., https://doi.org/10.3133/sir20225118. [Supersedes Scientific Investigations Report 2016–5140.] This report can be found on the USGS web site using the following link:

   https://pubs.usgs.gov/publication/sir20225118

3. U. S. Geological Survey, *Guidelines for Determining Flood Flow Frequency, Bulletin #17C* Revised May 2019.

4. U.S. Department of Agriculture, National Resources Conservation Service, Urban Hydrology for Small Watersheds, Technical Release 55 (2nd Edition), June 1986.

5. Ven Te Chow, Ph.D. *Open Channel Hydraulics* (New York, McGraw-Hill Book Company 1959).

6. U.S. Department of Transportation, Federal Highway Administration, *Design Charts for Open-Channel Flow Hydraulic Design,* Series No. 3, August 1961.

7. U.S. Army Corps of Engineers, *HEC-RAS River Analysis System Users Manual*, (CPD-68), Hydrologic Engineering Center, Davis CA, Version 4.1, January 2010.

8. U.S. Army Corps of Engineers, *HEC-RAS River Analysis System Hydraulic Reference Manual* (CPD-69), Hydrologic Engineering Center, Davis CA, Version 5.0, February 2016.

9. U.S. Army Corps of Engineers, *HEC-RAS River Analysis System Applications Guide* (CPD-70), Hydrologic Engineering Center, Davis CA, Version 4.1, January 2010.

10. U.S. Department of Interior, Geological Survey, *Measurement of Peak Discharge at Width Contractions by Indirect Methods; Techniques of Water-Resources Investigation of the U.S.G.S.*, Chapter A4, Book 3, Third printing 1976.

11. L.A. Arneson and J.O. Shearman, *User's Manual for WSPRO-A computer Model for Water Surface Profile Computations,* FHWA Report No. FHWA-SA-98-080, June 1998.

12. J.O. Shearman, W. H. Hirby, V.R. Schneider, H.N. Flippo, *Bridge Waterways Analysis Model*, Research Report, FHWA Report No. FHWO-RD-86/108, July 1986

13. U.S. Department of Transportation, FHWA, Hydraulic Design Series (HDS), Number 5, *Hydraulic Design of Highway Culverts*, Third Edition, April 2012.

14. U.S Department of Transportation, Federal Highway Administration, *Hydraulic Engineering Circular No. 18, Evaluating Scour at Bridges*, 5th Edition, April 2012.

15. U.S Department of Transportation, Federal Highway Administration, *Hydraulic Engineering Circular No. 20, Stream Stability at Highway Structures*, 4th Edition, April 2012.

16. U.S. Department of Transportation, Federal Highway Administration, *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*, Office of Engineering, Bridge Division, Report No. FHWA-PD-96-001, December 1995.

17. U.S. Department of Transportation, Federal Highway Administration, *Highways in the River Environment: Roads, Rivers, and Floodplains, Second Edition, Hydraulic Engineering Circular (HEC) No. 16,* Report No. FHWA HIF-23-004, January 2023.

18. U.S. Department of Transportation, FHWA, *Debris-Control Structures, Evaluation and Countermeasures, Third Edition,* Hydraulic Engineering Circular (HEC) No.9, Publication No. FHWA-IF-014-016, October 2005.

19. U.S. Department of Interior, Bureau of Reclamation, *Design of Small Dams*, 3rd Edition, Washington D.C. 1987.

20. U.S. Department of Transportation, Federal Highway Administration, *Hydraulic Design of Energy Dissipators for Culverts and Channels*, Hydraulic Engineering Circular (HEC) No. 14, Third Edition, Publication No. FHWA-NHI-06-086, July 2006.

21. Blaisdell, Fred W. and Donnelly, Charles A., *Hydraulic Design of the Box Inlet Drop Spillway*, U.S. Department of Agriculture, Soil Conservation Service, SCS-TP-106, July, 1951.

22. Blaisdell, Fred W. and Donnelly, Charles A., *Straight Drop Spillway Stilling Basin*, University of Minnesota, St. Anthony Falls Hydraulic Laboratory, November, 1954.

8.6 Appendix 8-A, Check List for Hydraulic/Site Report

A hydraulic and site report shall be prepared for all stream crossing bridge and culvert projects that are completed by consultants. The report shall be submitted to the Bureau of Structures for review along with the “Stream Crossing Structure Survey Report” and preliminary structure plans (see WisDOT Bridge Manual, 6.2.1). The hydraulic and site report needs to include information necessary for the review of the hydraulic analysis and the type, size and location of proposed structure. The following is a list of the items that need to be included in the hydraulic site report:

- Document the location of the stream crossing or project site. Indicate county, municipality, Section, Town, and Range.

- List available information and references for methodologies used in the report. Indicate when survey information was collected and what vertical datum was used as reference for elevations used in hydraulic models and shown on structure plans. Indicate whether the site is in a mapped flood hazard area and type of that mapping, if any.

- Provide complete description of the site, including description of the drainage basin, river reach upstream and downstream of the site, channel at site, surrounding bank and over bank areas, and gradient or slope of the river. Also, provide complete description of upstream and downstream structures.

- Provide a summary discussion of the magnitude and frequency of floods to be used for design. Hydrologic calculations shall be provided to the Bureau of Structures beforehand for their review and concurrence. Indicate in the hydraulic site report when calculations were submitted and whether approval was obtained.

- Provide a description of the hydraulic analyses performed for the project. Indicate what models were used and the basis for and assumptions used in the selection of various modeling parameters. Specifically, discuss the assumptions used for defining the modeling reach boundary conditions, roughness coefficients, location and source of hydraulic cross sections, and any assumptions made in selecting the bridge modeling methodology. (Hydraulic calculations shall be submitted with the hydraulic site report).

- Provide a complete description of the existing structure, including a description of the geometry, type, size and material. Indicate the sufficiency rating of the structure. Provide information about observed scour, flooding, roadway overtopping, ice or debris, navigation clearance and any other structurally or hydraulically pertinent information. Provide a discussion of calculated hydraulic characteristics at the site.

- Provide a description of the various sizing constraints considered at the site, including but not limited to regulatory requirements, hydraulic and roadway geometric conditions, environmental and constructability considerations, etc.

- Provide a discussion of the alternatives considered for this project including explanations of how certain alternatives are removed from consideration and how the recommended alternative is selected. Include a cost comparison.

- Provide complete description of proposed structure including calculated hydraulic characteristics.

- Provide a discussion of calculated scour depths for each scour component (LTD, Contraction, Local), total scour elevation and assigned scour code. This section should also include a discussion of proposed foundation type and depths, soil stratigraphy and ultimately a confirmation of structural stability at the total scour condition.

  Scour calculations shall be submitted with the hydraulic site report and should consist of hydraulic modeling outputs highlighting pertinent variables used for the analysis as well as output from a scour computation program (Hydraulic Toolbox, spreadsheet, etc). Scour calculations automatically performed by HEC RAS will not be accepted.

- Provide a summary table comparing calculated hydraulic characteristics for existing and proposed conditions.

8.7 Appendix 8-B, FHWA Hydraulic Engineering Publications

Note: Some links may be obsolete. For the full list of publications see the FHWA website: https://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm

Code	Title	Year	Publication #	NTIS #
HDS 01	Hydraulics of Bridge Waterways	1978	FHWA-EPD-86-101	PB86-181708
HDS 02	Highway Hydrology Third Edition	2024	FHWA-HIF-24-007	
HDS 03	Design Charts for Open-Channel Flow	1961	FHWA-EPD-86-102	PB86-179249
HDS 04	Introduction to Highway Hydraulics	2008	FHWA-NHI-08-090	
HDS 05	Hydraulic Design of Highway Culverts, Third Edition	2012	FHWA-HIF-12-026	PB20-12112032
HDS 06	River Engineering for Highway Encroachments	2001	FHWA-NHI-01-004	PB20-06114029
HDS 07	Highway Design of Safe Bridges, Second Edition	2024	FHWA-HIF-24-001	
HEC 09	Debris Control Structures Evaluation and Countermeasures	2005	FHWA-IF-04-016	
HEC 11	Design of Riprap Revetment	1989	FHWA-IP-89-016	
HEC 14	Hydraulic Design of Energy Dissipators for Culverts and Channels	2006	FHWA-NHI-06-086	
HEC 15	Design of Roadside Channels with Flexible Linings, Third Edition	2005	FHWA-IF-05-114	
HEC 16	Highways in the River Environment: Roads, Rivers, and Floodplains, Second Edition	2023	FHWA-HIF-23-004	
HEC 17	Highways in the River Environment - Floodplains, Extreme Events, Risk, and Resilience, 2nd Edition	2016	FHWA-HIF-16-018	
HEC 18	Evaluating Scour at Bridges, Fifth Edition	2012	FHWA-HIF-12-003	
HEC 19	Highway Hydrology: Evolving Methods, Tools, and Data	2023	FHWA-HIF-23-050	
HEC 20	Stream Stability at Highway Structures Fourth Edition	2012	FHWA-NIF-12-004	
HEC 21	Design of Bridge Deck Drainage	1993	FHWA-SA-92-010	
HEC 22	Urban Drainage Design Manual Fourth Edition	2024	FHWA-HIF-24-006	
HEC 23	Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance Third Edition, Volume 1	2009	FHWA-NHI-09-111	
HEC 23	Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance Third Edition, Volume 2	2009	FHWA-NHI-09-112	
HEC 24	Highway Stormwater Pump Station Design	2001	FHWA-NHI-01-007	PB20-01107891
HEC 25	Highways in the Coastal Environment - 3rd edition	2020	FHWA-HIF-19-059	

Code	Title	Year	Publication #	NTIS #
HEC 26	Culvert Design for Aquatic Organism Passage	2010	FHWA-HIF-11-008	
HIF	Emerging Issues Associated with Sea Level Rise: Findings from FHWA Peer Exchanges	2022	FHWA-HIF-22-051	
HIF	Infrastructure Resilience to Extreme Events and Climate Change - Federal Lands Sensitivity Case Studies	2022	FHWA-HIF-22-043	
HRT	Advanced Methodology to Assess Riprap Rock Stability At Bridge Piers and Abutments	2017	FHWA-HRT-17-054	
HRT	Assessing Stream Channel Stability at Bridges in Physiographic Regions	2006	FHWA-HRT-05-072	PB20-07100098
HRT	Bridge Pressure Flow Scour for Clear Water Conditions	2009	FHWA-HRT-09-041	PB20-10104557
HRT	Effects of Inlet Geometry on Hydraulic Performance of Box Culverts	2006	FHWA-HRT-06-138	PB20-10104553
HRT	Fish Passage in Large Culverts With Low Flow	2014	FHWA-HRT-14-064	PB20-15100735
HRT	Hydraulic Performance of Shallow Foundations for The Support of Vertical-Wall Bridge Abutments	2017	FHWA-HRT-17-013	
HRT	Junction Loss Experiments: Laboratory Report	2007	FHWA-HRT-07-036	
HRT	Hydraulics Laboratory Fact Sheet	2007	FHWA-HRT-07-054	
HRT	Hydrodynamic Forces on Inundated Bridge Decks	2009	FHWA-HRT-09-028	PB20-09111423
HRT	Pier Scour in Clear-Water Conditions With Non-Uniform Bed Materials	2012	FHWA-HRT-12-022	
HRT	Scour in Cohesive Soils	2015	FHWA-HRT-15-033	PB20-15105088
HRT	Submerged Flow Bridge Scour Under Clear Water Conditions	2012	FHWA-HRT-12-034	PB20-12114232
HRT	Updating HEC-18 Pier Scour Equations for Noncohesive Soils	2016	FHWA-HRT-16-045	
Other	Office of International Program Successes Video: HPIP Technologies and Benefits - YouTube	2022		
Other	2D Hydraulic Modeling for Highways in the River Environment	2019	FHWA-HIF-19-061	
Other	Design for Fish Passage at Roadway-Stream Crossings: Synthesis Report	2007	FHWA-HIF-07-033	
Other	NCHRP Report 25-25 (04) Environmental Stewardship Practices, Procedures, and Policies for Highway Construction and Maintenance	2004		
Other	Structural Design Manual for Improved Inlets and Culverts	1983	FHWA-IP-83-6	PB84-153485
Other	Tsunami Design Guidelines Factsheet: International Collaboration Improves Earthquake and Tsunami Hazard Mitigation in the United States	2022		

Code	Title	Year	Publication #	NTIS #
Other	Underwater Bridge Inspection	2010	FHWA-NHI-10-079	
Other	Underwater Bridge Repair, Rehabilitation, and Countermeasures	2010	FHWA-NHI-10-029	
Other	Culvert Inspection Manual	1986	FHWA-IP-86-2	PB87-151809
RC	Benchmarking of SRH-2D	2021	FHWA-RC-21-006	
RD	Bottomless Culvert Scour Study: Phase II Laboratory Report	2007	FHWA-HRT-07-026	
RD	Effects of Gradation and Cohesion on Scour, Volume 2, "Experimental Study of Sediment Gradation and Flow Hydrograph Effects on Clear Water Scour Around Circular Piers"	1999	FHWA-RD-99-184	PB2000-103271
RD	Effects of Gradation and Cohesion on Scour, Volume 1, "Effect of Sediment Gradation and Coarse Material Fraction on Clear Water Scour Around Bridge Piers"	1999	FHWA-RD-99-183	PB2000-103270
RD	Portable Instrumentation for Real Time Measurement of Scour At Bridges	1999	FHWA-RD-99-085	PB2000-102040
RD	Users Primer for BRI-STARS	1999	FHWA-RD-99-191	PB2000-107371
RD	Effects of Gradation and Cohesion on Scour, Volume 3, "Abutment Scour for Nonuniform Mixtures"	1999	FHWA-RD-99-185	PB2000-103272
RD	Remote Methods of Underwater Inspection of Bridge Structures	1999	FHWA-RD-99-100	PB9915-7968
RD	Hydraulics of Iowa DOT Slope-Tapered Pipe Culverts	2001	FHWA-RD-01-077	PB20-06101932
RD	Users Manual for BRI-STARS	1999	FHWA-RD-99-190	PB2000-107372
RD	Effects of Gradation and Cohesion on Scour, Volume 4, "Experimental Study of Scour Around Circular Piers in Cohesive Soils"	1999	FHWA-RD-99-186	PB2000-103273
RD	Effects of Gradation and Cohesion on Scour, Volume 5, "Effect of Cohesion on Bridge Abutment Scour"	1999	FHWA-RD-99-187	PB2000-103274
RD	Effects of Gradation and Cohesion on Scour, Volume 6, "Abutment Scour in Uniform and Stratified Sand Mixtures"	1999	FHWA-RD-99-188	PB2000-103275
RD	Durability Analysis of Aluminized Type 2 Corrugated Metal Pipe	2000	FHWA-RD-97-140	PB20-00103824
RD	Performance Curve for a Prototype of Two Large Culverts in Series Dale Boulevard, Dale City, Virginia	2001	FHWA-RD-01-095	PB20-06114025
RD	Bottomless Culvert Scour Study: Phase I Laboratory Report	2002	FHWA-RD-02-078	
RD	Bridge Scour in Nonuniform Sediment Mixtures and in Cohesive Materials: Synthesis Report	2003	FHWA-RD-03-083	
RD	Enhanced Abutment Scour Studies For Compound Channels	2004	FHWA-RD-99-156	PB20-05102667
RD	Field Observations and Evaluations of Streambed Scour at Bridges	2005	FHWA-RD-03-052	PB20-05106540

Code	Title	Year	Publication #	NTIS #
RD	South Dakota Culvert Inlet Design Coefficients	1999	FHWA-RD-01-076	PB20-06101908
TechBrief	Hydraulic Considerations for Abutments on Deep Foundations and Bridge Embankment Protection	2023	FHWA-HIF-23-048	
TechBrief	Hydraulic Considerations for Shallow Abutment Foundations	2018	FHWA-HIF-19-007	
TechBrief	Overview on Practices on 2D Models	2019	FHWA-HIF-19-058	
TechBrief	Pier Scour Estimation for Tsunami at Bridges, Office of Research, Development, and Technology	2021	FHWA-HRT-21-073	
TechBrief	Scour Considerations within AASHTO LRFD Design Specifications	2021	FHWA-HIF-19-060	
TechBrief	Scour Design within AASHTO LRFD Limit States	2023	FHWA-HIF-23-040	
AOP 01	Aquatic Organism Passage at Highway Crossings: An Implementation Guide	2024	FHWA-HIF-24-054	
CFL	Culvert Assessment and Decision-Making Procedures Manual, For Federal Lands Highway, First Edition	2010	FHWA-CFL/TD-10-005	
CFL	Culvert Pipe Liner Guide and Specifications	2005	FHWA-CFL/TD-05-003	PB20-07105405
EDC	A Primer on Modeling in the Coastal Environment	2017	FHWA-HIF-18-002	

Figure 8.7-1
FHWA Hydraulic Engineering Publications

FHWA Hydraulics Engineering Software		
Software	Title	Year
HY 7	WSPRO User's Manual (Version 061698) (pdf 2.1 MB)	1998
HY 8	Culvert Hydraulic Analysis Program, Version 8.0.1.2	2025
HDS 5	HDS 5 Hydraulic Design of Highway Culverts Third Edition (pdf, 49 mb)	2012
HEC-RAS	Hydrologic Engineering Center's River Analysis System (HEC-RAS)	2016
Climate Adaptation	Climate Change Adaptation Tools	2016
Climate Adaptation	CMIP Processing Tool Version 2.1	2020
FESWMS	FESWMS User's Manual	2003
Toolbox	Hydraulic Toolbox Version 5.4.0	2024
Toolbox	Urban Drainage Design Manual Third Edition	2009
Hydraulics Software by Others		
Software	Title	Year
HY 7	Bridge Waterways Analysis Model (WSPRO)	2016
BCAP	Broken-back Culvert Analysis Program (Version 4.11c)	
CHL	Coastal & Hydraulics Laboratory USACE	
FishXing	Fish Passage through Culverts USF	2012
HEC	Hydrologic Engineering Center USACE	
HyperCalc	HyperCalc Plus	2002
NSS	National Streamflow Statistics Program	2021
PEAKFQ	PEAKFQ	1995
SMS	Surface-Water Modeling System (SMS)	2023
SMS	SRH-2D Modeling Instructions and Guidance	
SMS	SRH-2D Tutorials (Basic Simulations, Bridge Pressure Flow, Culverts, Weirs, Diversions, etc.)	
StreamStats	StreamStats	
USGS	Water Resources Applications Software USGS	
WMS	Watershed Modeling System (WMS)	2016
WMS	WMS Instructions and Support	
WMS	WMS Tutorials	

Figure 8.7-2
FHWA Hydraulics Software List

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9.1 General

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

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

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

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

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

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

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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 Manual, **LRFD [10.4]** and FHWA Geotechnical Engineering Circular No. 5 – Geotechnical Site Characterization (2016). .

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 WisDOT Geotechnical Manual . 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:

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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 [11.5.5]** and **LRFD [11.6] thru [11.12]**. 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
Nongravity Cantilever and Anchored Walls		
Axial Compressive resistance of vertical elements		**LRFD [10.5]**
Passive resistance of vertical elements		0.75
Pullout resistance of anchors	• Cohesionless (granular) soils • Cohesive soils • Rock	0.65 0.70 0.50
Pullout resistance of anchors	• Where proof tests are conducted	1.0
Tensile resistance of anchor tendon	• Mild steel • High-strength steel	0.90 0.80
Overall stability, soil failure		**LRFD [11.6.3.7]**
Flexural capacity of vertical elements		0.90
Mechanically Stabilized Earth Walls, Gravity Walls, and Semi-Gravity		
Bearing resistance	• Gravity & Semi-gravity walls • MSE walls	0.55 0.65
Sliding	• Soil shear component • Passive component	1.00 0.50
Tensile resistance of metallic reinforcement and connectors	• Strip reinforcement • Grid reinforcement	0.75 0.65
Tensile resistance of geo-synthetic reinforcements and connectors	• Geotextile and Geogrid rein. • Geostrip rein.	0.80 0.55
Pullout resistance of tensile reinforcement	• Metallic reinforcement • Geosynthetic reinforcement	0.90 0.70
Service Limit, for soil failure using stiffness method		1.0
Overall and compound stability • Tensile Resistance • Pullout Resistance	• Soil Failure • Metallic reinforcement (strips) • Metallic reinforcement (grid) • Geosynthetic reinforcement • All reinforcement	**LRFD [11.6.3.7]** 0.75 0.65 0.90 0.90
Prefabricated Modular Walls		
Bearing resistance		0.55
Sliding	• Soil shear component • Passive component	1.00 0.50
Overall stability, soil failure		**LRFD [11.6.3.7]**
Soil Nail Walls **Refer to LRFD [Table 11.5.7-1]**		

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

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 Δh1:L (in/in)
CIP semi-gravity cantilever walls	1-2	1:500
MSE walls with large precast panel facing (panel front face area > 30ft^2 and ≤ 75ft^2)and ¾" joint width.	1-2	1:200
MSE walls with small precast panel facing (panel front face area ≤ 30ft^2) and ¾" joint width.	1-2	1:100
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 [11.6.3.7].** 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.

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:

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

$\sigma_{vmin} = 0$

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:

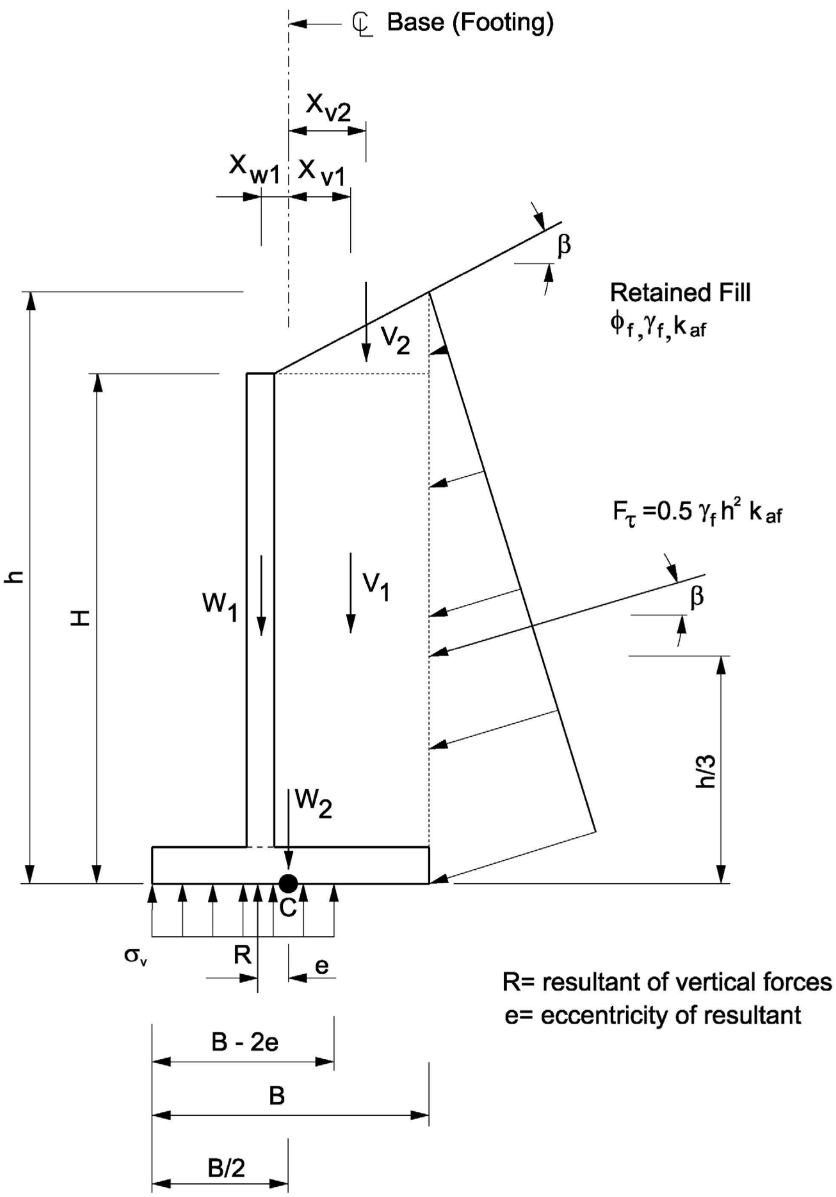
q_r = Factored bearing resistance

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

σ_v = Vertical stress

B = Base width

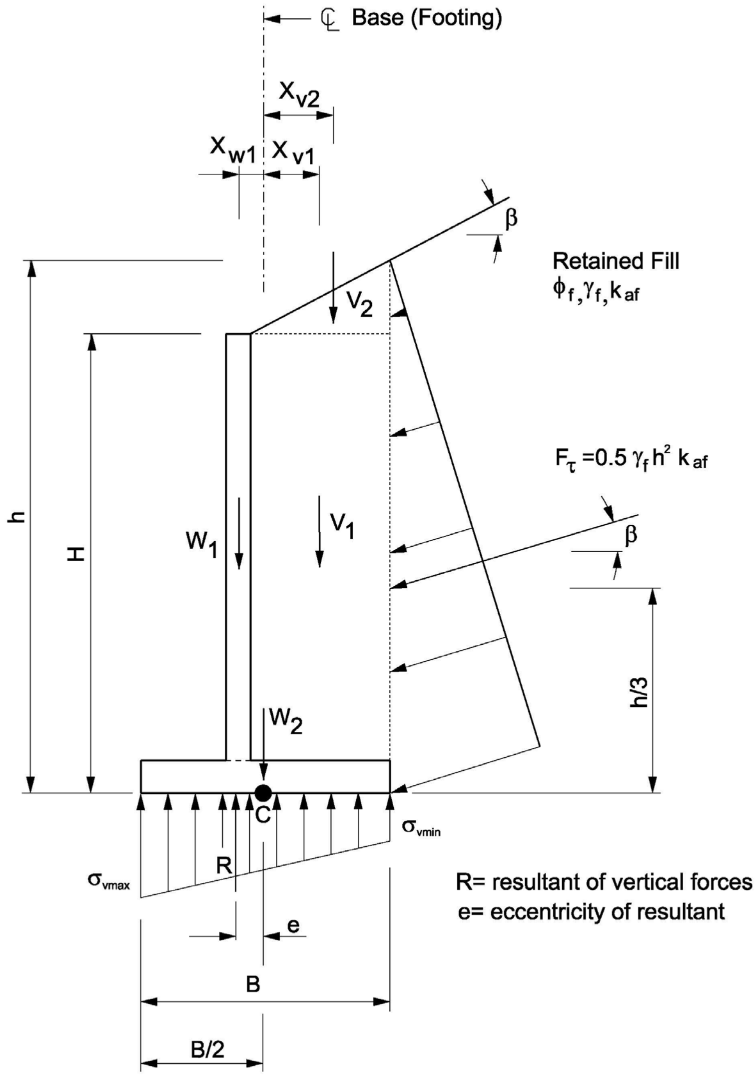
e = Eccentricity as shown in Figure 14.5-3 and Figure 14.5-4



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.

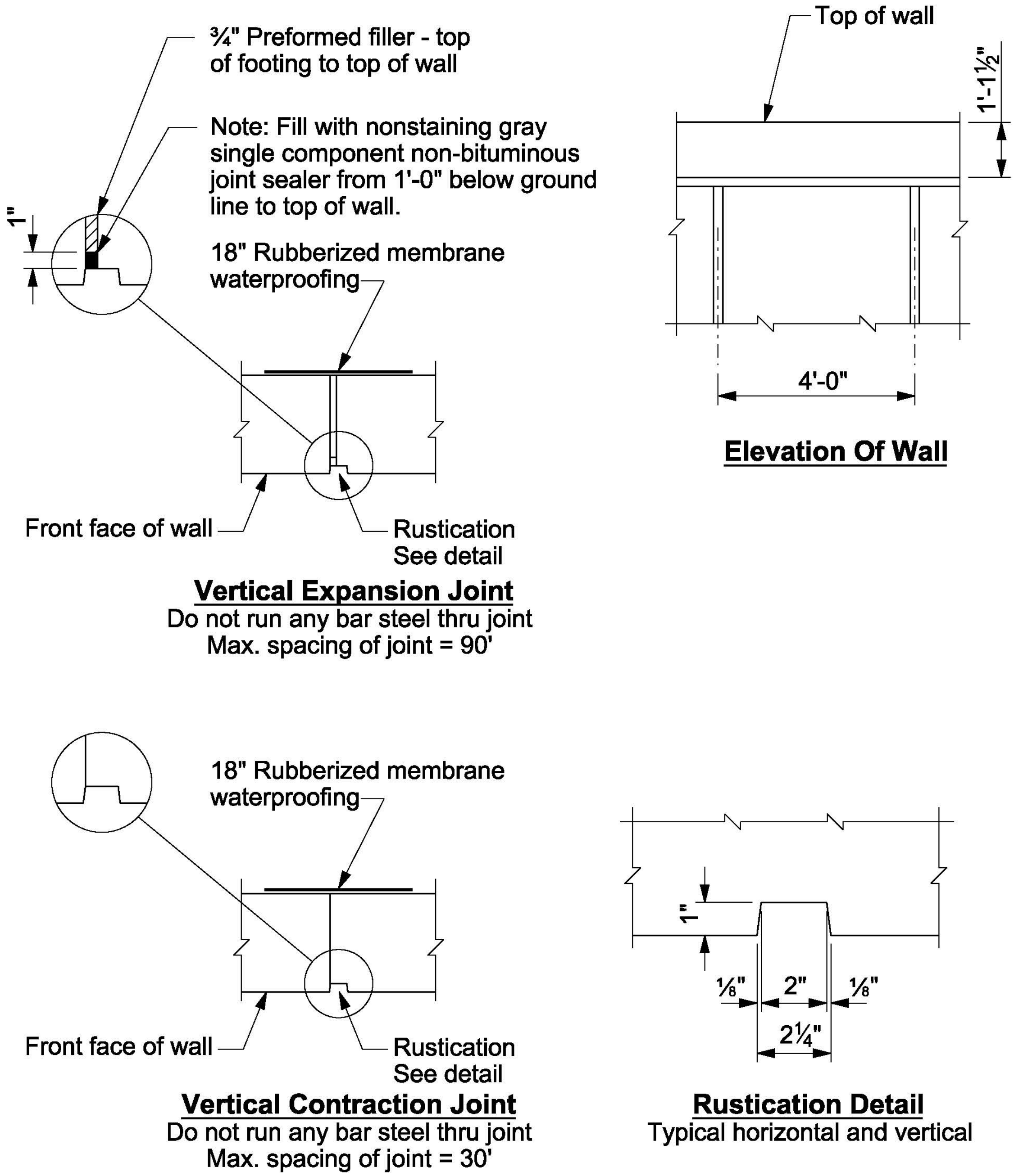


Figure 14.5-5
Retaining Wall Joint Details

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 3 feet 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

14.6 Mechanically Stabilized Earth Retaining Walls

14.6.1 General Considerations

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

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

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

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

The guidelines provided herein for MSE walls do not apply to geometrically complex MSE wall systems such as tiered walls (walls stacked on top of one another), back-to-back walls, or walls which have trapezoidal sections. Design guidelines for these cases are provided in publications FHWA-NHI-10-024, FHWA-NHI-10-025, and FHWA-NHI-24-002.

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

14.6.1.1 Usage Restrictions for MSE Walls

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

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

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

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

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

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

14.6.2 Structural Components

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

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

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

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 [11.10.5.6]**. Provision of **LRFD [11.6.3.7]** 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.

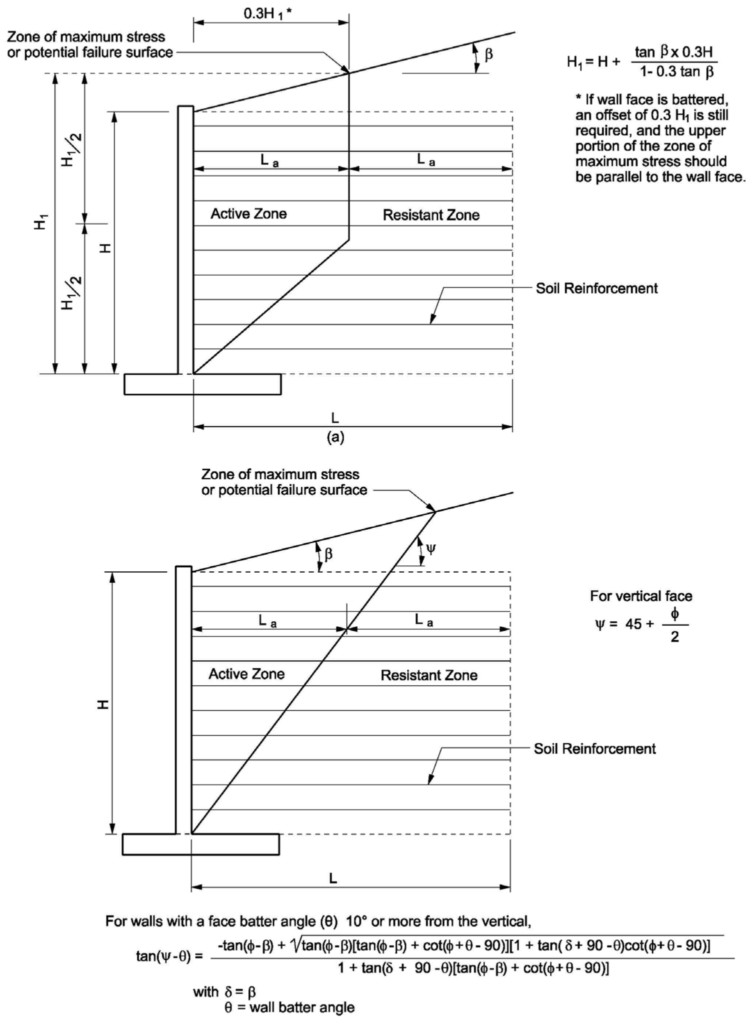


Figure 14.6-5
Location of Potential Failure Surface for Internal Stability of MSE Walls
(Source *AASHTO LRFD*)

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)

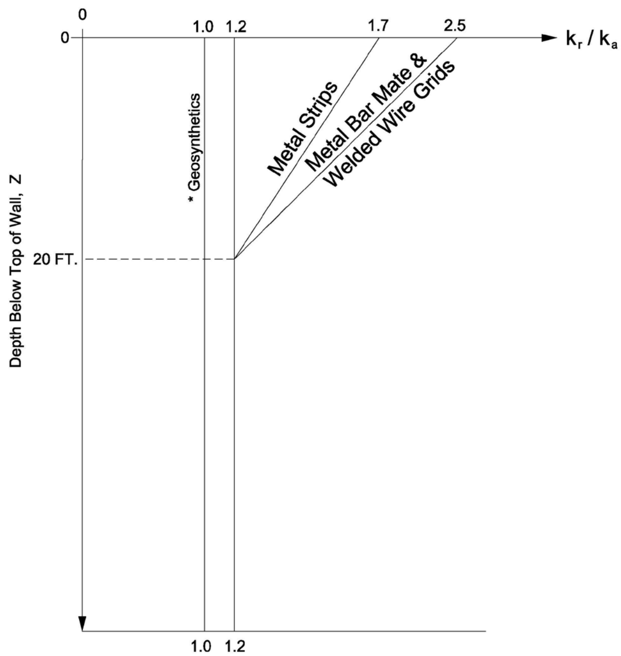
k_r = Lateral earth pressure coefficient computed using k_r/k_a

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.

Default Values for Pullout Friction Factor F4



* Does not apply to polymer strip reinforcement

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 $\leq$10 degrees, K_a is obtained using Rankine theory. For wall face with batter greater than 10 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 .

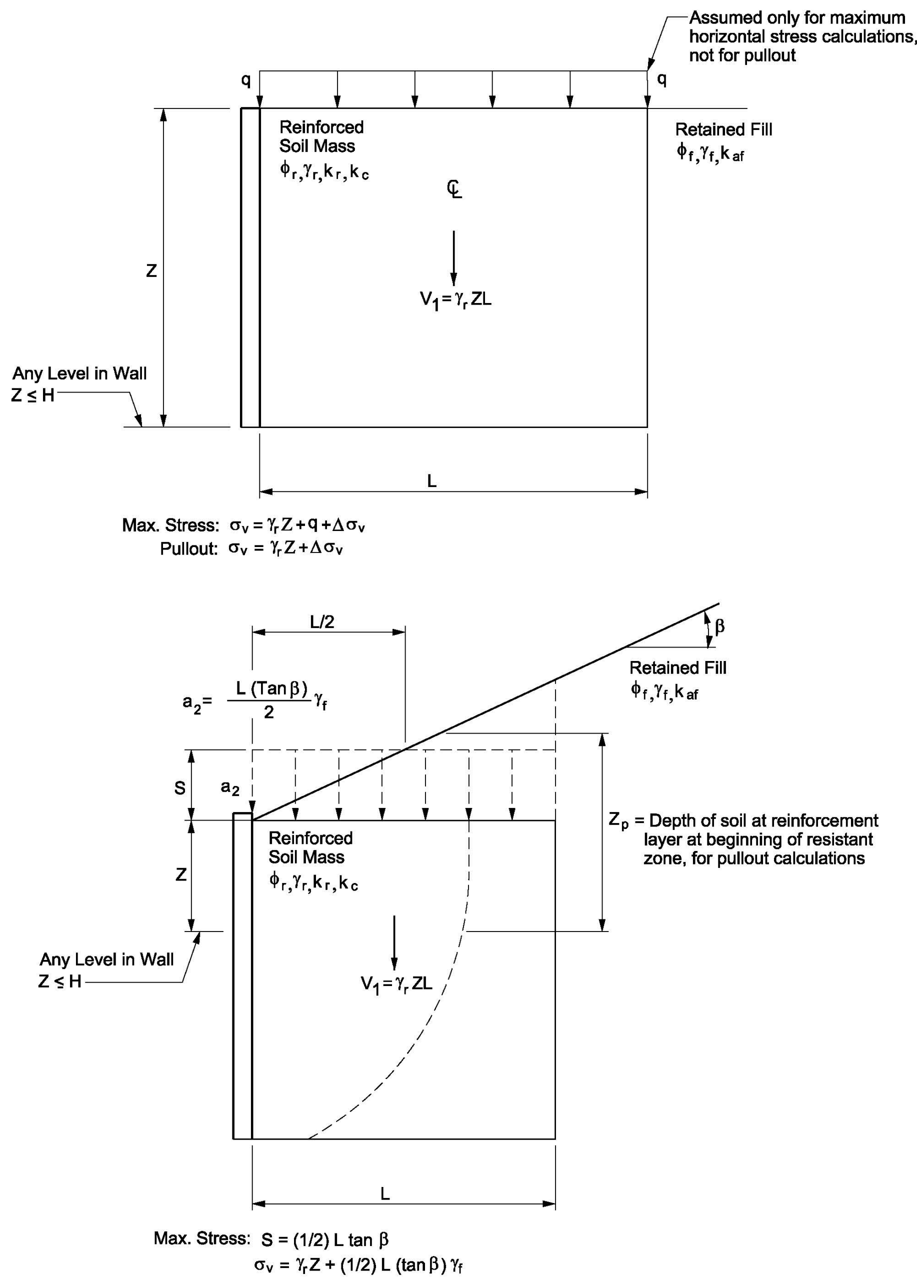


Figure 14.6-7
Calculation of Vertical Stress for Horizontal and Sloping Backslope for Internal Stability
(Source *AASHTO LRFD*)

14.6.3.8.4 Maximum Tension Force

The maximum tension load also referred as maximum 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	=	Maximum 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

$$L_e \geq \frac{\gamma_{p\text{-}EV} T_{max}}{\left(\phi F^* \alpha\, \sigma_v\, C R_c\right)}$$

Where:

L_e	=	Length of reinforcement in the resistance zone (ft)
T_{max}	=	Maximum tension load (kips/ft)
$\gamma_{p\text{-}EV}$	=	Load factor for vertical earth pressure
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 (ksf)

C = Overall reinforcement surface areas geometry factor based on the gross perimeter of the reinforcement. 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

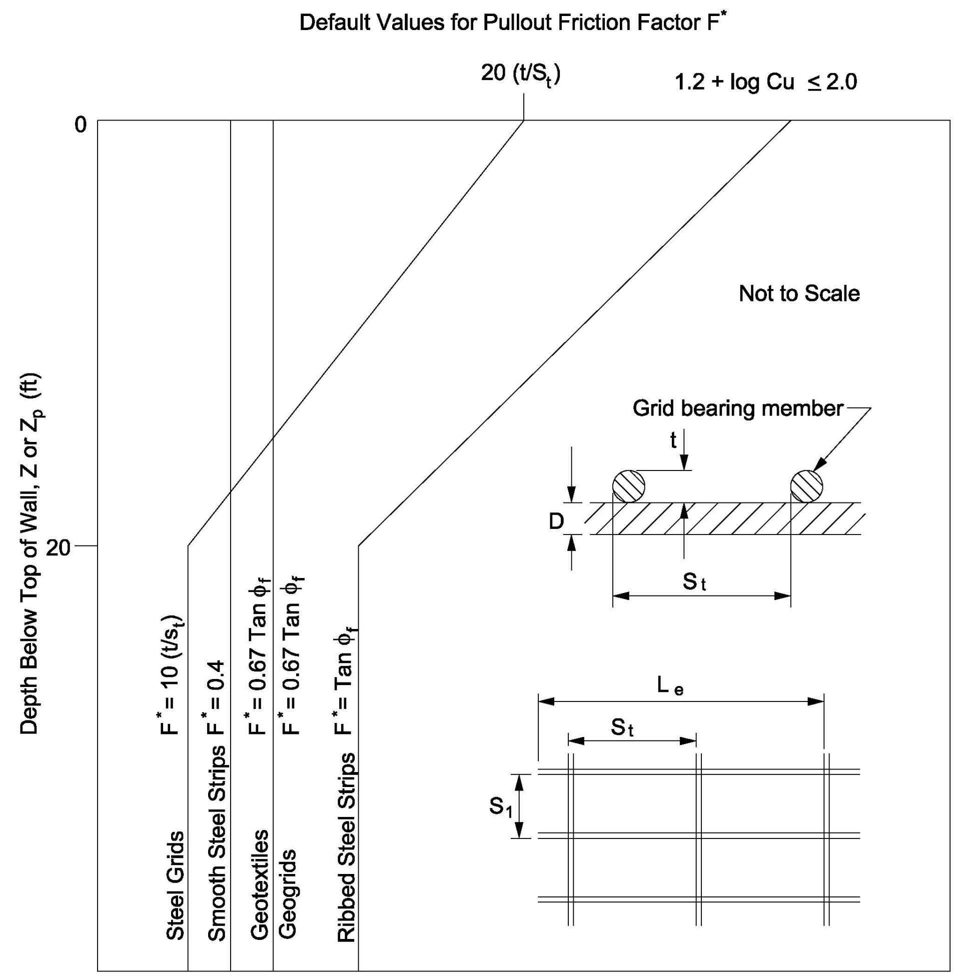


Figure 14.6-8
Default Values of F*
(Source: **LRFD [Figure 11.10.6.3.2-2]**)

14.6.3.8.6 Reinforced Design Strength

The maximum factored tensile stress ($\gamma_{p\text{-}EV} T_{MAX}$) 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:

$$\gamma_{p\text{-}EV}\, T_{MAX} \le \phi\, T_{al}\, R_C$$

Where

ϕ = Resistance factor for tensile resistance

$\gamma_{p\text{-}EV}$ = Load factor for vertical earth pressure

R_c = Reinforcement coverage ratio

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

The value for T_{MAX} is used 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	0.75
	Grid Reinforcement	0.65
Geosynthetic reinforcement	Geotextile and Geogrid Reinforcement	0.80
	Geostrip Reinforcement	0.55

Table 14.6-3
Resistance Factor for Tensile 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]**.

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 [11.6.3.7]** 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 degrees 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

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 [11.6.3.7]** 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

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18.1 Introduction

18.1.1 General

This chapter considers the following types of concrete structures:

- Flat Slab
- Haunched Slab

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

18.1.2 Limitations

For concrete slab structures over streams, this structure type is not recommended when the clearance from normal water is less than 4 feet, or less than 5 feet for spans exceeding 35 feet. This limitation accounts for a minimum falsework depth (2 to 3 feet), falsework removal and some hydraulic allowances.

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

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

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

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

WisDOT policy item:

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

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27.1 General

Bridges supported in the conventional way by abutments and piers require bearings to transfer girder reactions without overstressing the supports, ensuring that the bridge functions as intended. Bridges usually require bearings that are more elaborate than those required for building columns, girders and trusses. Bridge bearings require greater consideration in minimizing forces caused by temperature change, friction and restraint against elastic deformations. A more detailed analysis in bridge bearing design considers the following:

- Bridges are usually supported by reinforced concrete substructure units, and the magnitude of the horizontal thrust determines the size of the substructure units. The coefficient of friction on bridge bearings should be as low as possible.

- Bridge bearings must be capable of withstanding and transferring dynamic forces and the resulting vibrations without causing eventual wear and destruction of the substructure units.

- Most bridges are exposed to the elements of nature. Bridge bearings are subjected to more frequent and greater total expansion and contraction movement due to changes in temperature than those required by buildings. Since bridge bearings are exposed to the weather, they are designed as maintenance-free as possible.

WisDOT policy item:

The temperature range considered for steel girder superstructures is -30°F to 120°F. A temperature setting table for steel bearings is used for steel girders; where 45°F is the neutral temperature, resulting in a range of 120° - 45° = 75° for bearing design. Installation temperature is 60° if using laminated elastomeric bearings, resulting in a range of 60° - (-30°F) = 90°F.

The temperature range considered for prestressed concrete girder superstructures is 5°F to 85°F. Using an installation temperature of 60° for prestressed girders, the resulting range is 60° - 5° = 55° for bearing design. Use 45° as a neutral temperature for steel bearings. For prestressed girders, an additional shrinkage factor of 0.0003 ft/ft shall also be accounted for. (Do not include prestressed girder shrinkage when designing bearings for bridge rehabilitation projects). No temperature setting table is used for prestressed concrete girders.

See the Standard for Steel Expansion Bearing Details to determine bearing plate “A” sizing (steel girders) or anchor plate sizing (prestressed concrete girders). This standard also gives an example of a temperature setting table for steel bearings when used for steel girders.

WisDOT policy item:

According to **LRFD [14.4.1]**, the influence of dynamic load allowance need not be included for bearings. However, dynamic load allowance shall be included when designing bearings for bridges in Wisconsin. Apply dynamic load allowance in **LRFD [3.6.2]** to HL-93 live loads as stated in **LRFD [3.6.1.2, 3.6.1.3]** and distribute these loads, along with dead loads, to the bearings.

27.2 Bearing Types

Bridge bearings are of two general types: expansion and fixed. Bearings can be fixed in both the longitudinal and transverse directions, fixed in one direction and expansion in the other, or expansion in both directions. Expansion bearings provide for rotational movements of the girders, as well as longitudinal movement for the expansion and contraction of the bridge spans. If an expansion bearing develops a large resistance to longitudinal movement due to corrosion or other causes, this frictional force opposes the natural expansion or contraction of the span, creating a force within the span that could lead to a maintenance problem in the future. Fixed bearings act as hinges by permitting rotational movement, while at the same time preventing longitudinal movement. The function of the fixed bearing is to prevent the superstructure from moving longitudinally off of the substructure units. Both expansion and fixed bearings transfer lateral forces, as described in **LRFD [Section 3]**, from the superstructure to the substructure units. Both bearing types are set parallel to the direction of structural movement; bearings are not set parallel to flared girders.

When deciding which bearings will be fixed and which will be expansion on a bridge, several guidelines are commonly considered:

- The bearing layout for a bridge must be developed as a consistent system. Vertical movements are resisted by all bearings, longitudinal horizontal movements are resisted by fixed bearings and facilitated in expansion bearings, and rotations are generally allowed to occur as freely as possible.

- For maintenance purposes, it is generally desirable to minimize the number of deck joints on a bridge, which can in turn affect the bearing layout.

- The bearing layout must facilitate the anticipated thermal movements, primarily in the longitudinal direction, but also in the transverse direction for wide bridges.

- It is generally desirable for the superstructure to expand in the uphill direction, wherever possible.

- If more than one substructure unit is fixed within a single superstructure unit, then forces will be induced into the fixed substructure units and must be considered during design. If only one pier is fixed, unbalanced friction forces from expansion bearings will induce force into the fixed pier.

- For curved bridges, the bearing layout can induce additional stresses into the superstructure, which must be considered during design.

- Forces are distributed to the bearings based on the superstructure analysis.

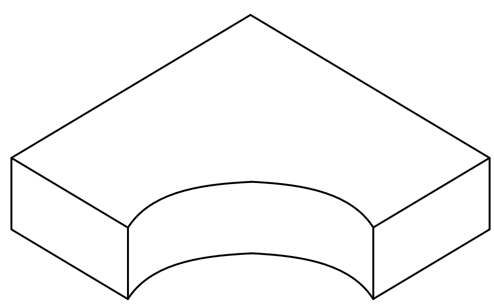
A valuable tool for selecting bearing types is presented in **LRFD [Table 14.6.2-1]**, in which the suitability of various bearing types is presented in terms of movement, rotation and resistance to loads. In general, it is best to use a fixed or semi-expansion bearing utilizing an unreinforced elastomeric bearing pad whenever possible, provided adverse effects such as excessive force transfer to the substructure does not occur. Where a fixed bearing is required with greater rotational capacity, steel fixed bearings can be utilized. Laminated elastomeric bearings are the preferred choice for expansion bearings. When such expansion bearings fail to meet project requirements, steel Type “A-T” expansion bearings should be used. For curved and/or highly skewed bridges, consideration should be given to the use of pot bearings.

27.2.1 Elastomeric Bearings

Elastomeric bearings are commonly used on small to moderate sized bridges. Elastomeric bearings are either fabricated as plain bearing pads (consisting of elastomer only) or as laminated (steel reinforced) bearings (consisting of alternate layers of steel reinforcement and elastomer bonded together during vulcanization). A sample plain elastomeric bearing pad is illustrated in Figure 27.2-1, and a sample laminated (steel reinforced) elastomeric bearing is illustrated in Figure 27.2-2.

These bearings are designed to transmit loads and accommodate movements between a bridge and its supporting structure. Plain elastomeric bearing pads can be used for small bridges, in which the vertical loads, translations and rotations are relatively small. Laminated (steel reinforced) elastomeric bearing pads are often used for larger bridges with more sizable vertical loads, translations and rotations. Performance information indicates that elastomeric bearings are functional and reliable when designed within the structural limits of the material. See **LRFD [Section 14]**, *AASHTO LRFD Bridge Construction Specifications*, *Section 18*, and *AASHTO M251* for design and construction requirements of elastomeric bearings.

<u>WisDOT policy item:</u>
WisDOT currently uses plain or laminated (steel reinforced) elastomeric bearings which are rectangular in shape. No other shapes or configurations are used for elastomeric bearings in Wisconsin.



<u>Figure 27.2-1</u>
Plain Elastomeric Bearing

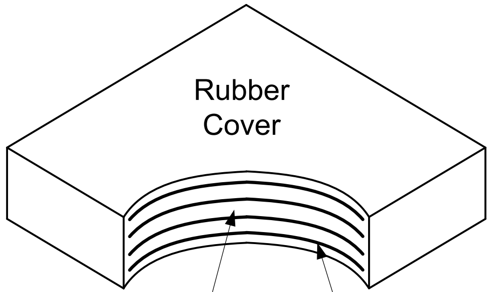


Figure 27.2-2
Laminated (Steel Reinforced) Elastomeric Bearing

AASHTO LRFD does not permit tapered elastomer layers in reinforced bearings **LRFD [14.7.5.1]**. Laminated (steel reinforced) bearings must be placed on a level surface; otherwise gravity loads will produce shear strain in the bearing due to inclined forces. The angle between the alignment of the underside of the girder (due to the slope of the grade line, camber and dead load rotation) and a horizontal line must not exceed 0.01 radians, as per **LRFD [14.8.2]**. If the angle is greater than 0.01 radians or if the rotation multiplied by the top plate length is 1/8" or more, the 1 1/2" top steel plate must be tapered to provide a level load surface along the bottom of this plate under these conditions. The tapered plate will have a minimum thickness of 1 1/2" per *AASHTO LRFD Bridge Construction Specifications, Section 18 (Article 18.2.6)*.

Plain and laminated (steel reinforced) elastomeric bearings can be designed by Method A as outlined in **LRFD [14.7.6]** and NCHRP-248 or by Method B as shown in **LRFD [14.7.5]** and NCHRP-298.

WisDOT policy item:

WisDOT uses Method A, as described in **LRFD [14.7.6]**, for elastomeric bearing design.

Method A results in a bearing with a lower capacity than a bearing designed using Method B. However, the increased capacity resulting from the use of Method B requires additional testing and quality control, and WisDOT currently does not have a system in place to verify these requirements.

For several years, plain elastomeric bearing pads have performed well on prestressed concrete girder structures. Refer to the Standard for *Bearing Pad Details for Prestressed Concrete Girders* for details. Prestressed concrete girders using this detail are fixed into the concrete diaphragms at the supports, and the girders are set on 1/2" thick plain elastomeric bearing pads. Laminated (steel reinforced) bearing details and steel plate and elastomer thicknesses are given on the Standard for *Elastomeric Bearings for Prestressed Concrete Girders.*

The design of an elastomeric bearing generally involves the following **Steps** :

1. Obtain required design input **LRFD [14.4, 14.6 & 14.7]**

   The required design input for the design of an elastomeric bearing at the service limit state is dead load, live load plus dynamic load allowance, minimum vertical force due to permanent load, and design translation. The required design input at the strength limit state is shear force. Other required design input is expansion length, girder or beam bottom flange width, minimum grade of elastomer **LRFD [Table 14.7.5.2-1]**, and temperature zone. Two temperature zones are shown for Wisconsin in **LRFD [Figure 14.7.5.2-1]**, zones C and D. **WisDOT policy** is for all elastomeric bearings to meet Zone D requirements.

2. Select a feasible bearing type – plain or laminated (steel reinforced)

3. Select preliminary bearing properties **LRFD [14.7.6.2]**

   The preliminary bearing properties can be obtained from **LRFD [14.7.6.2]** or from past experience. The preliminary bearing properties include elastomer cover thickness, elastomer internal layer thickness, elastomer hardness, elastomer shear modulus and elastomer creep deflection **LRFD [Table 14.7.6.2-1]**, pad length, pad width, number of steel reinforcement layers, steel reinforcement thickness, steel reinforcement yield strength and steel reinforcement constant-amplitude fatigue threshold **LRFD [Table 6.6.1.2.3-1]**. **WisDOT** uses the following properties:

   - Elastomer cover thickness = 1/4"
   - Elastomer internal layer thickness = 1/2"
   - Elastomer hardness: Durometer 60 +/- 5
   - Elastomer shear modulus (G): 0.1125 ksi < G < 0.165 ksi
   - Elastomer creep deflection @ 25 years divided by instantaneous deflection = 0.30
   - Steel reinforcement thickness = 1/8"
   - Steel reinforcement yield strength = 36 ksi or 50 ksi
   - Steel reinforcement constant-amplitude fatigue threshold = 24 ksi

   However, not all of these properties are needed for a plain elastomeric bearing design.

4. Check shear deformation **LRFD [14.7.6.3.4, 14.7.5.3.2]**

   Shear deformation, Δ_S, is the sum of deformation from thermal effects, Δ_{ST}, as well as creep and shrinkage effects, $\Delta_{Scr/sh}$; ($\Delta_S = \Delta_{ST} + \Delta_{Scr/sh}$).

   $$\Delta_{ST} = (\text{Expansion length})(\Delta_T)(\alpha)$$

   Where:

Δ_T = Change in temperature (see 27.1 – **WisDOT policy**) (degrees)

α = Coefficient of thermal expansion ; **LRFD [5.4.2.2, 6.4.1]**
= 6×10^{-6} / °F for concrete, 6.5×10^{-6} / °F for steel

Shear deformation due to creep and shrinkage effects, $\Delta_{Scr/sh}$, should be added to Δ_{ST} for prestressed concrete girder structures. The creep and shrinkage coefficient is 0.0003 ft/ft – per (27.1 – **WisDOT policy**). The value of $\Delta_{Scr/sh}$ is computed as follows:

$$\Delta_{Scr/sh} = (\text{Expansion length})(0.0003\ \text{ft/ft})$$

LRFD [14.7.6.3.4] provides shear deformation limits to help prevent rollover at the edges and delamination. The shear deformation, Δ_S, can be checked as specified in **LRFD [14.7.6.3.4]** and by the following equation:

$$h_{rt} \geq 2\,\Delta_S$$

Where:

h_{rt} = Total elastomer thickness for steel reinforced bearing or elastomer thickness for plain pad (inches)

Δ_S = Maximum total shear deformation of the bearing at the service limit state (inches)

5. Check compressive stress **LRFD [14.7.6.3.2]**

The compressive stress, σ_S, at the service limit state can be checked as specified in **LRFD [14.7.6.3.2]** and by the following equations:

$\sigma_s \leq 0.80\ ksi$ and $\sigma_s \leq 1.00GS$ - for plain elastomeric pads

$\sigma_s \leq 1.25\ ksi$ and $\sigma_s \leq 1.25GS$ - for laminated (steel reinforced) elastomeric pads

Where:

σ_s = Average compressive stress due to total service load (ksi)

G = Shear modulus of elastomer (ksi)

S = Shape factor for plain pad or internal layer of steel reinforced brg.

LRFD [14.7.6.3.2] states that the stress limits may be increased by 10 percent where shear deformation is prevented, but this is not considered applicable to **WisDOT** bearings.

The shape factor for individual elastomer layers is the plan area divided by the area of the perimeter free to bulge. For laminated (steel reinforced) elastomeric bearings, the following requirements must be satisfied before calculating the shape factor:

- All internal layers of elastomer must be the same thickness **LRFD [14.7.6.1]**.
- The thickness of the cover layers cannot exceed 70 percent of the thickness of the internal layers, or 5/16”, whichever is greater **LRFD [14.7.6.1]**.

The shape factor, S_i, for rectangular bearings without holes can be determined as specified in **LRFD [14.7.5.1, 14.7.6.1]** and by the following equation:

$$S_i = \frac{LW}{2h_{ri}(L+W)}$$

Where:

S_i	=	Shape factor for internal layer in the steel reinforced bearing
h_{ri}	=	Thickness of internal elastomer layer in steel reinf bearing (inches)
L	=	Length of a rectangular elastomeric bearing (parallel to longitudinal bridge axis) (inches)
W	=	Width of the bearing in the transverse direction (inches)

Check $S_i^2 / n < 20$ per **LRFD [C14.7.6.1]** and recalculate if S_i changes in stability check.

6. Check stability **LRFD [14.7.6.3.6]**

For stability, the total thickness of the rectangular pad must not exceed one-third of the pad length or one-third of the pad width as specified in **LRFD [14.7.6.3.6]**, or expressed mathematically:

$$H \le \frac{L}{3} \text{ and } H \le \frac{W}{3}$$

Where:

H	=	Total thickness of the elastomeric bearing (excluding top plate) (inches)
L	=	Length of a rectangular elastomeric bearing (parallel to longitudinal bridge axis) (inches)
W	=	Width of the bearing in the transverse direction (inches)

7. Check compressive deflection **LRFD [14.7.5.3.6, 14.7.6.3.3]**

The compressive deflection, δ, of the bearing shall be limited to ensure the serviceability of the deck joints, seals and other components of the bridge. Deflections of elastomeric bearings due to total load and to live load alone should be considered separately. Relative deflections across joints must be restricted so that a step doesn't occur at a deck joint. **LRFD [C14.7.5.3.6]** recommends that a maximum relative live load deflection across a joint be limited to 1/8".

WisDOT policy item:

WisDOT uses a live load + creep deflection limit of 1/8" for elastomeric bearing design.

Laminated (steel reinforced) elastomeric bearings have a nonlinear load deflection curve in compression. In the absence of information specific to the particular elastomer to be used, **LRFD [Figure C14.7.6.3.3-1]** may be used as a guide. Creep effects should be determined from information specific to the elastomeric compound used. Use the material properties given in this section. The compressive deflection, δ, can be determined as specified in **LRFD [14.7.5.3.6, 14.7.6.3.3]** and by the following equation:

$$\delta = \sum \varepsilon_i \, h_{ri}$$

Where:

δ = compressive deflection due to service loads (inches)

ε_i = compressive strain in the i^{th} elastomer layer in a steel reinforced bearing

h_{ri} = Thickness of i^{th} elastomer layer in a steel reinforced bearing (inches)

Based on **LRFD [14.7.6.3.3]**, the compressive deflection of a plain elastomeric pad or an internal layer of a laminated (steel reinforced) elastomeric bearing at the service limit state (incl. dynamic load allowance per **WisDOT policy item** in 27.1) shall not exceed $0.09h_{ri}$.

8. Check anchorage

WisDOT exception to AASHTO:

Design anchorage for laminated elastomeric bearings if the unfactored dead load stress is less than 200 psi. This is an exception to **LRFD [14.8.3]** based on past practice and good performance of existing bearings.

The factored force due to the shear deformation of an elastomeric element shall be taken as specified in **LRFD [14.6.3.1]** by the following equation:

$H_{bu} = GA\ (\Delta_u / h_{rt})$

Where:

H_{bu}	=	Lateral force from applicable strength load combinations in **LRFD [Table 3.4.1-1]** (kips)
G	=	Shear modulus of the elastomer (ksi)
A	=	Plan area of elastomeric element or bearing (inches^2)
Δ_u	=	Factored shear deformation (inches)
h_{rt}	=	Total elastomer thickness (inches)

9. Check reinforcement **LRFD [14.7.5.3.5, 14.7.6.3.7]**

Reinforcing steel plates increase compressive and rotational stiffness, while maintaining flexibility in shear. The reinforcement must have adequate capacity to handle the tensile stresses produced in the plates as they counter the lateral bulging of the elastomer layers due to compression. These tensile stresses increase with compressive load. The reinforcement thickness must also satisfy the requirements of the *AASHTO LRFD Bridge Construction Specifications*. The reinforcing steel plates can be checked as specified in **LRFD [Equation 14.7.5.3.5-1,2]**:

$$h_s \geq \frac{3\ h_{max}\ \sigma_s}{F_y} \quad \text{- for service limit state}$$

$$h_s \geq \frac{2.0\ h_{max}\ \sigma_L}{\Delta F_{TH}} \quad \text{- for fatigue limit state}$$

Where:

h_s	=	Thickness of the steel reinforcement (inches);(min. thick. = 0.0747”)
h_{max}	=	Thickness of the thickest elastomer layer in elastomeric bearing (inches)
σ_s	=	Average compressive stress due to total service load (ksi)
F_y	=	Yield strength of steel reinforcement (ksi)
σ_L	=	Average compressive stress due to live load (ksi)
ΔF_{TH}	=	Constant amplitude fatigue threshold for Category A as specified in **LRFD [Table 6.6.1.2.3-1]** (ksi)

If holes exist in the reinforcement, the minimum thickness shall be increased by a factor equal to twice the gross width divided by the net width.

10. Rotation **LRFD [14.7.6.3.5, C14.7.6.1]**

WisDOT exception to AASHTO:

Lateral rotation about the longitudinal axis of the bearing shall not be considered for straight girders.

WisDOT policy item:

Per **LRFD [14.8.2]**, a tapered plate shall be used if the inclination of the underside of the girder to the horizontal exceeds 0.01 radians. Additionally, if the rotation multiplied by the plate length is 1/8 inch or more, taper the plate.

27.2.2 Steel Bearings

For fixed bearings, a rocker plate attached to the girder is set on a masonry plate which transfers the girder reaction to the substructure unit. The masonry plate is attached to the substructure unit with anchor bolts. Pintles set into the masonry plate prevent the rocker from sliding off the masonry plate while allowing rotation to occur. This bearing is represented on the Standard for *Fixed Bearing Details Type "A" - Steel Girders*.

For expansion bearings, two additional plates are utilized, a stainless steel top plate and a Teflon plate allowing expansion and contraction to occur, but not in the transverse direction. This bearing is shown on the Standard for *Stainless Steel - TFE Expansion Bearing Details Type "A-T".*

Type "B" rocker bearings have been used for reactions greater than 400 kips and having a requirement for smaller longitudinal forces on the substructure unit. However, in the future, **WisDOT** plans to eliminate rocker bearings for new bridges and utilize pot bearings.

Pot and disc bearings are commonly used for moderate to large bridges. They are generally used for applications requiring a multi-directional rotational capacity and a medium to large range of load.

Hold down devices are additional details added to the Type "A-T" bearings for situations where live load can cause uplift at the abutment end of a girder. Ideally, proper span configurations would eliminate the need for hold down devices as they have proven to be a maintenance problem.

Since strength is not the governing criteria, anchor bolts are designed with Grade 36 steel for all steel bearings.

27.2.2.1 Type "A" Fixed Bearings

Type "A" Fixed Bearings prevent translation both transversely and longitudinally while allowing rotation in the longitudinal direction. This bearing is represented on the Standard for *Fixed Bearing Details Type "A" - Steel Girders*. An advantage of this bearing type is that it is very low maintenance. See 27.2.2.2 Type "A-T" Expansion Bearings for design information.

27.2.2.2 Type "A-T" Expansion Bearings

Type "A-T" Expansion bearings are designed to translate by sliding an unfilled polytetrafluoroethylene (PTFE or TFE) surface across a smooth, hard mating surface of stainless steel. Expansion bearings of Teflon are not used without provision for rotation. A rocker plate is provided to facilitate rotation due to live load deflection or change of camber. The Teflon sliding surface is bonded to a rigid back-up material capable of resisting horizontal shear and bending stresses to which the sliding surfaces may be subjected.

Design requirements for TFE bearing surfaces are given in **LRFD [14.7.2]**. Stainless steel-TFE expansion bearing details are given on the Standard for *Stainless Steel – TFE Expansion Bearing Details Type "A-T."*

Friction values are given in the **LRFD [14.7.2.5]**; they vary with loading and temperature. It is permissible to use 0.10 for a maximum friction value and 0.06 for a minimum value when determining unbalanced friction forces.

The design of type "A-T" bearings is relatively simple. The first consideration is the rocker plate length which is proportional to the contact stress based on a radius of 24" using Grade 50W steel. The rocker plate thickness is determined from a minimum of 1 1/2" to a maximum computed from the moment by assuming one-half the bearing reaction value (N/2) acting at a lever arm of one-fourth the width of the Teflon coated plate (W/4) over the length of the rocker plate. The Teflon coated plate is designed with a minimum width of 7" and the allowable stress as specified in **LRFD [14.7.2.4]** on the gross area; in many cases this controls the capacity of the expansion bearings as given in the Standard for *Stainless Steel – TFE Expansion Bearing Details Type "A-T."*

The design of the masonry plate is based on a maximum allowable bearing stress as specified in **LRFD [14.8.1]**. The masonry plate thickness is determined from the maximum bending moments about the x-or y-axis using a uniform pressure distribution.

In lieu of designing specific bearings, the designer may use Service I limit state loading, including dynamic load allowance, and Standards for *Fixed Bearing Details Type "A" – Steel Girders, Stainless Steel – TFE Expansion Bearing Details Type "A-T"* and *Steel Bearings for Prestressed Concrete Girders* to select the appropriate bearing.

27.2.2.3 High-Load Multi-Rotational Bearings

High-Load Multi-Rotational bearings, such as pot or disc bearings, are commonly used for moderate to large bridges. They are generally used for curved and/or highly skewed bridge applications requiring a multi-directional rotational capacity and a medium to large range of load.

27.4 Design Example

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E27-1 DESIGN EXAMPLE - STEEL REINFORCED ELASTOMERIC BEARING

This design example is for a 3-span prestressed girder structure. The piers are fixed supports and the abutments accommodate expansion.
(Example is current through LRFD Tenth Edition - 2024)

E27-1.1 Design Data

Bearing location: Abutment (Type A3)
Girder type: 72W

$L_{exp} := 220$ Expansion length, ft

$b_f := 2.5$ Bottom flange width, ft

$DL_{serv} := 167$ Service I limit state dead load, kips

$DL_{ws} := 23$ Service I limit state future wearing surface dead load, kips

$LL_{serv} := 62$ Service I limit state live load, kips (incl. dynamic load allowance)

$h_{rcover} := 0.25$ Elastomer cover thickness, in (see 27.2.1-Step 3)

$h_s := 0.125$ Steel reinforcement thickness, in (see 27.2.1-Step 3)

$F_y := 36$ Min. yield strength of the steel reinf., ksi (see 27.2.1-Step 3)

Temperature Zone:	D (Use for Entire State)	**LRFD [Fig. 14.7.5.2-1]**
Minimum Grade of Elastomer:	4	**LRFD [Table 14.7.5.2-1]**
Elastic Hardness:	Durometer 60 +/- 5	(used 55 for design)
Shear Modulus (G):	0.1125 ksi < G <0.165 ksi	**LRFD [Table 14.7.6.2-1]**
Creep Deflection @ 25 Years		
divided by instantaneous deflection:	0.3	**LRFD [Table 14.7.6.2-1]**

E27-1.2 Design Method

Use Design Method A **LRFD [14.7.6]**
Method A results in a bearing with a lower capacity than a bearing designed using Method B. However the increased capacity resulting from the use of Method B requires additional testing and quality control.

E27-1.3 Dynamic Load Allowance

The influence of impact need not be included for bearings **LRFD [14.4.1]**; however, dynamic load allowance will be included to follow a **WisDOT policy item** (see 27.1).

E27-1.4 Shear

The maximum shear deformation (Δ_s) of the pad shall be taken as the maximum horizontal superstructure displacement, reduced to account for the pier flexibility. **LRFD [14.7.6.3.4]**

$h_{rt} \geq 2 \cdot \Delta_s$ Total elastomer thickness **LRFD [Equation 14.7.6.3.4-1]**

Temperature range: T_{low} , T_{high} , $T_{install}$, S_{crsh} values below are in 27.1-**WisDOT policy item**

$T_{low} := 5$ Minimum temperature, °F

$T_{high} := 85$ Maximum temperature, °F

$\gamma_{TU} := 1.2$ Load factor for deformation **LRFD [Table 3.4.1-6]**

$T_{install} := 60$ Installation temperature, °F

$\alpha_c := 0.000006$ Coefficient of thermal expansion of concrete, ft/ft/ °F

$S_{crsh} := 0.0003$ Coefficient of creep and shrinkage of concrete, ft/ft

$\Delta_T := T_{install} - T_{low}$ Temperature range $\Delta_T = 55$ °F

Maximum total shear deformation of the elastomer at Service Limit State

$\Delta_s := L_{exp} \cdot \alpha_c \cdot \Delta_T \cdot 12 + L_{exp} \cdot S_{crsh} \cdot 12$ $\Delta_s = 1.663$ in

Required total elastomer thickness

$H_{rt} \geq 2 \cdot \gamma_{TU} \cdot \Delta_s$ $H_{rt} = 3.992$ in

Select elastomer internal layer thickness (see 27.2.1-Step 3)

$h_{ri} := 0.5$ in

Look at elastomer internal and cover layer relationship **LRFD [14.7.6.1)**

$h_{rcover} \leq 0.7(h_{ri})$ or 5/16" whichever is greater ; $h_{rcover1} := 0.7 \cdot h_{ri}$ $h_{rcover2} := 0.3125$ in

$h_{rcover\_max} = 0.35$ in Check if $h_{rcover} \leq h_{rcover\_max}$ check = "OK"

Determine the number of internal elastomer layers:

$$n := \frac{H_{rt} - 2 \cdot h_{rcover}}{h_{ri}}$$

Note:

$h_{rcover} = 0.25$ in

$n = 6.98$ layers

Use: $n = 7$ internal layers

Total elastomer thickness:

$h_{rt} := 2 \cdot h_{rcover} + n \cdot h_{ri}$ $h_{rt} = 4.0$ in

Total height of reinforced elastomeric pad:

$H := h_{rt} + (n + 1) \cdot h_s$ $H = 5.000$ in

E27-1.5 Compressive Stress

$\sigma_{s\_all} \le 1.25$ ksi and $\sigma_{s\_all} \le 1.25 \cdot G \cdot S_i$ ksi **LRFD [14.7.6.3.2]**

$edge := 3$ in Transverse distance from the edge of the flange to edge of bearing

$W := 12 \cdot b_f - 2 \cdot edge$ Transverse bearing dimension $W = 24$ in

$L \ge \dfrac{DL_{serv} + LL_{serv}}{W \cdot \sigma_{s\_all}}$ Since $\sigma_{s\_all} \le \dfrac{DL_{serv} + LL_{serv}}{L \cdot W}$

$\sigma_{s\_all} := 1.25$ ksi (Now select "L" based on 1st stress limit)

$L := \dfrac{DL_{serv} + LL_{serv}}{W \cdot \sigma_{s\_all}}$ Longitudinal bearing dimension $L = 7.633$ in

$increment := 5$ in <== Rounding increment

$L = 10$ in

(Use a 1 inch minimum rounding increment for design. For this example, the rounding increment is used to increase L dimension to satisfy subsequent stress checks, etc.)

Determine shape factor for internal layer **LRFD [14.7.5.1, 14.7.6.1]**

$S_i := \dfrac{L \cdot W}{2 \cdot h_{ri} \cdot (L + W)}$ **LRFD [Equation 14.7.5.1-1]** $S_i = 7.059$

$G := 0.1125$ ksi $0.1125 \text{ ksi} < G < 0.165 \text{ ksi}$ Shear Modulus (min.)

(Verify that **LRFD** is satisfied for a full range of G values. The minimum G value is used here. See also E27-1.8)

$1.25 \cdot G \cdot S_i = 0.993$ ksi (Now check 2nd stress limit)

$\sigma_s := \dfrac{DL_{serv} + LL_{serv}}{L \cdot W}$ Avg. compressive stress due to Total Load $\sigma_s = 0.954$ ksi

$\sigma_s =$ "< 1.25GS, OK"

Check **LRFD [C14.7.6.1]**: $S_i^2 / n < 20$ (for rectangular shape with $n \geq 3$)

$S_i^2 / n = (7.059)^2 / 8 = 6.3 < 20$ "OK"

where n = (7 inter. layers + 1/2 (2 exter. layers)) = 8

E27-1.6 Stability

$H \leq \frac{L}{3}$ and $H \leq \frac{W}{3}$ **LRFD [14.7.6.3.6]**

$H = 5.000$ in Total height of reinforced elastomeric pad (from E27-1.4)

Bearing length check:

$L_{min} := 3 \cdot H$ $L_{min} = 15$ in

(from E27-1.5) $L = 10$ in

Use the larger value: $L = 15$ in

Bearing width check:

$W_{min} := 3 \cdot H$ $W_{min} = 15$ in

(from E27-1.5) $W = 24$ in

Use the larger value: $W = 24$ in

Revise shape factor and recheck compressive stress for internal layer:

$h_{ri} = 0.5$ in Elastomer internal layer thickness

$G = 0.1125$ ksi Shear Modulus (min.)

$S_i := \frac{L \cdot W}{2 \cdot h_{ri} \cdot (L + W)}$ $S_i = 9.231$

(Now check 2nd stress limit) $1.25 \cdot G \cdot S_i = 1.298$ ksi

$\sigma_s := \frac{DL_{serv} + LL_{serv}}{L \cdot W}$ Avg. compressive stress due to Total Load $\sigma_s = 0.636$ ksi

$\sigma_s =$ "< 1.25GSi, OK"

Check **LRFD [C14.7.6.1]**: $S_i^2 / n < 20$ (for rectangular shape with $n \geq 3$)

$S_i^2 / n = (9.231)^2 / 8 = 10.7 < 20$ "OK"

where n = (7 inter. layers + 1/2 (2 exter. layers)) = 8

E27-1.7 Compressive Deflection

LRFD [14.7.6.3.3, 14.7.5.3.6] (Service Limit State)

Average vertical compressive stress:

Average compressive stress due to total load

$\sigma_s = 0.636$ ksi (see E27-1.6)

Average compressive stress due to live load

$\sigma_L := \dfrac{LL_{serv}}{L \cdot W}$ $\sigma_L = 0.172$ ksi

Average compressive stress due to dead load

$\sigma_D := \dfrac{DL_{serv}}{L \cdot W}$ $\sigma_D = 0.464$ ksi

Use **LRFD [Figure C14.7.6.3.3-1]** to estimate the compressive strain in the interior and cover layers. Average the values from the 50 Durometer and 60 Durometer curves to obtain values for 55 Durometer bearings.

Calculate a shape factor to estimate the compressive strain in the cover layer:

$S_{cover} := \dfrac{L \cdot W}{2 \cdot h_{rcover} \cdot (L + W)}$ $S_{cover} = 18.462$

LAYER	LOAD	S	STRESS (ksi)	50 DUROMETER STRAIN	60 DUROMETER STRAIN	AVERAGE STRAIN
INTERNAL	DEAD LOAD	9.231	0.464	2.3%	2.1%	2.2%
	TOTAL LOAD	9.231	0.636	3.1%	2.7%	2.9%
COVER	DEAD LOAD	18.462	0.464	1.8%	1.5%	1.7%
	TOTAL LOAD	18.462	0.636	2.2%	1.9%	2.1%

Initial compressive deflection of n-internal layers and 2 cover layers under total load:

$\varepsilon_{int} = 0.029$ Compressive strain in the interior layer

$\varepsilon_{cover} = 0.021$ Compressive strain in the cover layer

$n = 7$ # of internal layers

$h_{ri} = 0.5$ in Internal layer thickness

$h_{rcover} = 0.25$ in Cover layer thickness

$\delta := n \cdot h_{ri} \cdot \varepsilon_{int} + 2 \cdot h_{rcover} \cdot \varepsilon_{cover}$ Modification of **LRFD [Equation 14.7.5.3.6-1]**

Total load defl. $\delta = 0.112$ in

Initial compressive deflection under dead load:

$\varepsilon_{intDL} = 0.022$

$\varepsilon_{coverDL} = 0.017$

$\delta_{DL} := n \cdot h_{ri} \cdot \varepsilon_{intDL} + 2 \cdot h_{rcover} \cdot \varepsilon_{coverDL}$ Dead load defl. $\delta_{DL} = 0.086$ in

Deflection due to creep:

$C_d := 0.30$ Average value between 50 and 60 Durometer **LRFD [Table 14.7.6.2-1]** (see E27-1.1)

$\delta_{CR} := C_d \cdot \delta_{DL}$ Creep effect defl. $\delta_{CR} = 0.026$ in

Compressive deflection due to live load:

$\delta_{LL} := \delta - \delta_{DL}$ Live load defl. $\delta_{LL} = 0.027$ in

Deflection due to creep and live load: **LRFD [C14.7.5.3.6]**

$\delta_{CRLL} := \delta_{CR} + \delta_{LL} \le 1/8"$ $\delta_{CRLL} = 0.052$ in

(see 27.2.1-Step 7-**WisDOT policy item**) $\delta_{CRLL} =$ "< 0.125 in., OK"

Initial compressive deflection of a single internal layer due to total load:

$\varepsilon_{int} \cdot h_{ri} < 0.09 \cdot h_{ri}$ **LRFD [14.7.6.3.3]** $\varepsilon_{int} \cdot h_{ri} = 0.015$ in

$0.09 \cdot h_{ri} = 0.045$ in

check = "OK"

E27-1.8 Anchorage

LRFD [14.8.3]

Shear force generated from deformation in the bearing due to temperature movement:

$H_{bu} := G \cdot A \cdot \frac{\Delta_u}{h_{rt}}$ **LRFD [Equation 14.6.3.1-2]**

$G := 0.165$ ksi conservative assumption, maximum value of G

Factored shear deformation of the elastomer

$\Delta_u := \gamma_{TU} \cdot \Delta_s$ (see E27-1.4 for γ_{TU} and Δ_s) $\Delta_u = 1.996$ in

Plan area of elastomeric element

$L = 15$ in $W = 24$ in

$A := L \cdot W$ $A = 360$ in^2

$H_{bu} := G \cdot A \cdot \frac{\Delta_u}{h_{rt}}$ (see E27-1.4 for h_{rt}) $H_{bu} = 29.638$ kips

(This value of H_{bu} can be used for substructure design)

Minimum vertical force due to permanent loads: (Check if Anchorage Design is req'd.)

$\gamma_{DLserv} := 1.0$ (Dead load factor - Service load **LRFD[Table 3.4.1-1]**)

$P_{sd} := \gamma_{DLserv} \cdot (DL_{serv} - DL_{ws})$ $P_{sd} = 144$ kips

$\sigma := \frac{P_{sd}}{A}$ $\sigma = 0.400$ ksi

$\sigma =$ "> 0.200 ksi, OK, anchorage is not required per WisDOT exception to AASHTO"

(see 27.2.1-Step 8- where this **WisDOT exception** is stated)

E27-1.9 Reinforcement

LRFD [14.7.6.3.7, 14.7.5.3.5] Min. reinforcement thickness = 0.0747 in.

check_min._thick. = "OK"

Service limit state:

$h_{max} := h_{ri}$ Internal layer thickness $h_{max} = 0.5$ in

$\sigma_s = 0.636$ ksi (see E27-1.6) Avg. compressive stress due to Total Load

$F_y = 36$ ksi Reinf. yield strength

$h_s \geq \dfrac{3 \cdot h_{max} \cdot \sigma_s}{F_y}$ **LRFD [Eq 14.7.5.3.5-1]** $h_s = 0.125\ in$ (steel plate thickness)

$\dfrac{3 \cdot h_{max} \cdot \sigma_s}{F_y} = 0.027\ in$

check = "< hs, OK"

Fatigue limit state:

$h_s \geq \dfrac{2 \cdot h_{max} \cdot \sigma_L}{\Delta F_{TH}}$ **LRFD [Eq 14.7.5.3.5-2]** $h_s = 0.125\ in$ (steel plate thickness)

$\sigma_L = 0.172$ ksi   (see E27-1.7)   Avg. compressive stress due to Live Load

$\Delta F_{TH} := 24.0$ ksi   Constant amplitude fatigue threshold for Category A
LRFD [Table 6.6.1.2.3-1]

$\dfrac{2 \cdot h_{max} \cdot \sigma_L}{\Delta F_{TH}} = 0.007$ in

check = "< hs, OK"

E27-1.10 Rotation

LRFD [14.7.6.3.5, C14.7.6.1]

Design for rotation in Method A is implicit in the geometric and stress limit requirements spelled out for this design method. Therefore no additional rotation calculations are required. (see 27.2.1-Step 10 ; **WisDOT Exception to AASHTO**)

Check requirement for tapered plate: **LRFD [14.8.2]**

Find the angle between the alignment of the underside of the girder and a horizontal line. Consider the slope of the girder, camber of the girder, and rotation due to unfactored dead load deflection.

Inclination due to grade line:

$L_{span} := 150$ Span length, ft (Between Abut. and Pier)

@ pier:

$EL_{Pseat} := 856.63$ Beam seat elevation at the pier, in feet

$h_{Pbrg} := 0.5$ Bearing height at the pier, in

Bottom of girder elevation at the pier, in feet

$EL_1 := EL_{Pseat} + \dfrac{h_{Pbrg}}{12}$ $EL_1 = 856.672\ ft$

@ abutment:

$EL_{Aseat} := 853.63$ Beam seat elevation at the abutment, in feet

$t_{plate} := 1.5$ Steel top plate thickness, in

$H = 5$ Total elastomeric bearing height, in (from E27-1.4)

Total bearing height, at the abutment, in

$h_{Abrg} := H + t_{plate}$ $h_{Abrg} = 6.5$ in

Bottom of girder elevation at the abutment, in feet

$$EL_2 := EL_{Aseat} + \frac{h_{Abrg}}{12}$$ $EL_2 = 854.172$ ft

Slope of girder

$$S_{GL} := \frac{\left|EL_1 - EL_2\right|}{L_{span}}$$ $S_{GL} = 0.017$ ft/ft

Note: The slope of girder is positive (+) when measured from the assumed "minimum thickness" side of the plate. Based on this orientation, the residual camber will either be positive (+) or negative (-).

Inclination due to grade line in radians

$\theta_{GL} := \operatorname{atan}\left(S_{GL}\right)$ $\theta_{GL} = 0.017$ radians

Inclination due to residual camber:

$\Delta_{camber} := 3.83$ Maximum camber of girder, in

$\Delta_{DL} := 2.54$ Maximum dead load deflection, in

$\Delta_{LL} := 0.663$ Maximum live load deflection, in

Residual camber, in

$\Delta_{RC} := \Delta_{camber} - \Delta_{DL}$ $\Delta_{RC} = 1.290$ in

To determine the slope due to residual camber, use a straight line from C/L Bearing to the 1/10 point. Assume that camber at 1/10 point is 40% of maximum camber (at midspan).

$$S_{RC} := \frac{0.4 \cdot \Delta_{RC}}{0.1 \cdot L_{span} \cdot 12}$$ Slope due to residual camber $S_{RC} = 0.003$ in/in

Inclination due to residual camber in radians

$\theta_{RC} := \text{atan}(S_{RC})$ $\theta_{RC} = 0.003$ radians

Total inclination due to grade line and residual camber in radians (θ_{SX}) :

Note: In this example, inclination due to grade line and residual camber are in the same direction. As such, the residual camber value is added (+) to the grade line value. If inclinations are not in the same direction, the residual camber value is subtracted (-) from the grade line value.

$\theta_{SX} := \theta_{GL} + \theta_{RC}$ (Check if $\theta_{SX} \le 0.01$ radians) $\theta_{SX} = 0.020$ radians

$\theta_{SX} =$ “> 0.01 rad, NG, top plate must be tapered”

(The plate should also be tapered if θ_{SX} x L_P >= 1/8") ; where L_p is the length of the top plate (see 27.2.1-Step 10-**WisDOT policy item**)

Top plate dimensions:

$t_{plate} = 1.5$ Minimum thickness of top plate, in

$L_p := L + 2$ Length of top plate, in (see E27-1.6 for L)

$L_p = 17$ in

$L_p \cdot \theta_{SX} = 0.332$ in

Thickness of top plate on thicker edge

$t_{pmax} := t_{plate} + L_p \cdot \tan(\theta_{SX})$ $t_{pmax} = 1.832$ in

E27-1.11 Bearing Summary

Laminated Elastomeric Bearing Pad:

Length = 15 inches

Width = 24 inches

Steel reinforcing plates: 8 @ 1/8"

Internal elastomer layers: 7 @ 1/2"

Cover elastomer layers: 2 @ 1/4"

Total pad height: 5 inches

Steel Top Plate (See standard detail):

Length = 17 inches

Width = 30 inches

Thickness = 1 1/2" to 1 7/8"

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32.1 General

The Regional Office shall determine the utilities that will be affected by the construction of any bridge structure at the earliest possible stage. It shall be their responsibility to deal with these utilities and to provide location plans or any other required sketches for their information. When the utility has to be accommodated on the structure, the Regional Office shall secure approval from the representative of the utility and the Bureau of Structures for the location and method of support.

Due consideration shall be given to the weight of the pipes, ducts, etc. in the design of the beams and diaphragms. To insure that the function, aesthetics, painting and inspection of stringers of a structure are maintained, the following applies to the installation of utilities on structures:

1. Permanent installations, which are to be carried on and parallel to the longitudinal axis of the structure, shall be placed out of sight, between the fascia beams and above the bottom flanges, on the underside of the structure.

2. Conglomeration of utilities in the same bay shall be avoided in order to facilitate maintenance painting and future inspection of girders in a practical manner.

3. In those instances where the proposed type of superstructure is not adaptable to carrying utilities in an out-of-sight location in the underside of the structure, an early determination must be made as to whether or not utilities are to be accommodated and, if so, the type of superstructure must be selected accordingly.

32.7 High Mast Lighting

High mast lighting structures are generally large diameter post structures with typical heights of 100 to 150 feet supporting luminaires. Foundations are designed by the Department (in-house or consultant) using criteria found in Chapter 39.5, and anchor rods and poles are contractor designed in accordance with Section 532 of the Standard Specifications. Poles and foundations should be designed to support 6 luminaires, regardless of the final lighting configuration. Currently, these structures are not designed to be breakaway upon impact. Refer to 2.5 for assigning structure numbers. Refer to 10.2 for subsurface exploration information. Contact the BOS ancillary structures maintenance engineer for additional questions.

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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.

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 3.11.6.4.1-1]**, 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. **LRFD [3.6.1.2.6a]** also states, if designing a culvert with fill of 2 feet or more, calculate live load design moments using the method in **LRFD [3.6.1.2.6b-c]** and also calculate live load design moments using the method in **LRFD [4.6.2.10]**. Then select and use the method that provides the smaller live load design moments.

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:

36.4.6.2 Depth of Fill Greater than or Equal to 2.0 ft.

Where the depth of fill is 2.0 ft or greater, follow **LRFD [3.6.1.2.6b-c]**. The effect of multiple lanes shall be considered. Use the multiple presence factor, m, as required per **LRFD [3.6.1.1.2]**.

36.4.6.2.1 Case 1 – Traffic Travels Parallel to Span

When the traffic travels primarily parallel to the span, follow **LRFD [3.6.1.2.6b]**.

For live load distribution <u>transverse</u> to span, the wheel/axle load interaction depth, H_{int-t}, shall be:

$$H_{\text{int}-t} = \frac{S_w - W_t/12 - 0.06D/12}{LLDF} \quad \text{(ft)}$$

where $H < H_{int-t}$ (no lateral interaction); then $W_w = W_t/12 + LLDF \cdot (H) + 0.06 \cdot (D/12)$

where $H \geq H_{int-t}$ (lateral interaction); then $W_w = W_t/12 + S_w + LLDF \cdot (H) + 0.06 \cdot (D/12)$

For live load distribution <u>parallel</u> to span, the wheel/axle load interaction depth H_{int-p} shall be:

$$H_{\text{int}-p} = \frac{S_a - \ell_t/12}{LLDF} \quad \text{(ft)}$$

where $H < H_{int-p}$ (no longit. interaction); then $\ell_w = \ell_t/12 + LLDF \cdot (H)$

where $H \geq H_{int-p}$ (longit. interaction); then $\ell_w = \ell_t/12 + S_a + LLDF \cdot (H)$

Where:

D	=	Clear span of the culvert (in)
H	=	Depth of fill from top of culvert to top of pavement (in)
H_{int-t}	=	Wheel interaction depth transverse to span (ft)
H_{int-p}	=	Axle interaction depth parallel to span (ft)
LLDF	=	Live load distribution factor per **LRFD [Table 3.6.1.2.6a-1]**; (1.15)
W_t	=	Width of tire contact area, per **LRFD [3.6.1.2.5]**; (20 in)
ℓ_t	=	Length of tire contact area, per **LRFD [3.6.1.2.5]**; (10 in)
S_w	=	Wheel spacing; (6.0 ft)

S_a = Axle spacing (ft)

W_w = Live load patch width at depth H (ft)

ℓ_w = Live load patch length at depth H (ft)

$$A_{LL} = \ell_w \cdot W_w$$

Where:

A_{LL} = Rectangular area at depth H (ft^2)

The live load vertical crown pressure shall be:

$$P_L = \frac{P(1 + IM/100)(m)}{A_{LL}}$$

Where:

IM = Dynamic load allowance (%); (see 36.4.8)

m = Multiple presence factor per **LRFD [3.6.1.1.2]**

P = Live load applied at surface on all interacting wheels (kip)

P_L = Live load vertical crown pressure (ksf)

The longitudinal and transverse distribution widths for depths of fill greater than or equal to 2.0 feet are illustrated in Figure 36.4-4.

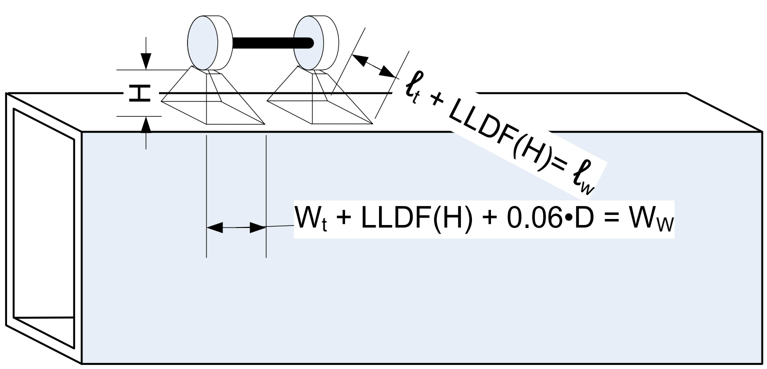
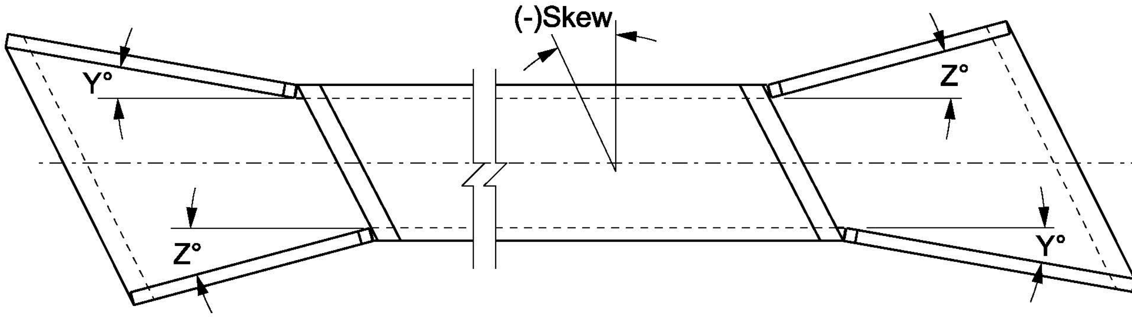
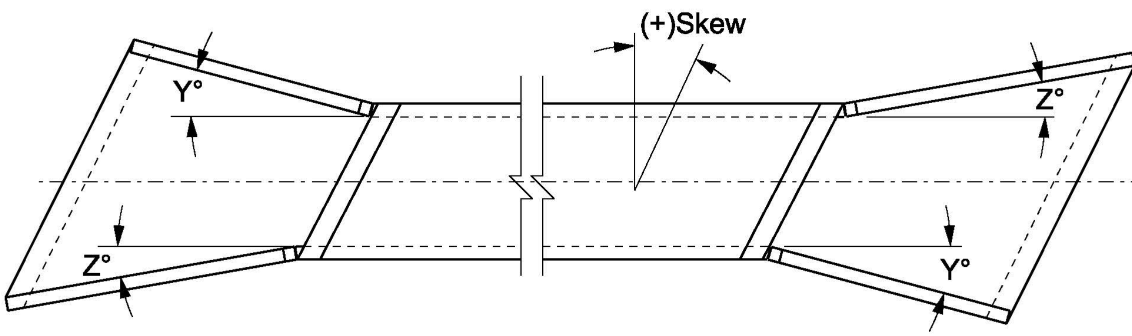


Figure 36.4-4
Distribution of Wheel Loads, Depth of Fill $\geq$ 2.0 feet (no lateral interaction)



Skew		Wing Type B		Wing Type C		Wing Type D	
Greater Than	To Include	Angle Y	Angle Z	Angle Y	Angle Z	Angle Y	Angle Z
0°	7.5°	15°	15°	30°	30°	45°	45°
7.5°	15.0°	15°	15°	25°	30°	40°	45°
15.0°	22.5°	10°	15°	25°	30°	35°	45°
22.5°	37.5°	10°	15°	20°	30°	30°	45°
37.5°	45.0°	10°	15°	15°	30°	25°	45°
45.0°	52.5°	5°	15°	15°	30°	20°	45°
52.5°	67.5°	5°	15°	10°	30°	15°	45°
67.5°	75.0°	5°	15°	5°	30°	10°	45°
75.0°	82.5°	0°	15°	5°	30°	5°	45°
82.5°	90.0°	0°	15°	0°	30°	0°	45°



Skew		Wing Type B		Wing Type C		Wing Type D	
Greater Than	To Include	Angle Y	Angle Z	Angle Y	Angle Z	Angle Y	Angle Z
0°	7.5°	15°	15°	30°	30°	45°	45°
7.5°	15.0°	15°	15°	30°	25°	45°	40°
15.0°	22.5°	15°	10°	30°	25°	45°	35°
22.5°	37.5°	15°	10°	30°	20°	45°	30°
37.5°	45.0°	15°	10°	30°	15°	45°	25°
45.0°	52.5°	15°	5°	30°	15°	45°	20°
52.5°	67.5°	15°	5°	30°	10°	45°	15°
67.5°	75.0°	15°	5°	30°	5°	45°	10°
75.0°	82.5°	15°	0°	30°	5°	45°	5°
82.5°	90.0°	15°	0°	30°	0°	45°	0°

Figure 36.7-2
Wing Type B, C, D (Angles vs. Skew)

36.7.3 Type E

Type E is used primarily in urban areas where a sidewalk runs over the culvert and it is necessary to have a parapet and railing along the sidewalk. For Type E the wingwalls run parallel to the roadway just like the abutment wingwalls of most bridges. It is also used where Right of Way (R/W) is a problem and the aprons would extend beyond the R/W for other types. Wingwall lengths for Type E wings are based on a minimum channel side slope of 1.5 to 1.

36.7.4 Wingwall Design

Culvert wingwalls are designed using a 1 foot surcharge height, a unit weight of backfill of 0.120 kcf and a coefficient of lateral earth pressure of 0.5, as discussed in 36.4.3. When the wingwalls are parallel to the direction of traffic and where vehicular loads are within ½ the wall height from the back face of the wall, design using a surcharge height representing vehicular load per **LRFD [Table 3.11.6.4.1-2]**. Load and Resistance Factor Design is used, and the load factor for lateral earth pressure of γ_{EH} = 1.69 is used, based on past design experience. The lateral earth pressure was conservatively selected to keep wingwall deflection and cracking to acceptable levels. Many wingwalls that were designed for lower horizontal pressures have experienced excessive deflections and cracking at the footing. This may expose the bar steel to the water that flows through the culvert and if the water is of a corrosive nature, corrosion of the bar steel will occur. This phenomena has led to complete failure of some wingwalls throughout the State.

For wing heights of 7 feet or less determine the area of steel required by using the maximum wall height and use the same bar size and spacing along the entire wingwall length. The minimum amount of steel used is #4 bars at 12 inch spacing. Wingwall thickness is made equal to the barrel wall thickness.

For wing heights over 7 feet the wall length is divided into two or more segments to determine the area of steel required. Use the same bar size and spacing throughout each segment, as determined by using the maximum wall height in the segment.

Wingwalls must satisfy Strength I Limit State for flexure and shear, and Service I Limit State for crack control, minimum reinforcement, and reinforcement spacing. Adequate shrinkage and temperature reinforcement shall be provided.

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40.1 General

New bridges are designed for a minimally expected life of 75 years. Preliminary design considerations are site conditions, structure type, geometrics, and safety. Refer to Bridge Manual Chapters 9 and 17 for Materials and Superstructure considerations, respectively. Comprehensive specifications and controlled construction inspection are paramount to obtaining high quality structures. Case history studies show that adequately consolidated and properly cured concrete with low water-cement ratios and good air void systems have lower absorption rates and provide greater resistance to scaling and chloride penetration under heavy traffic and exposure to de-icing chemicals. Applying protective surface treatments to new decks improves their resistance to first year applications of de-icing chemicals.

Most interstate and freeway structures are not subject to normal conditions and traffic volumes. Under normal environmental conditions and traffic volumes, original bridge decks have an expected life of 40 years. Deck deterioration is related to the deck environment which is usually more severe than for any of the other bridge elements. Decks are subjected to the direct effects of weather, the application of chemicals and/or abrasives, and the impact of vehicular traffic. For unprotected bar steel, de-icing chemicals are the primary cause of accelerated bridge deck deterioration. Chlorides cause the steel to corrode and the corrosion expansion causes concrete to crack along the plane of the top steel. Traffic breaks up the delaminated concrete leaving potholes on the deck surfaces. In general, deck rehabilitation on Wisconsin bridges has occurred after 15 to 22 years of service due to abnormally high traffic volumes and severe environment.

Full depth transverse floor cracks and longitudinal construction joints leak salt water on the girders below causing deterioration and over time, section loss.

Leaking expansion joints allow salt water seepage which causes deterioration of girder ends and steel bearings located under them. Also, concrete bridge seats will be affected in time. Concrete bridge seats should be finished flat, and sealed with a penetrating epoxy coating.

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

40.16 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 318-22 manual, hereafter referred to as ACI. (Refer to **LRFD [5.13]** for current AASHTO guidance)

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 [17.5.2.9]** and **ACI [17.5.9]** 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 318-22 Chapter 17 is allowable, which may yield higher capacities.

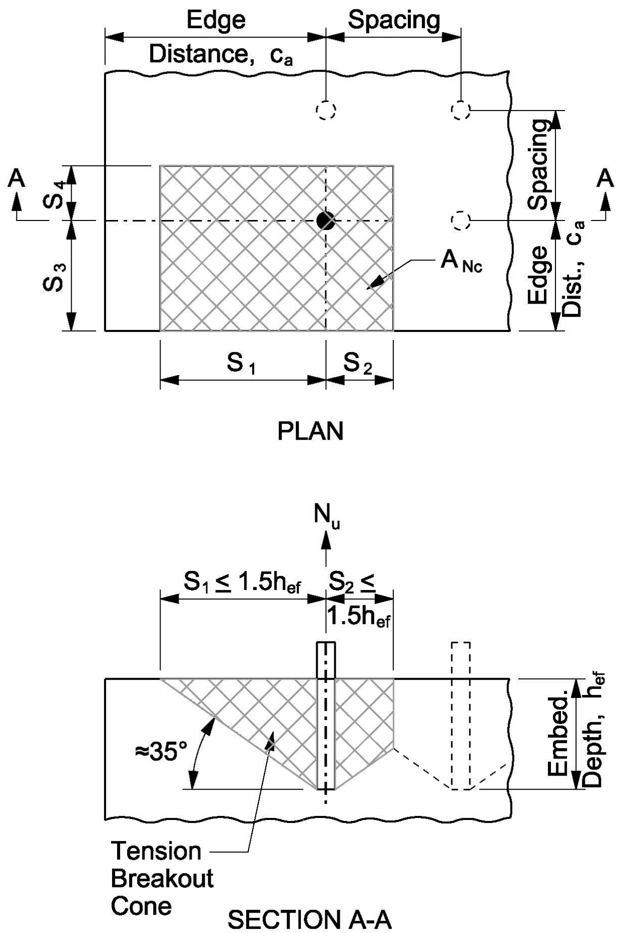

Figure 40.16-1
Concrete Breakout of Concrete Anchors in Tension

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.

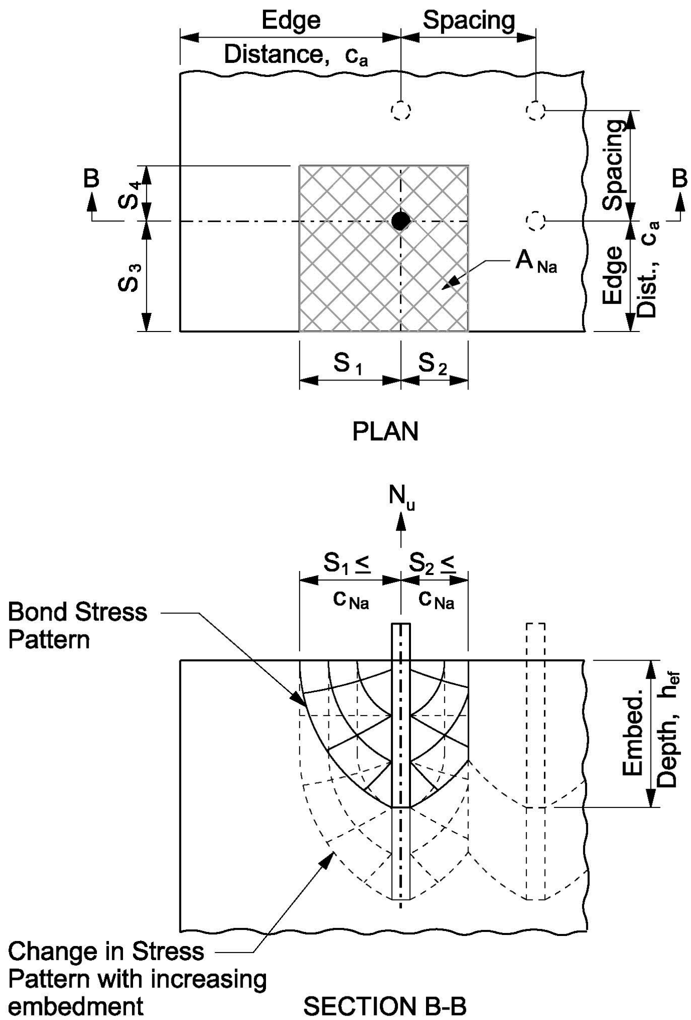

Figure 40.16-2
Bond Failure of Concrete Adhesive Anchors in Tension

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	670	370	280
#5 or 5/8"	970	720	510	410
#6 or 3/4"	950	580	500	420
#7 or 7/8"	930	580	490	420
#8 or 1"	770	580	600	490

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:

$$N_r = \phi_{ts} N_{sa} \le \phi_{tc} N_{cb} \le \phi_{tc} N_{pn}$$

In which:

ϕ_{ts} = Strength reduction factor for anchors in concrete, **ACI [17.5.3]**
= 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 [17.6.1.2]**

= $A_{se,N} f_{uta}$

$A_{se,N}$ = Effective cross-sectional area of anchor in tension (in^2)

f_{ya} = Specified yield strength of anchor steel (ksi)

f_u = Specified minimum tensile strength of anchor steel (ksi)

f_{uta} = Specified tensile strength of anchor steel (ksi) and not to exceed $1.9f_{ya}$ or 125 ksi, **ACI [17.6.1.2]**

= min (f_u, 1.9 f_{ya}, 125 ksi)

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 [17.6.2.1]**

$$= \frac{A_{Nc}}{9(h_{ef})^2}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$$

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 [17.6.2.1.2]** 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 [17.6.2.5]**

= 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 [17.6.2.6]**
= 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 [17.6.2.6]**

= 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), **ACI [17.9.5]**

= $4.0h_{ef}$

N_b = Concrete breakout strength of a single anchor in tension in uncracked concrete, **ACI [17.6.2.2]**

= $0.538\sqrt{f'_c}(h_{ef})^{1.5}$ (kips)

N_{pn} = Nominal pullout strength of a single anchor in tension, **ACI [17.6.3]**

= $\psi_{c,P}N_p$

$\psi_{c,P}$ = Modification factor for pullout strength of anchors based on the presence or absence of cracks in concrete, **ACI [17.6.3.3]**

= 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

<u>For adhesive anchors:</u>

$$N_r = \phi_{ts}N_{sa} \leq \phi_{tc}N_{cb} \leq \phi_{tc}N_a$$

In which:

N_{cb} = Nominal concrete breakout strength in tension, **ACI [17.6.2.1]**

= $\frac{A_{Nc}}{9(h_{ef})^2}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$

h_{ef} = Effective embedment depth of anchor. May be reduced per **ACI [17.6.2.1.2]** 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 [17.6.2.6]**

= 1.0 if $c_{a,min} \geq c_{ac}$

= $\dfrac{c_{a,min}}{c_{ac}} \geq \dfrac{1.5h_{ef}}{c_{ac}}$ 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 [17.6.5]**

= $\dfrac{A_{Na}}{4c_{Na}^{2}}\psi_{ed,Na}\psi_{cp,Na}N_{ba}$

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 [17.6.5.4]**

= 1.0 if $c_{a,min} \geq c_{Na}$

= $0.7 + 0.3\dfrac{c_{a,min}}{c_{Na}}$ 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

= $10d_a\sqrt{\dfrac{\tau_{uncr}}{1100}}$ (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 [17.6.5.5]**

= 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 [17.6.5.2]**
= $\tau_{cr} \pi d_a h_{ef}$

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 [17.5.2.2]**:

$0.50\ \phi_{tc} N_{ba} \geq N_{ua,s}$

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).

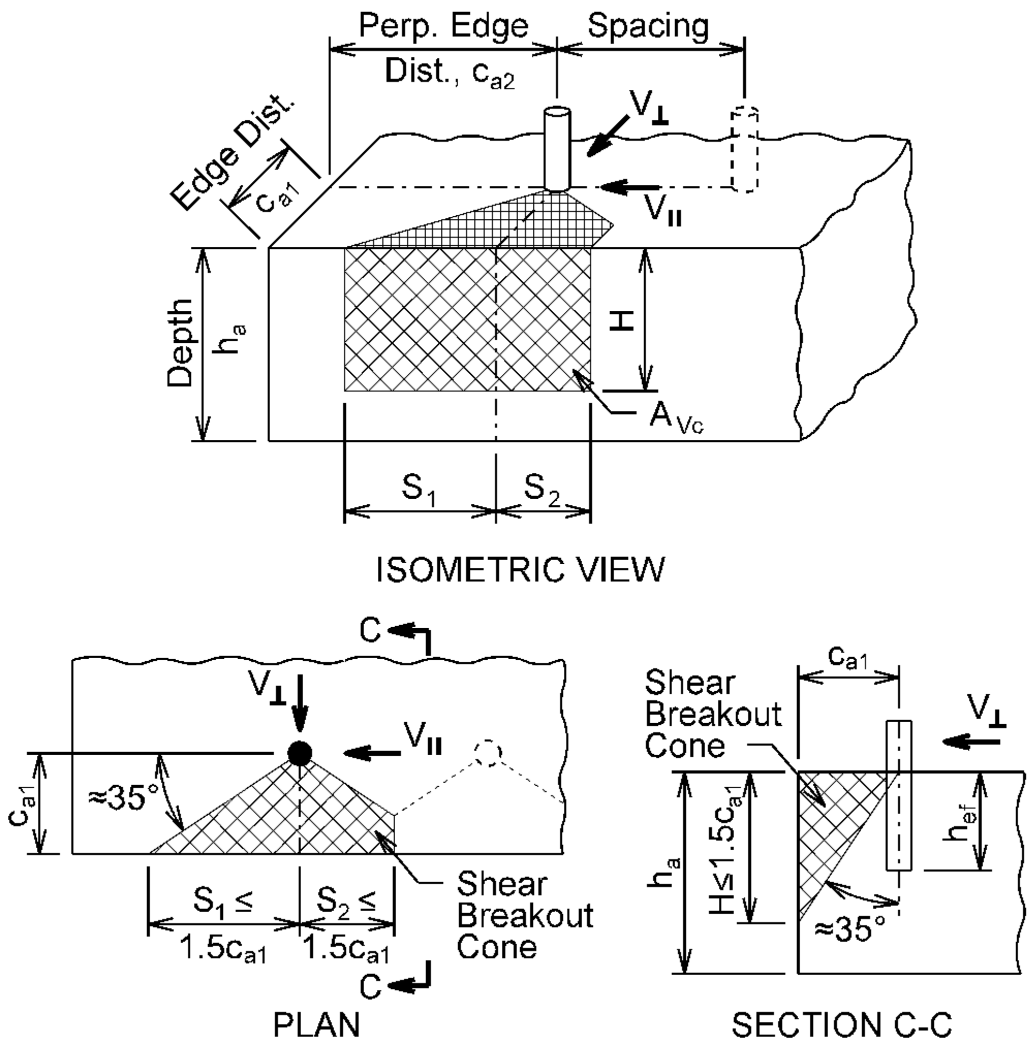

Figure 40.16-3
Concrete Breakout of Concrete Anchors in Shear

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.

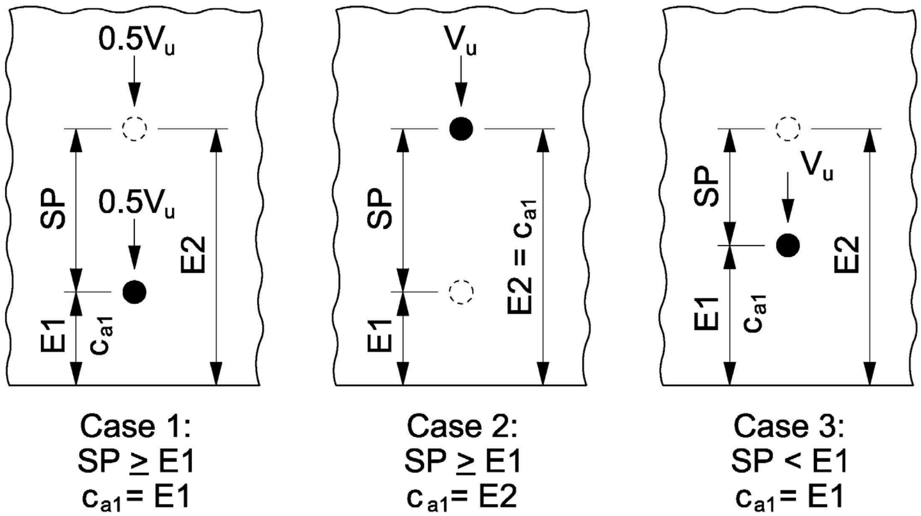

Figure 40.16-4
Concrete Anchor Shear Force Cases

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 [17.5.3]**
= 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 [17.7.1.2]**
= $0.6 A_{se,V} f_{uta}$

$A_{se,V}$ = Effective cross-sectional area of anchor in shear (in^2)

ϕ_{vc} = Strength reduction factor for anchors in concrete, **ACI [17.5.3]**
= 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 [17.7.2.1]**
= $\dfrac{A_{Vc}}{4.5(c_{a1})^2} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{p,V} V_b$

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 [17.7.2.4]**
= 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 [17.7.2.5]**
= 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 [17.7.2.6]**
= $\sqrt{\dfrac{1.5c_{a1}}{h_a}} \geq 1.0$

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 [17.7]**
= 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 [17.7.2.2]** , 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 [17.7.3.1]**
= $2.0N_{cp}$

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 [17.8]**.

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.

<u>Typical notes for bridge plans (shown in all capital letters):</u>

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.