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6.3.3.3 Miscellaneous Structures

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

6.3.3.4 Standard Drawings

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

6.3.3.5 Insert Sheets

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

6.3.3.6 Change Orders and Maintenance Work

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

6.3.3.7 Name Plate and Benchmarks

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

A benchmark location shall be shown on bridge and larger culvert plans. Locate the benchmark on a horizontal surface flush with the concrete and in close proximity to the name plate. When possible, locate on top of the parapet on the bridge deck, above the abutment. Do not locate benchmarks at locations where elevations are subject to movement (e.g. midspan) and avoid placing below a rail or fence system. Benchmarks are typically metal survey disks, which are to be supplied by the department and set by the contractor. See FDM 9-25-5 for additional benchmark information.

6.3.3.8 Removing Structure and Debris Containment

This section provides guidance for selecting the appropriate Removing Structure bid item and determining when to use the "Debris Containment" bid item.

The "Removing Structure (structure)" bid item is most typically used for complete or substantial removals, as described in 6.3.3.8.2, of grade separation structures. In addition to this Standard Specification bid item, there are three additional Standard Specification bid items for complete or substantial removal work over waterways: "Removing Structure Over Waterway Remove Debris (structure)"; "Removing Structure Over Waterway Minimal Debris (structure)"; and "Removing Structure Over Waterway Debris Capture (structure)". If these four Standard Specification bid items do not encapsulate site specific constraints for specialized cases, which should be a rare occurrence, the designer can utilize special provisions to augment the standard spec removal items.

The designer should review all of these Standard Specifications, and coordinate with the Wisconsin Department of Natural Recourses (DNR) to reach consensus on which bid items to use when removing a particular structure. **The designer should not automatically defer to the recommendation from the initial DNR letter, but should work with WisDOT and DNR environmental coordinators, considering constructability and cost impacts of the items.** For unique or difficult removals, designers should consult with the contracting community to assess costs and the feasibility of a particular removal technique. One of the following Removing Old Structure bid items should be selected for removals over waterways:

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

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

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

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

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

6.3.3.8.1 Structure Repairs

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

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

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

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

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

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

6.3.3.8.2 Complete or Substantial Removals

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

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

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

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

6.3.4 Checking Plans

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

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

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

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

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

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

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

1. QC/QA sign-off sheet
2. Design computations and computer runs
3. Quantity computations
4. Bridge Special Provisions and STSP’s (only those STSP’s requiring specific blanks to be filled in or contain project specific information)
5. Final Structure Survey Report form (not including photos, cross-sections, project location maps, etc.)

6. Final Geotechnical Report
7. Final Hydrology and Hydraulic computations and structure sizing report
8. Contour map

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

1. Miscellaneous correspondence and transmittal letters
2. Preliminary drawings and computations
3. Prints of soil borings and plan profile sheets
4. Shop steel quantity computations*
5. Design checker computations
6. Layout sheets
7. Elevation runs and bridge geometrics
8. Falsework plans*
9. Miscellaneous Test Report
10. Photographs of bridge rehabs

* These items are added to the packet during construction.

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

1. All "void" material
2. All copies except one of preliminary drawings
3. Extra copies of plan and profile sheets
4. Preliminary computer design runs

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

6.4.9 Piling Test Treated Timber (Structure)

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

6.4.10 Piling CIP Concrete (Size)(Shell Thickness), Piling Steel HP (Size)

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

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

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

6.4.11 Preboring CIP Concrete Piling or Steel Piling

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

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

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

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

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

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

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

Record this quantity to the nearest 1 cubic yard.

6.4.15 Pile Points

When recommended in soils report. Bid as each.

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

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

6.4.17 Cofferdams (Structure)

Lump Sum

6.4.18 Rubberized Membrane Waterproofing

Record the quantity to the nearest square yard.

6.4.19 Expansion Devices

For "Expansion Device" and "Expansion Device Modular", bid the items in lineal feet. The distance measured is from the outermost extent of the expansion device along the skew (do not include turn-ups into parapets or medians).

6.4.20 Electrical Work

Refer to Standard Construction Specifications for bid items.

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

Record this quantity in feet.

6.4.22 Preparation Decks Type 1 or Preparation Decks Type 2

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

6.4.23 Cleaning Decks

Record this quantity to the nearest square yard.

6.4.24 Joint Repair

Record this quantity to the nearest square yard.

6.4.25 Concrete Surface Repair

Record this quantity to the nearest square foot.

6.4.26 Full-Depth Deck Repair

Record this quantity to the nearest square yard.

6.4.27 Concrete Masonry Overlay Decks

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

6.4.28 Removing Structure and Debris Containment

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

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

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

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

Bid as each.

6.4.29 Anchor Assemblies for Steel Plate Beam Guard

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

6.4.30 Steel Diaphragms (Structure)

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

6.4.31 Welded Stud Shear Connectors X -Inch

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

6.4.32 Concrete Masonry Seal

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

6.4.33 Geotextile Fabric Type

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

6.4.34 Concrete Adhesive Anchors

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

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

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

6.4.36 Piling Steel Sheet Temporary

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

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

6.4.37 Temporary Shoring

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

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

6.4.38 Concrete Masonry Deck Repair

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

6.4.39 Sawing Pavement Deck Preparation Areas

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

6.4.40 Removing Bearings

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

6.4.41 Ice Hot Weather Concreting

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

6.4.42 Asphaltic Overlays

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

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

Coordinate asphaltic quantity assumptions with the Region and roadway designers.

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

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

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

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

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

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

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

Provisions for periodic reviews and update of the QA/QC plan with a frequency no less than 5 years; or as needed due to changes in the firm's personnel or firm's processes or procedures; or as requested by BOS.A QA/QC verification summary sheet is required as part of every final structure plan submittal, demonstrating that the QA/QC plan and procedures were followed for that structure. The QA/QC verification summary sheet shall include the signoff or initialing by each individual that performed the tasks (design, checking, plan review, technical review, etc.) documented in the QA/QC plan and procedures. The summary sheet must be submitted with the final structure plans as part of the e-submit process.

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

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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 (Q_{100}) frequency flood. In floodplain management this is also referred to as the Regional or Base flood. Design frequency is determined from requirements in Federal Highway Administration (FHWA) directives and the co-operative agreement between Wisconsin Department of Transportation (DOT) and Wisconsin Department of Natural Resources (DNR). The following publications are suggested for guidance.

8.1.3.1 FHWA Directive

Title 23, Chapter 1, Sub Chapter G, Part 650, Subpart A of the FHWA – Federal-Aid Policy Guide, "*Location and Hydraulic Design of Encroachments on Flood Plains*", prescribes FHWA policy and procedures. Copies of this directive may be found on the FHWA website.

8.1.3.2 DNR-DOT Cooperative Agreement

The Wisconsin Department of Transportation and the Wisconsin Department of Natural Resources have signed a co-operative agreement to provide a reasonable and economical procedure for carrying out their respective duties in a manner that is in the total public interest. The provisions in this agreement establish the basic considerations for highway stream crossings. A copy of this agreement can be found in Facilities Development Manual (FDM) 20-5-15.

8.1.3.3 DOT Facilities Development Manual

Refer to FDM Chapter 10 – Erosion Control and Storm Water Quality, FDM Chapter 11 – Design, FDM Chapter 13 - Drainage, and FDM Chapter 20 - Environmental Documents, Reports and Permits.

8.1.4 Hydraulic Site Report

The "Stream Crossings Structure Survey Report" shall be submitted for all bridge and box culvert projects. When submitting preliminary structure plans for a stream crossing, a hydraulic site report shall also be included. A check list of the various discussion items that need to be provided in the hydraulic site report is included as 8.6 Appendix 8-A. Plan survey datum must conform to datum in use by local zoning authorities. In most cases elevations are referenced to the National Geodetic Vertical Datum (NGVD) of 1929, or to the North American Vertical Datum of 1988 (NAVD 88). The Hydraulic Site Report discusses and documents the hydrologic, hydraulic, site conditions, and all other pertinent factors that influence the type, size, and location of the proposed structure.

8.1.5 Hydraulic Design Criteria for Temporary Structures

The basic design criteria for temporary structures will to be the ability to pass a 5-year storm (Q5) with only 0.5 feet of backwater over existing conditions. This criteria is only a general guideline and site specific factors and engineering judgment may indicate that this criteria is inappropriate. Separate hydraulic design criteria should be used for the design of temporary construction causeways. Factors that should be considered in the design of temporary structures and approach embankments are:

- Effects on surrounding property and buildings
- Velocities that would cause excessive scour
- Damage or inconvenience due to failure of temporary structure

- DNR concerns
- Temporary roadway profile
- Structure depths will be 36” for short spans and 48” or more for longer spans.

If possible and practical, the temporary roadway profile should be designed and constructed in such a manner that infrequent flood events are not obstructed from overflowing the temporary profile and creating excessive backwaters upstream of the construction. The temporary roadway profile should provide adequate clearance for the temporary structure.

The roadway designer should indicate the need for a temporary structure on the Stream Crossing Structure Survey Report. Preliminary and Final plans should indicate the hydraulic parameters of the temporary structure. The required parameters are the 5-year flood discharge (Q5), the 5-year high-water elevation (HW5), and the flow area of the temporary structure required to pass the 5-year flood (Abr).

8.1.6 Erosion Control Parameters

In order to assist designers in determining the appropriate erosion control measures to be provided at Bridge construction site, preliminary and final plans should indicate the 2-year flood discharge (Q2), 2-year velocity, and the 2-year high-water elevation (HW2).

8.1.7 Bridge Rehabilitation and Hydraulic Studies

Generally no hydraulic study will be required in bridge rehabilitation projects that do not involve encroachment to the Base Floodplain. This includes entire super structure replacement provided that the substructure and berm configuration remain unchanged and the low cord elevation is not significantly lowered.

The designer should consider historical high-water elevations, Flood Insurance Studies and the potential of inundation when choosing the replacement superstructure type. The risk of damage to the structure as the result of Scour should also be considered.

8.2 Hydrologic Analysis

The first step in designing a hydraulic structure is to determine the design discharge for the waterway. The problem is particularly difficult for small watersheds, say under five square miles, because the smaller the area, the more sensitive it is to conditions which affect runoff and the less likely there are runoff records for the area.

Acceptable methods of determining the design discharge for the 100 year flood shall be based on the guidelines contained in the *State Administrative Code NR 116.07, Wisconsin's Floodplain Management Program*[1]. Generally, a minimum of two methods should be used in determining a design discharge.

The most frequently used methods for determining the design discharge for bridges and box culverts in the State of Wisconsin are discussed below.

8.2.1 Regional Regression Equations

The U. S. Geological Survey (USGS) in cooperation with the Wisconsin Department of Transportation prepared a report entitled *Flood Frequency Characteristics of Wisconsin Streams*[2] which considers the flood potentials for a site using regional regression equations based on flood data from gaging stations on Wisconsin's rivers and streams. The flood-frequency regression equations are correlated with three or more of seven parameters, namely, drainage area, main-channel slope, storage, forest cover, mean annual snowfall, precipitation intensity index, and soil permeability. These equations are applicable to all drainage areas in Wisconsin except for highly regulated streams, and highly urbanized areas of the state.

8.2.2 Watershed Comparison

The results obtained from the above regression equations should be compared to similar gaged watersheds listed in reference (2) above using the area transfer formulas and procedures detailed in that document. A good discussion and examples of the use of regression equations and basin comparison methods can be seen in the WisDOT Facilities Development Manual, Procedure 13-10-5. The flood frequency discharges listed in reference (2) are for flood records up to the year 2000. More years of data are available from the USGS for most of the gaged watersheds.

The flood frequency discharges for the gaged watersheds can be updated past water year 2000 by using the Log-Pearson Type III distribution method as described in *Bulletin #17B entitled Guidelines For Determining Flood Flow Frequency*[3] and the guidelines for weighting the station skew with the generalized skew in *NR116.07, Wisconsin's Floodplain Management Program*[1].

8.2.3 Flood Insurance and Floodplain Studies

The Federal Emergency Management Agency (FEMA) had contracted for detailed flood studies throughout Wisconsin. They were developed for floodplain management and flood insurance purposes. These Flood Insurance Studies (FIS) which are on file with Floodplain-

Shoreland Management Section of the Wisconsin Dept. of Natural Resources (DNR) contain discharge values for many sites. These studies, along with other various floodplain studies, may be obtained from the DNR's Floodplain Analysis Interactive Map by using the following link:

https://dnr.wi.gov/topic/floodplains/mapindex.html

8.2.4 Natural Resources Conservation Service

For small watersheds in urban and rural areas, the National Resources Conservation Service (NRCS) has developed procedures to calculate storm runoff volumes, peak rates of discharge, hydrographs and storage volumes. The procedure is documented in *Technical Release 55 Urban Hydrology for Small Watersheds*[4].

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. Generally, velocities through bridges of less than 10 feet per second are acceptable.

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. 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.

- If Precast Pretensioned Slab and Box Sections are in contact with water for flood events equal to or less than a 100-year event, the void in Precast Pretensioned Slab and Box Sections must be cast with a non-water absorbing material.

8.3.1.6 Scour

Investigation of the potential for scour at the bridge site is a design consideration for the bridge opening geometry and size, as well as pier and abutment design. Bridges shall be designed to withstand the effects of scour from a super-flood (a flood exceeding the 100-year flood) without failing; i.e., experiencing foundation movement of a magnitude that requires corrective action. See 8.3.2.7. Generally, scour associated with a 100-year event without significant reduction in foundation factor of safety will accomplish this objective. For situations where a combination of flow through a bridge and over the roadway exist, scour should also be evaluated for flow conditions at the onset of flow over topping when velocity through the bridge may be the greatest.

8.3.2 Design Procedures

8.3.2.1 Determine Design Discharge

See 8.2 for procedures.

8.3.2.2 Determine Hydraulic Stream Slope

The primary method of determining the hydraulic slope of a stream is surveying the water surface elevation through a reach of stream 1500 feet upstream to 1500 feet downstream of the site. Intermediate points through this reach should also be surveyed to detect any significant slope variation.

There are situations, particularly on flat stream profiles, where it is difficult to determine a realistic slope using survey data. This will occur at normal water surface elevation at the mouth of a stream, upstream of a dam, or other significant restriction in the stream. In this case a USGS 7-1/2" quadrangle map and existing flood studies of the stream can be investigated to determine a reasonable stream slope.

8.3.2.3 Select Floodplain Cross-Section(s)

Generally, a minimum of two floodplain valley cross-section(s) are required to perform the hydraulic analysis of a bridge. The sections shall be normal to the stream flow at flood stage and approximately one bridge length upstream and downstream of the structure. A detailed cross-section of one or both faces of the bridge will also be required. If the section is skewed to the flow, the horizontal stationing shall be adjusted using the cosine of the skew angle.

If the downstream boundary condition of the hydraulic model is using normal depth, then the most downstream cross-section in the model should be located far enough downstream from the bridge and should reflect the natural floodplain conditions.

Field survey cross-sections will be needed when a contour map is plotted using stereographic methods. A field survey section is needed for that portion below the normal water surface.

Cross-sections taken from contour maps are acceptable when the information is supplemented with field survey sections and data. Additional sections may be required to develop a proper hydraulic model for the site.

The hydraulic cross-sections should not include slack water portions of the flood plain or portions not contributing to the downstream movement of water.

Refer to FDM 9-55 for a discussion of Drainage Structure Surveys.

8.3.2.4 Assign "Manning n" Values to Section(s)

"Manning n" values are assigned to the cross-section sub-areas. Generally, the main channel will have different "manning n" values than the overbank areas. Values are chosen by on-site inspection, pictures taken at the section, and use of aerial photos defining the extent of each "n" value. There are several published sources on open channel hydraulics which contain tables for selecting appropriate "n" values. See 8.5 References (5) and (6).

8.3.2.5 Select Hydraulic Model Methodology

There are several public and private computer software programs available for modeling open channel hydraulics, bridge hydraulics, and culvert hydraulics. Public domain computer software programs most prevalent and preferred in Wisconsin bridge design work are "HEC-RAS" and "HY8".

The HEC-RAS program is currently the most widely used methodology for floodplain and bridge hydraulic modeling. HEC-RAS should be used where existing HEC-2 data is available from a previous Flood Insurance Study. "HY8" is a FHWA sponsored culvert analysis package based on the FHWA publication "Hydraulic Design of Highway culverts" (HDS-5), see 8.5 Reference (13).

1. HEC-RAS

   The hydrologic Engineering Center's River Analysis System (HEC-RAS) is the first of the U.S. Army Corps of Engineers "Next Generation" software packages. It is the successor to the HEC-2 program, which was originally developed by the Corps of Engineers in the early 1970's. HEC-RAS includes several data entry, graphing, and reporting capabilities. It is well suited for modeling water flowing through a system of open channels and computing water surface profiles to be used for floodplain management and evaluation of floodway encroachments. HEC-RAS can also be used for bridge and culvert design and analysis and channel modification studies.

   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 insure 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:

Q	=	discharge
C_f	=	coefficient of discharge for free flow conditions
B	=	length of flow section along the road normal to the direction of flow

H = total head = h + h_v

k_t = submergence factor

The length of overflow section (B) will be a function of the roadway profile grade line and depth of over-topping (h). Coefficient (C_f) is obtained by computing h/L and using Figure 8.3-1 or Figure 8.3-2, for paved or gravel roads.

The degree of submergence of a highway embankment is defined by ratio ht/H. The effect of submergence on the discharge coefficient (C_f) is expressed by the factor k_t as shown in Figure 8.3-3. The factor k_t is multiplied by the discharge coefficient (C_f) for free-flow conditions to obtain the discharge coefficient for submerged conditions. For roadway overflow conditions with high degree of submergence, HEC-RAS switches to energy based calculations of the upstream water surface. The default maximum submergence is 0.95, however that criterion may be modified by the user.

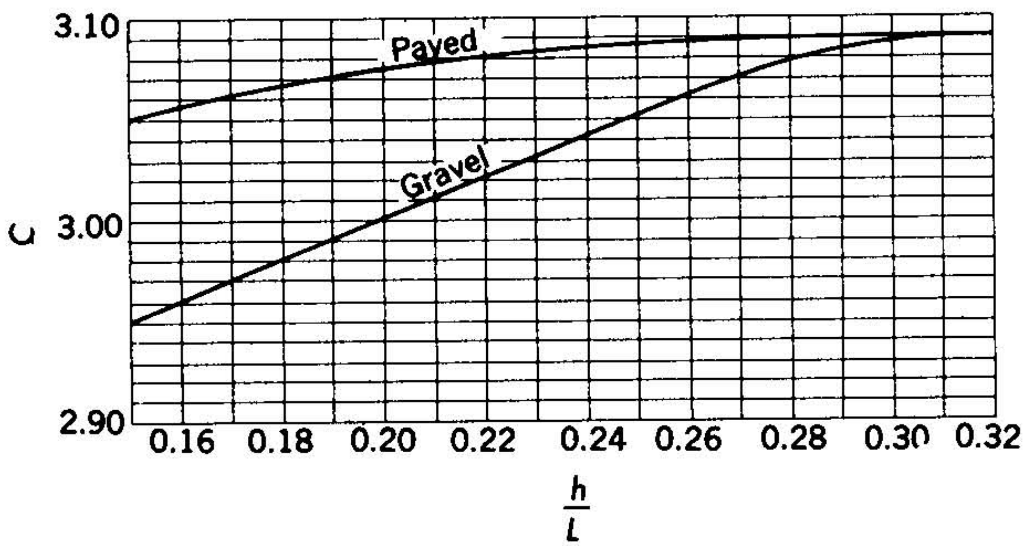


Figure 8.3-1
Discharge Coefficients, C_f, for Highway Embankments for H/L Ratios > 0.15

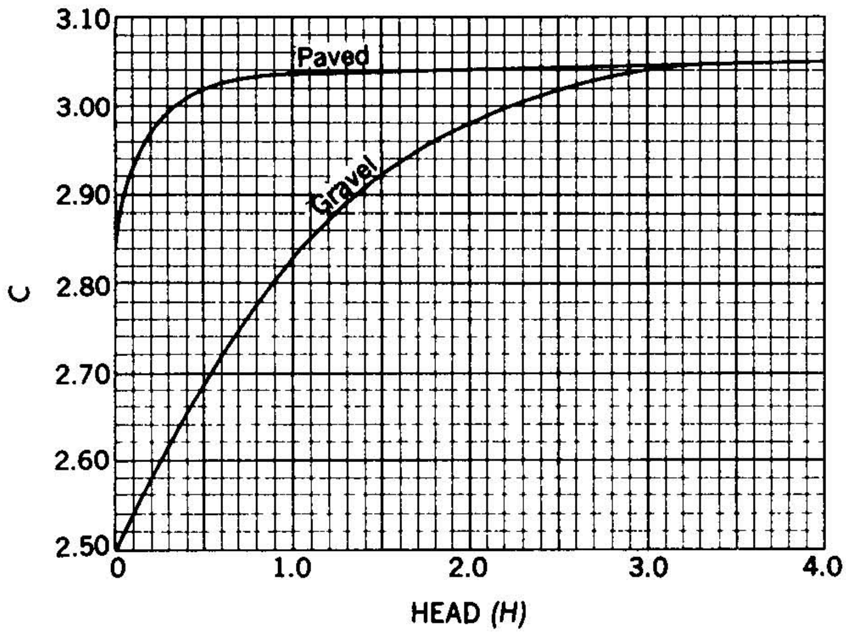


Figure 8.3-2
Discharge Coefficients, C_f, for Highway Embankments for H/L Ratios < 0.15

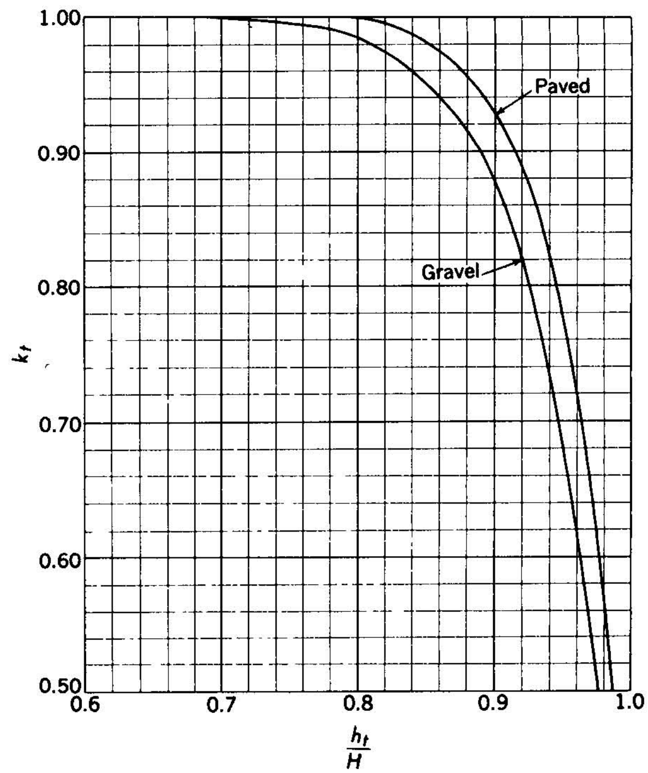


Figure 8.3-3
Definition of Adjustment Factor, k_t, for Submerged Highway Embankments

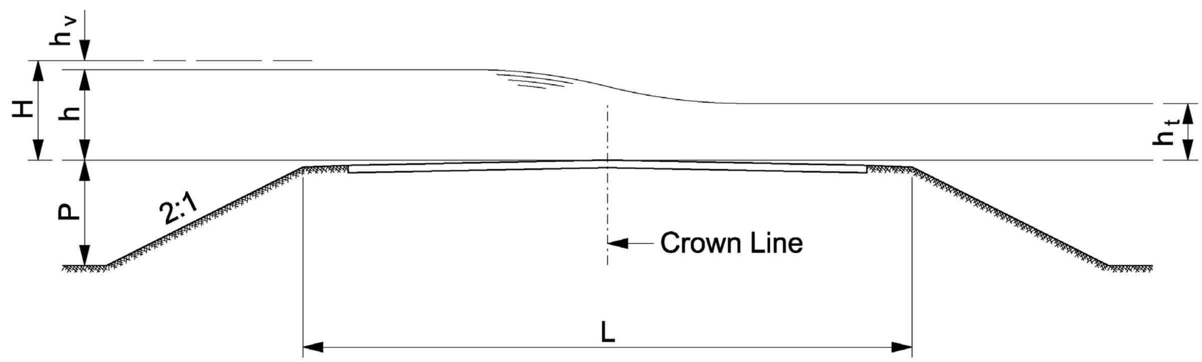


Figure 8.3-4
Definition Sketch of Flow Over Highway Embankment

8.3.2.7 Conduct Scour Evaluation

Evaluating scour potential at bridges is based on recommendations and background from FHWA Technical Advisory "*Evaluating Scour at Bridges*" dated October 28, 1991 and procedures from the *FHWA Hydraulic Engineering Circular No. 18, Evaluating Scour at Bridges, Fifth Edition*, April 2012[14], and *Hydraulic Engineering Circular No. 20, Stream Stability at Highway Structures, Fourth Edition*, April 2012[15]. Consult FHWA's website for the most current versions of the above publications.

All bridges shall be evaluated to determine the vulnerability to scour. In the FHWA publication *Recording and Coding Guide for Structure Inventory and Appraisal of the Nation's Bridges*[16], a code system has been established for evaluation. A section in this guide "Item 113 - Scour Critical Bridges" uses a single-digit code to identify the status of the bridge regarding its vulnerability to scour. The most current version of the Item 113 Scour Coding Guide can be found here: https://www.fhwa.dot.gov/engineering/hydraulics/policymemo/revguide.cfm.

A common program used to perform a full bridge scour analysis is FHWA's Hydraulic Toolbox. Hydraulic Toolbox software and supporting documentation can be downloaded directly from FHWA's website. The hydraulic sizing report should include a discussion of scour analysis results and provide justification for scour critical code selection. FHWA's Hydraulic Toolbox can be found here: https://www.fhwa.dot.gov/engineering/hydraulics/software/toolbox404.cfm

There are three main components of total scour at a bridge site. They are Long-term Aggradation and Degradation, Contraction Scour, and Local Scour. In addition, lateral migration of the stream must be assessed when evaluating total scour at substructure units. Contraction and local scour will be evaluated in the context of clear-water and live bed scour conditions. In most of the methods for determining individual scour components, hydraulic characteristics at the approach section are required. The approach section should be understood as the cross section located approximately one bridge length upstream of the bridge opening.

8.3.2.7.1 Live Bed and Clear Water Scour

Clear-water scour occurs when there is insignificant or no movement (transport) of the bed material by the flow upstream of the crossing, but the acceleration of flow and vortices created by the piers or abutments causes the bed material in the vicinity of the crossing to move.

Live-bed scour occurs when there is significant transport of bed material from the upstream reach into the crossing.

8.3.2.7.2 Long-term Aggradation and Degradation

Aggradation is the deposition of eroded material in the stream from the upstream watershed. Degradation is the scouring (removal) of the streambed resulting from a deficient supply of sediment. These are subtle long term streambed elevation changes. These processes are natural in most cases. However, unnatural changes like dam construction or removal, as well as urbanization may cause Aggradation and Degradation. Excellent reference on this subject and the geomorphology of streams is the FHWA publication *Highways in the River Environment*[17], *HEC-18, Evaluating Scour at Bridges*[14], and *HEC-20, Stream Stability at Highway Structures*[15].

8.3.2.7.3 Contraction Scour

Generally, Contraction scour is caused by bridge approaches encroaching onto the floodplain and decreasing the flow area resulting in an increase in velocity through a bridge opening. The higher velocities are able to transport sediment out of the contracted area until an equilibrium is reached. Contraction scour can also be caused by short term changes in the downstream water surface elevation, such as bridges located on a meander bend or bridges located in the backwater of dams with highly fluctuating water levels. See 8.5 reference (14) & (15) for discussion and methods of analysis. If a pressure flow condition exists at the bridge opening, then vertical contraction scour must be evaluated. Reference HEC-18 for a description of the method used to estimate this scour component.

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 2)	
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 3)			
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 4)		
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	$3 > H \geq 0.6$	1.1
Medium Dunes	$9 > H \geq 3$	1.2 to 1.1
Large Dunes	$H \geq 9$	1.3

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

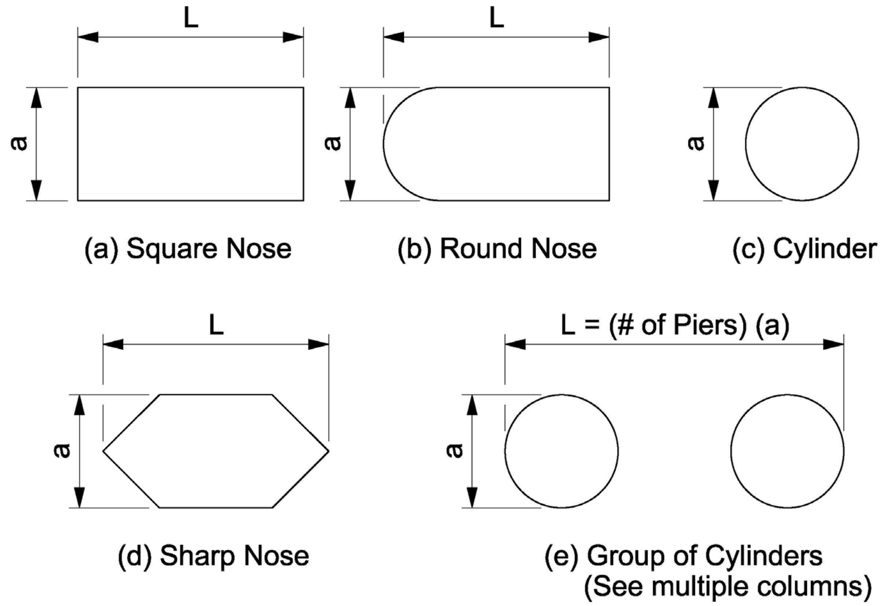


Figure 8.3-5
Common Pier Shapes

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 equations are more physically representative of the abutment scour process, greater confidence can be placed in the scour depths resulting from the NCHRP approach.

8.3.2.7.5 Design Considerations for Scour

Provide adequate free board (2 feet desirable) to prevent occurrences of pressure flow conditions.

Pier foundation elevations on floodplains should be designed considering the potential of channel or thalweg migration over the design life of the structure.

Align all substructure units and especially piers with the direction of flow. Improper alignment may significantly increase the magnitude of scour.

Piers in the water should have a rounded or streamline nose to reduce turbulence and related scour potential.

Spill-through (sloping) abutments are less vulnerable to scour than vertical wall abutments.

8.3.2.8 Select Bridge Design Alternatives

In most design situations, the “proposed bridge” design will be based on the various pertinent design factors discussed in 8.3.1. They will dictate the final selection of bridge length, abutment design, superstructure design and approach roadway design. The Hydraulic/Site report should adequately document the site characteristics, hydrologic and hydraulic calculations, as well as the bridge type and size alternatives considered. See 8.6 Appendix 8-A for a sample check list of items that need to be included in the Hydraulic/Site Report.

8.4 Hydraulic Design of Box Culverts

Box culverts are an efficient and economical design alternative for roadway stream crossings with design discharges in the 300 to 1500 cfs range. As a general guide culvert pipes are best suited for smaller discharge values while bridges are better suited for larger values. Although multi-cell box culverts are designed for larger discharges, the larger size culverts tend to lose the hydraulic and economic advantage over bridges. The following subsections discuss the design considerations and hydraulic design procedures for box culverts.

8.4.1 Hydraulic Design Factors

As in the hydraulic design of bridges, several hydraulic factors dictate the design of both the culvert and approach roadway. The critical hydraulic factors for design considerations are:

8.4.1.1 Economics

The best economics for box culvert design are realized with the culvert flowing full and producing a reasonable headwater depth (HW) within the boundary of other hydraulic and roadway design constraints.

For long box culverts, particularly on steep slopes, considerable savings can be realized by incorporating an improved inlet design known as "Tapered Inlets". The improved efficiency of the inlet where the inlet controls the headwater, will allow for design of a smaller culvert barrel. See 8.5 reference (13) for discussion on "Tapered Inlets".

8.4.1.2 Minimum Size

If the highway grade permits, a minimum five foot box culvert height is desirable for clean-out purposes.

8.4.1.3 Allowable Velocities and Outlet Scour

Generally, for velocities under 10 fps no riprap is needed at the discharge end of a box culvert, although close examination of local soil conditions is advisable.

For outlet velocities from 10-14 fps heavy riprap shall be used extending 15 to 35 feet from the end of the culvert apron.

For velocities greater than 14 fps energy dissipators should be considered. These are the most expensive means of end protection. See 8.4.2.7 for the hydraulic design of energy dissipators.

When heavy riprap is used it is carried up the slopes around the ends of the outlet apron to an elevation at mid-length of apron wing.

8.4.1.4 Roadway Overflow

See 8.3.1.2.

8.4.1.5 Culvert Skew

See 8.3.1.3.

8.4.1.6 Backwater and Highwater Elevations

The "Highwater elevation" commonly referred to as headwater for culverts, is the backwater created at the upstream end of the culvert. Although culverts are more hydraulically efficient and economical when flowing under a reasonable headwater, several factors shall be considered in determining an allowable highwater elevation. For further discussion see Section 8.3.1.4.

8.4.1.7 Debris Protection

Debris protection is provided where physical study of the drainage area indicates considerable debris collection. Where used, structural design of debris protection features should be part of the culvert design. The box culvert survey report must justify the need for protection. Sample debris protection devices are presented in the FHWA publication, *Hydraulic Engineering Circular No. 9, Debris Control Structures, Evaluation and Countermeasures.* See 8.5 reference (18).

8.4.1.8 Anti-Seepage Collar

Anti-seepage collars are used to prevent the movement of water along the outside of the culvert and the failure by piping of the fill next to the culvert. They are used in sandy fills where the culvert has a large headwater.

Collars are located at the midpoint and upstream quarter point on long box culverts. If only one collar is used, it is located far enough from the inlet to intercept the phreatic (zero pressure) line to prevent seepage over the top of the collar. See 8.5 reference (19).

A typical collar is shown in Figure 8.4-1 and is applicable to all single and twin box structures.

An alternate method of preventing seepage is to use a minimum one foot thick impervious soil blanket around the culvert inlet extending five feet over undisturbed embankment. The same effect can be obtained by designing seepage protection into the endwalls.

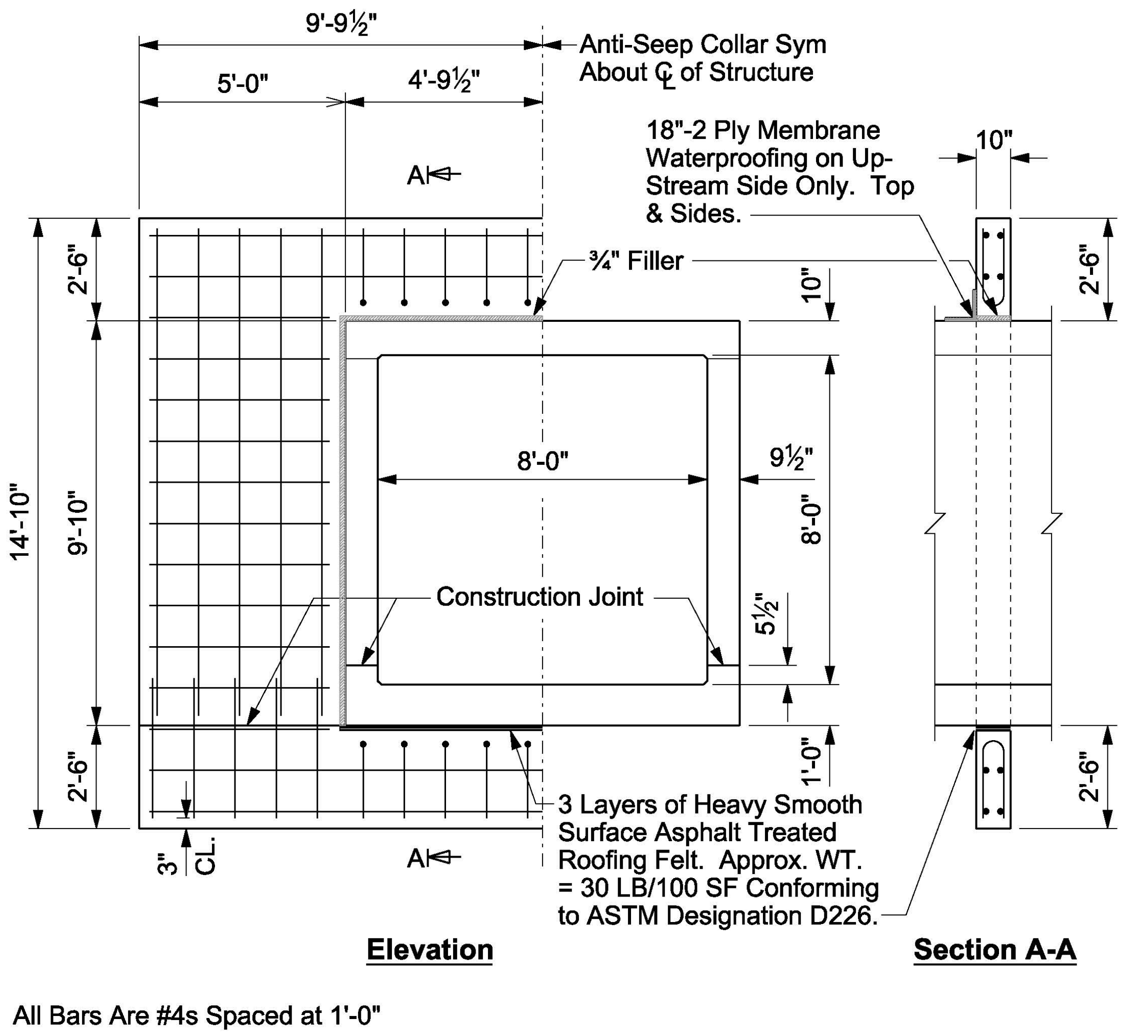


Figure 8.4-1
Anti-Seepage Collar

8.4.1.9 Weep Holes

The need for weep holes should be investigated for clay type soils with high fills, and should be eliminated in other cases.

If weep holes are necessary, alternate layers of fine and coarse aggregate are placed around the holes starting with coarse aggregate next to the hole.

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 DHS 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 “Ke” 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.

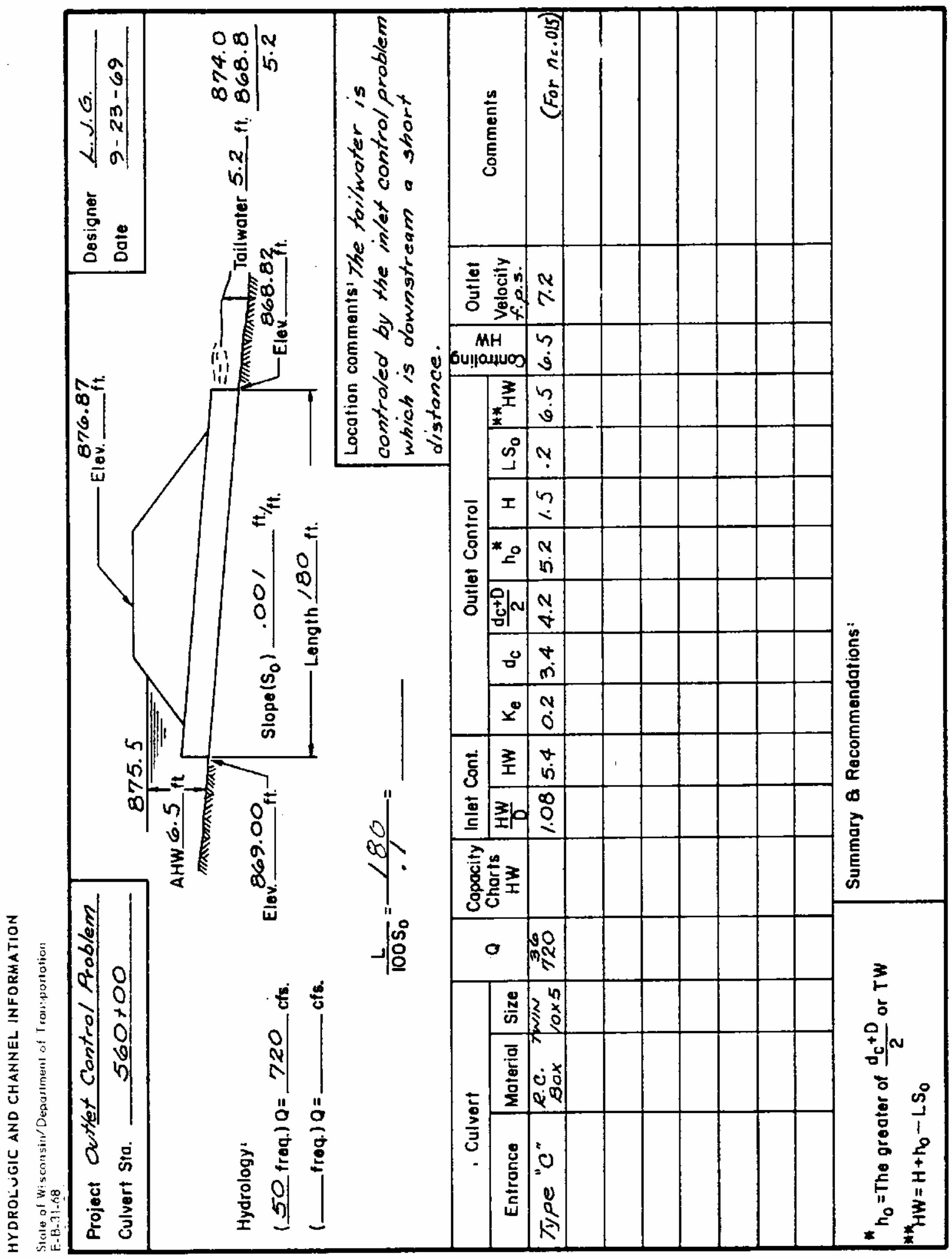
   Check for Inlet Control: For a Q/B value of 36 and a twin 10 x 5 box with type “C” inlet; HW/D = 1.08 from Figure 8.4-3.

   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.



Culvert Entrance	Culvert Material	Culvert Size	Q	Capacity Charts HW	Inlet Cont. HW/D	Inlet Cont. HW	Outlet Control K_e	d_c	$\frac{d_c+D}{2}$	h_o*	H	LS_o	**HW	Controlling HW	Outlet Velocity f.p.s.	Comments
Type "C"	R.C. Box	TWIN 10x5	36 720		1.08	5.4	0.2	3.4	4.2	5.2	1.5	.2	6.5	6.5	7.2	(For n=.015)

Figure 8.4-2
Culvert Computation Form

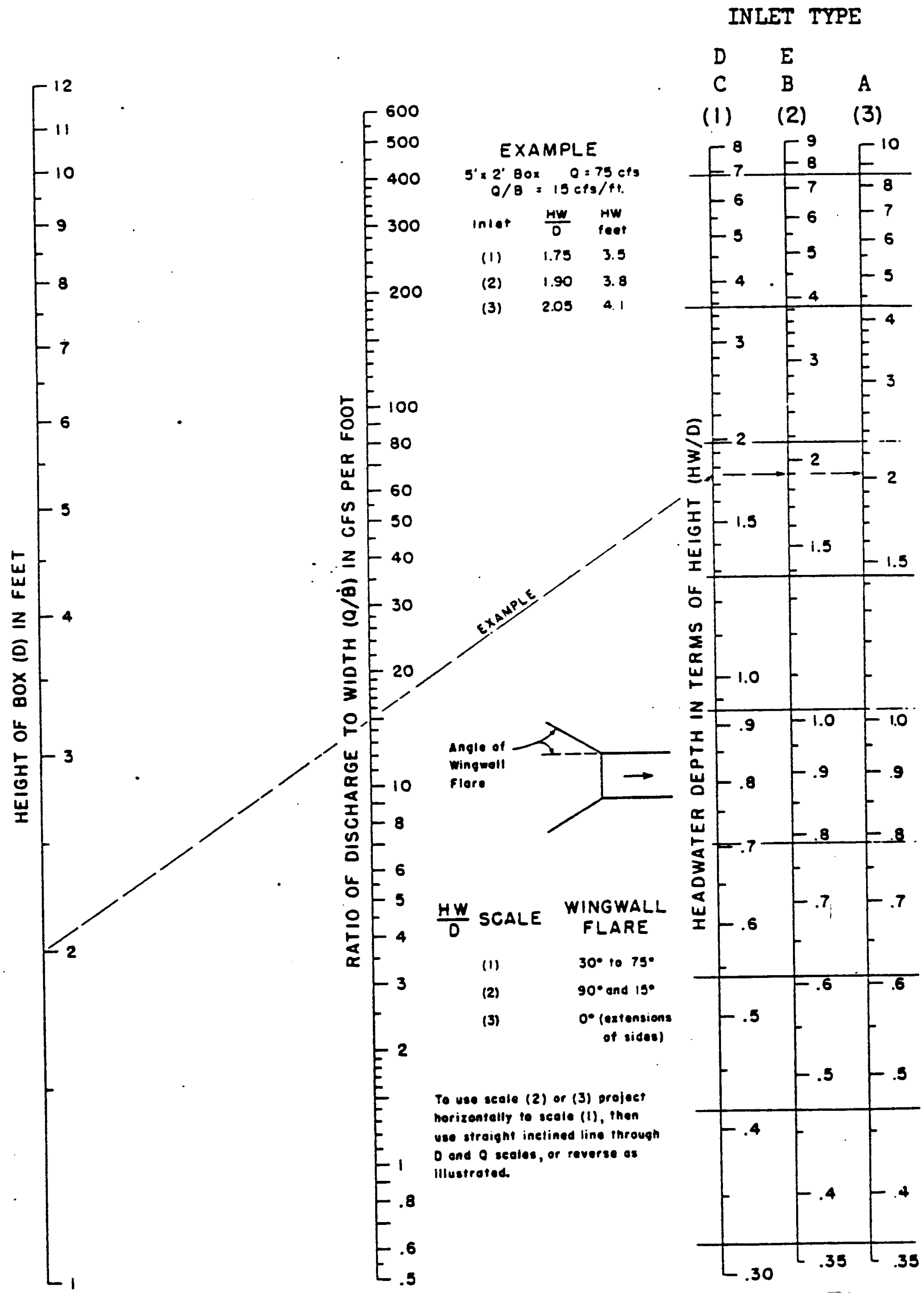


Figure 8.4-3

Headwater Depth for Box Culverts with Inlet Control

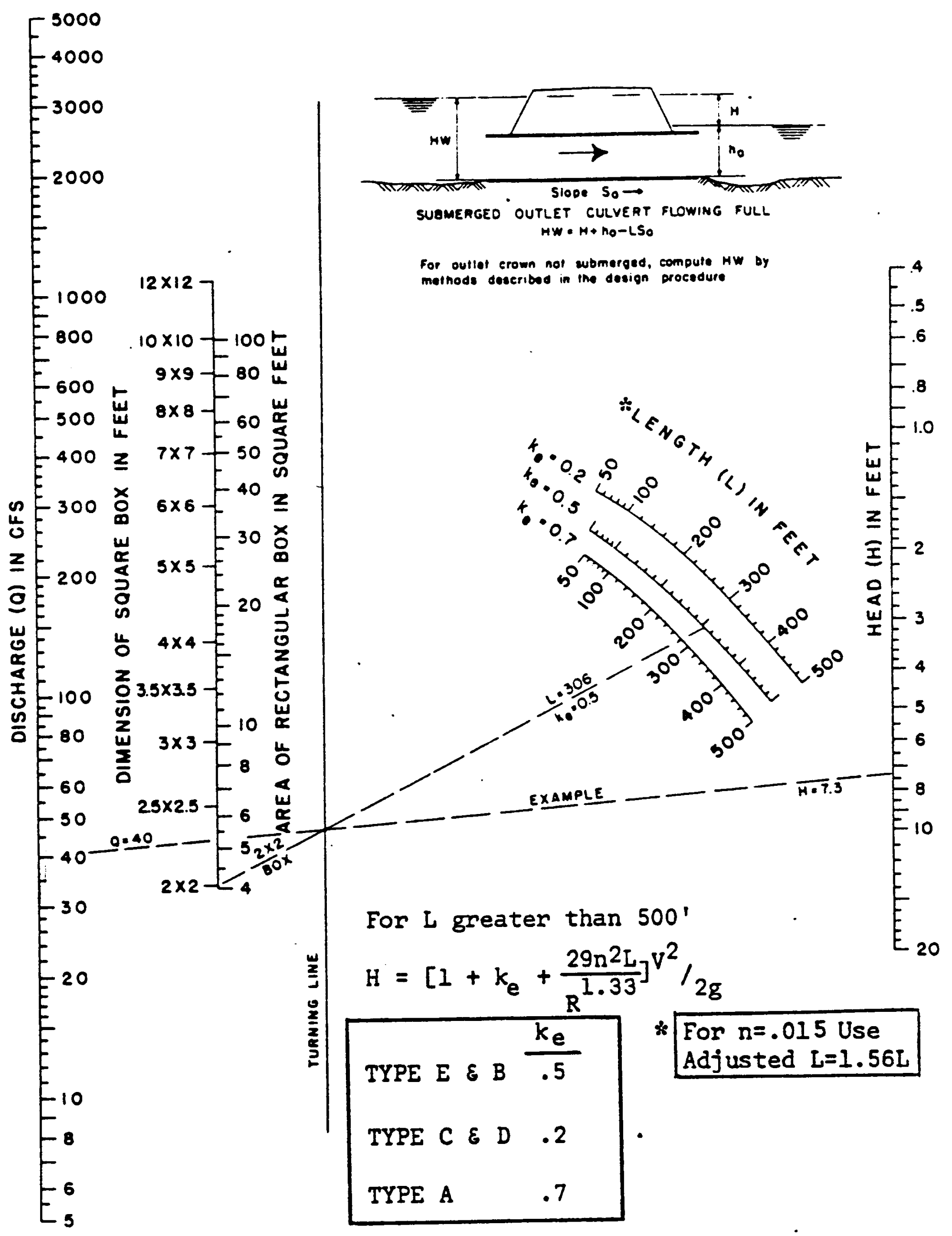
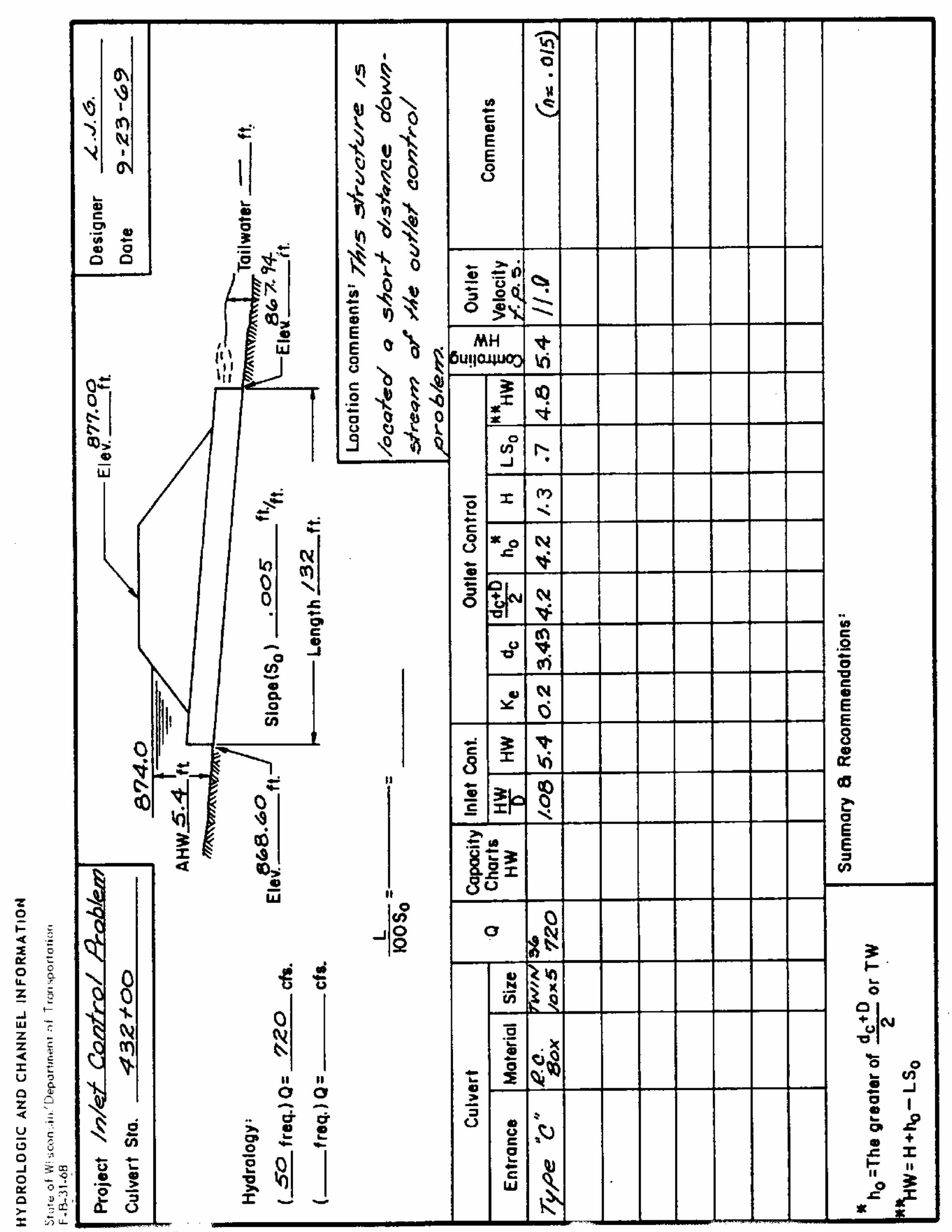


Figure 8.4-4

Head for Concrete Box Culverts Flowing Full, n = 0.012



Culvert			Q	Capacity Charts HW	Inlet Cont.		Outlet Control								Controlling HW	Outlet Velocity f.p.s.	Comments
Entrance	Material	Size			$\frac{HW}{D}$	HW	K_e	d_c	$\frac{d_c+D}{2}$	h_o*	H	LS_o	**HW				
Type "C"	R.C. Box	TWIN 10x5	36 720		1.08	5.4	0.2	3.43	4.2	4.2	1.3	.7	4.8		5.4	11.0	(n= .015)

* h_o = The greater of $\frac{d_c+D}{2}$ or TW

** $HW = H + h_o - LS_o$

Summary & Recommendations:

Figure 8.4-5
Culvert Computation Form

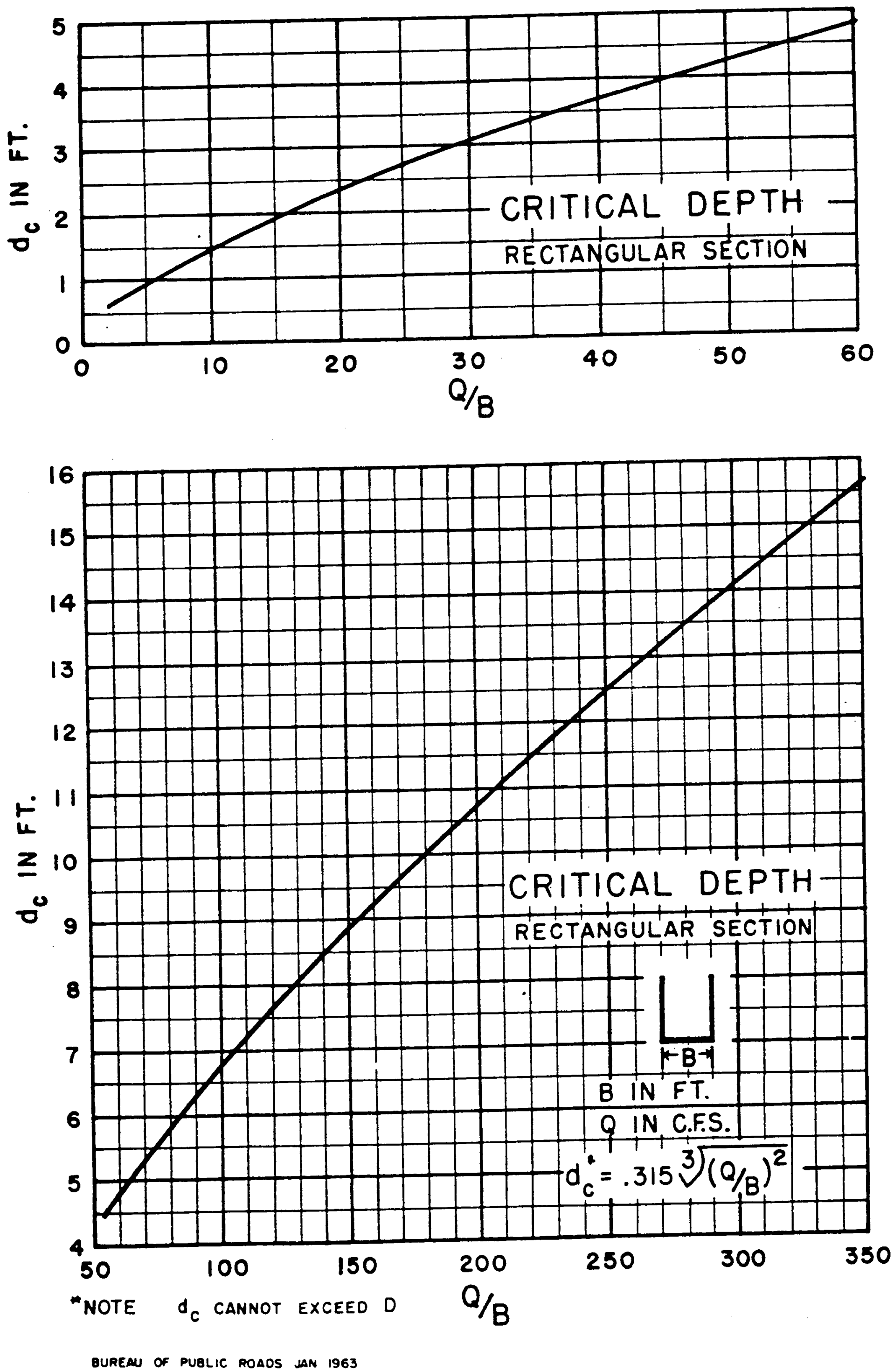


Figure 8.4-6
Critical Depth – Rectangular Section

8.4.2.6 Roadway Overflow

See 8.3.2.6.

8.4.2.7 Outlet Scour and Energy Dissipators

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

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

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

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

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

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

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

8.4.2.7.1 Drop Inlet.

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

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

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

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

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

and

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

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

1. Correction for head H/W (Table 8.4-1)
2. Correction for box-inlet shape B/W. (Table 8.4-2)
3. Correction for approach channel width W_c/L (Table 8.4-3).

   Where: W_c = approach channel width = Area/Depth
4. Correction for dike effect X/W (Table 8.4-4)

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

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

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

Sample computations are shown in 8.4.2.7.1.1.

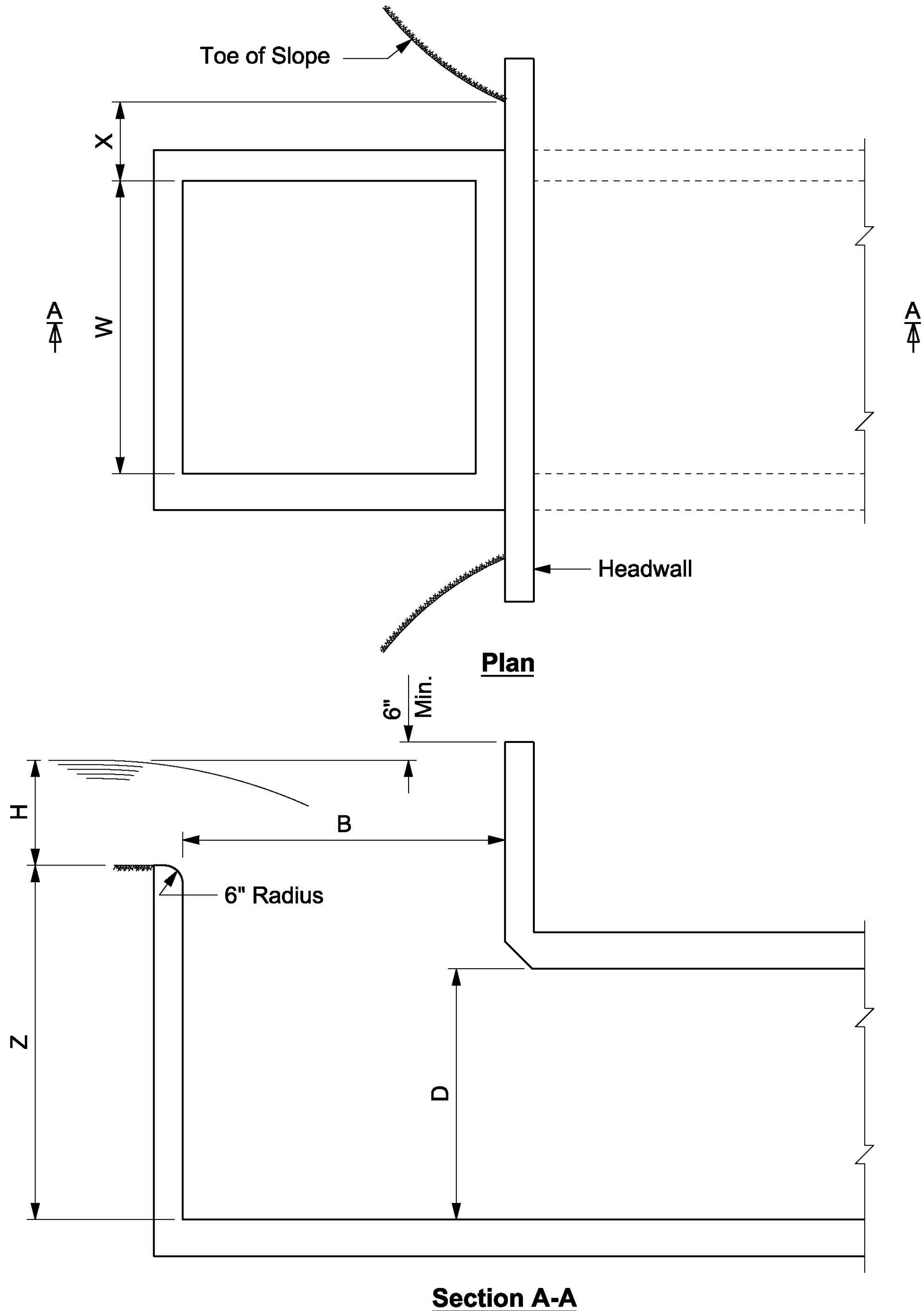


Figure 8.4-7
Box Drop Inlet

H/W	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	--	--	--	--	--	0.76	0.8	0.82	0.84	0.86
0.1	0.8	0.88	0.89	0.9	0.91	0.91	0.92	0.92	0.93	0.93
0.2	0.93	0.94	0.94	0.95	0.95	0.95	0.95	0.96	0.96	0.96
0.3	0.97	0.97	0.97	0.97	0.98	0.98	0.98	0.98	0.98	0.98
0.4	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	1
0.5	1	1	1	1	1	1	1	1	1	1
0.6	1	--	--	--	--	--	--	--	--	--
Correction is 1.00 when H/W exceeds 0.6										

Table 8.4-1
Correction for Head
(Control at Box-Inlet Crest)

B/W	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.98	1.01	1.03	1.03	1.04	1.04	1.03	1.02	1.01	1.01
1	1	0.99	0.99	0.98	0.98	0.98	0.97	0.97	0.96	0.96
2	0.96	0.96	0.95	0.95	0.95	0.95	0.95	0.95	0.94	0.94
3	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.93	0.93
4	0.93	--	--	--	--	--	--	--	--	--

Table 8.4-2
Correction for Box-Inlet Shape
(Control at Box-Inlet Crest)

Wc/L	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0.09	0.18	0.27	0.35	0.44	0.53	0.62	0.71	0.8
1	0.84	0.87	0.9	0.92	0.93	0.94	0.95	0.96	0.97	0.97
2	0.98	0.98	0.99	0.99	0.99	0.99	1	1	1	1
3	1	--	--	--	--	--	--	--	--	--
Correction is 1.00 when Wc/L exceeds 3.0										

Table 8.4-3
Correction for Approach-Channel Width
(Control at Box-Inlet Crest)

B/W	X/W						
	0	0.1	0.2	0.3	0.4	0.5	0.6
0.5	0.9	0.96	1	1.02	1.04	1.05	1.05
1	0.8	0.88	0.93	0.96	0.98	1	1.01
1.5	0.76	0.83	0.88	0.92	0.94	0.96	0.97
2	0.76	0.83	0.88	0.92	0.94	0.96	0.97

Table 8.4-4
Correction for Dike Effect
(Control at Box-Inlet Crest)

8.4.2.7.1.1 Drop Inlet Example Calculations

Given:

Q = 420 cfs through single 9'x6' box

H = 4.4' in a 27 ft. wide channel

Drop = 5 ft

Assume:

B = $\frac{W}{2} = 4.5$

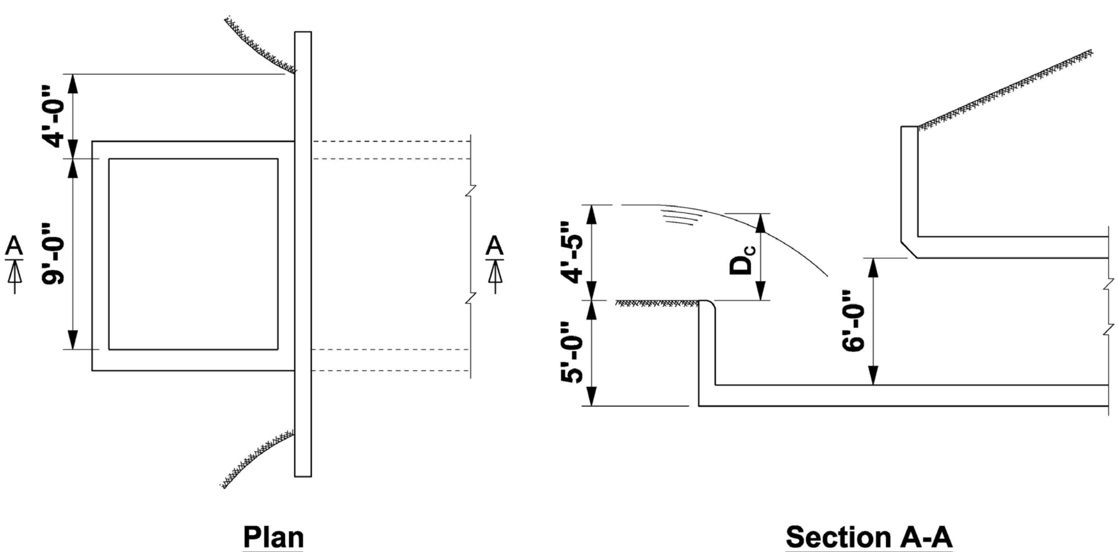


Figure 8.4-8
Drop Inlet Example

Control at inlet crest: $L = \dfrac{Q}{3.43 \cdot H^{3/2}}$

Corrections:

1. $\dfrac{H}{W} = \dfrac{4.4}{9} = 0.49 \Rightarrow 1.00$

2. $\dfrac{B}{W} = \dfrac{4.5}{9} = 0.5 \Rightarrow 1.04$

3. $\dfrac{W_c}{L} = \dfrac{27}{9 + 2(4.5)} = \dfrac{27}{18} = 1.50 \Rightarrow 0.94$

4. $\dfrac{X}{W} = \dfrac{4.0}{9.0} = 0.44 \Rightarrow 1.04$

Total Correction = 1.00 x 1.04 x 0.94 x 1.04 = 1.02

$$L = \frac{420}{1.02 \cdot 3.43 \cdot 4.4^{3/2}} = \frac{420}{1.02 \cdot 3.43 \cdot 9.23} = 13.01 < (2B + W) = 18 \Rightarrow OK$$

$$d_c = \sqrt[3]{\frac{Q^2}{L^2 g}} = \left(\frac{17.64x10^4}{3.24 \cdot 3.22x10^3} \right)^{1/3} = 16.85^{1/3} = 2.56$$

HW must be less than Z+d_c to prevent submerged weir. With inlet control, from Figure 8.4-3:

$$\frac{HW}{D} = 1.19$$

HW = 1.19x6 = 7.14

7.14 < (5+2.56) = 7.56, therefore weir controls

8.4.2.7.2 Drop Outlets

This generalized design is applicable to relative heights of fall ranging from 1.0 y/d_c to 15 y/d_c and to crest lengths greater than 1.5 d_c. Here y is the vertical distance between the crest and the stilling basin floor and d_c is the critical depth of flow.

$d_c = 0.315[(Q/B)^2]^{1/3}$

Referring to Figure 8.4-10 and Figure 8.4-9, this design uses the following formulas:

1. The minimum length L_b of the stilling basin is:

$X_a + X_b + X_c = X_a + 2.55\ d_c$

a. The distance X_a from the headwall to the point where the surface of the upper nappe strikes the stilling basin floor is solved graphically in Figure 8.4-9.

b. The distance X_b from the point at which the surface of the upper nappe strikes the stilling basin floor to the upstream face of the floor blocks is:

$X_b = 0.8\ d_c$

c. The distance X_c, between the upstream face of the floor blocks and the end of the stilling basin is:

$X_c \geq 1.75\ d_c$

2. The floor blocks are proportioned as follows:

a. The height of the floor blocks is:

$0.8\ d_c$

b. The width and spacing of the floor blocks are approximately:

$0.4\ d_c$

A variation of $\pm$ 0.15 d_c from this limit is permissible.

c. The floor blocks are square in plan.

d. The floor blocks occupy between 50 and 60 percent of the stilling basin width.

3. The height of the end sill is:

$0.4\ d_c$

4. The sidewall height above the tailwater level is:

$0.85\ d_c$

5. The minimum height d_2, of the tailwater surface above the floor of the stilling basin is:

$d_2 = 2.15\ d_c$

In cases where the approach velocity head is greater than 1/3 of the specific head (velocity head + elevation head), X_a is checked by the formula below and the greater X_a value is used.

$$X_a^{\,2} = \left(\frac{2 \cdot V^2}{g}\right) \cdot y_1$$

Where:

y_1 = top of water at crest

V = velocity of approach

Sometimes high values of d_c become unworkable, resulting in a need for large drops, high end sills and floor blocks. To prevent this d_c may be reduced by flaring the end of the barrel. The flare angle is approximately 150/V where V is the velocity at the beginning of the taper.

Sample computations are shown in 8.4.2.7.2.1.

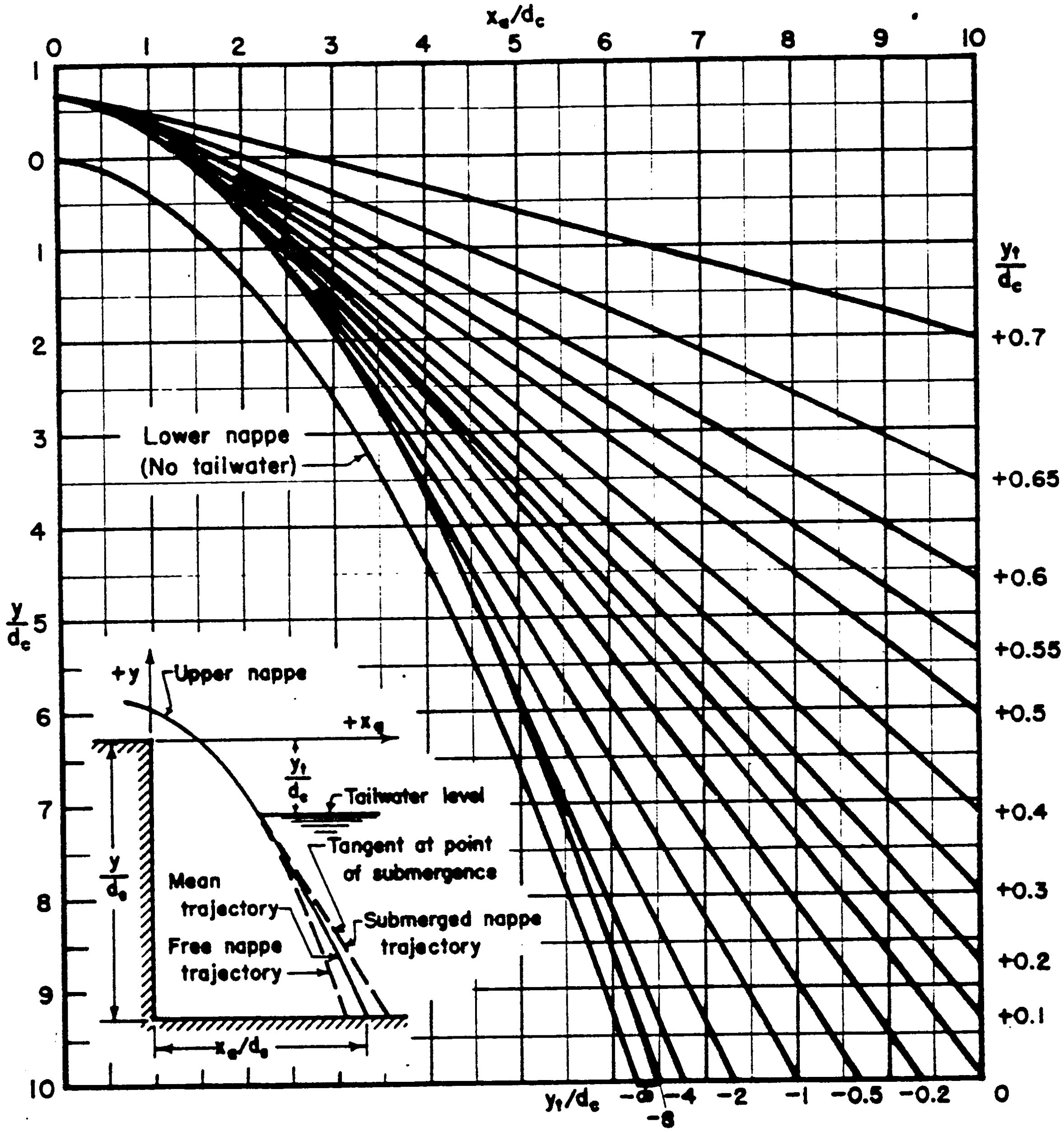
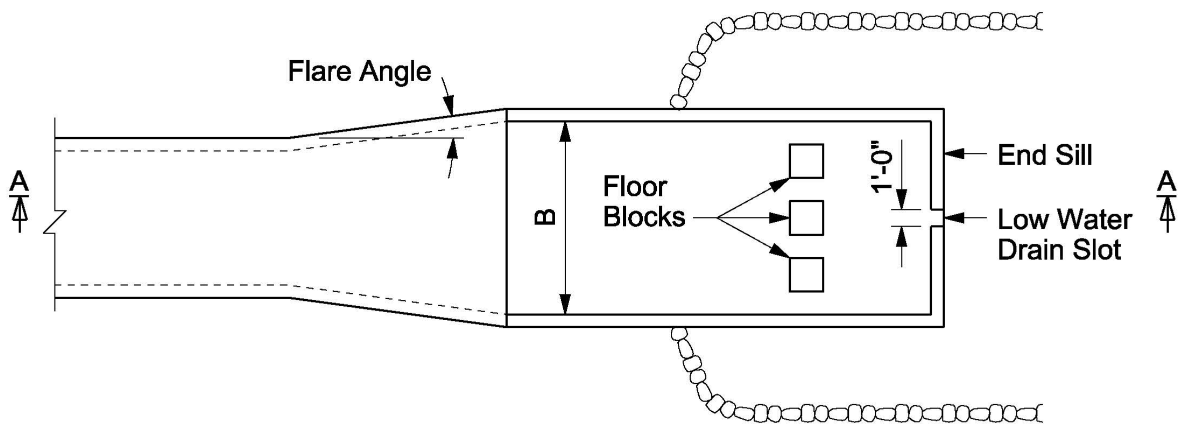
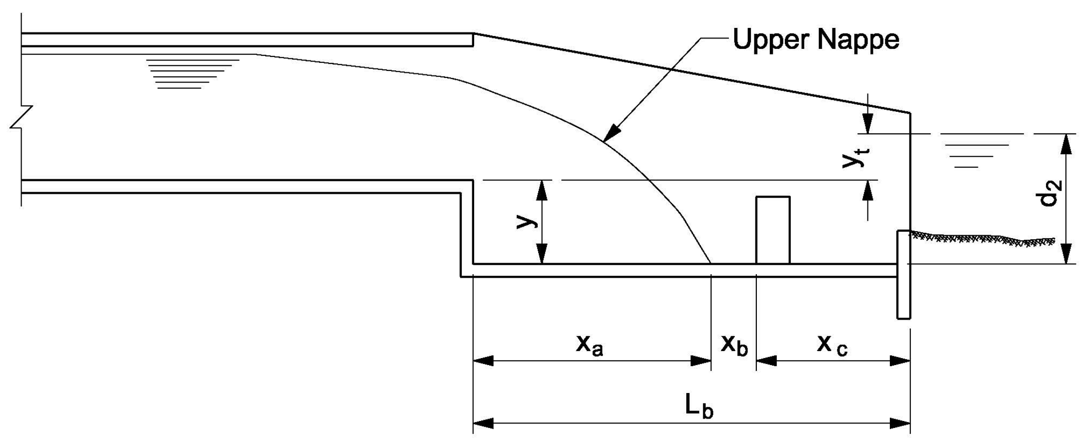


Figure 8.4-9
Design Chart for Determination of "X_a"



Plan



Section A-A

Figure 8.4-10
Straight Drop Outlet Stilling Basin

8.4.2.7.2.1 Drop Outlet Example Calculations

Given:

Q	=	800 cfs through single 8'x8' box
V	=	13.5 fps in the box
Drop	=	5 ft

Depth = 7.5 ft

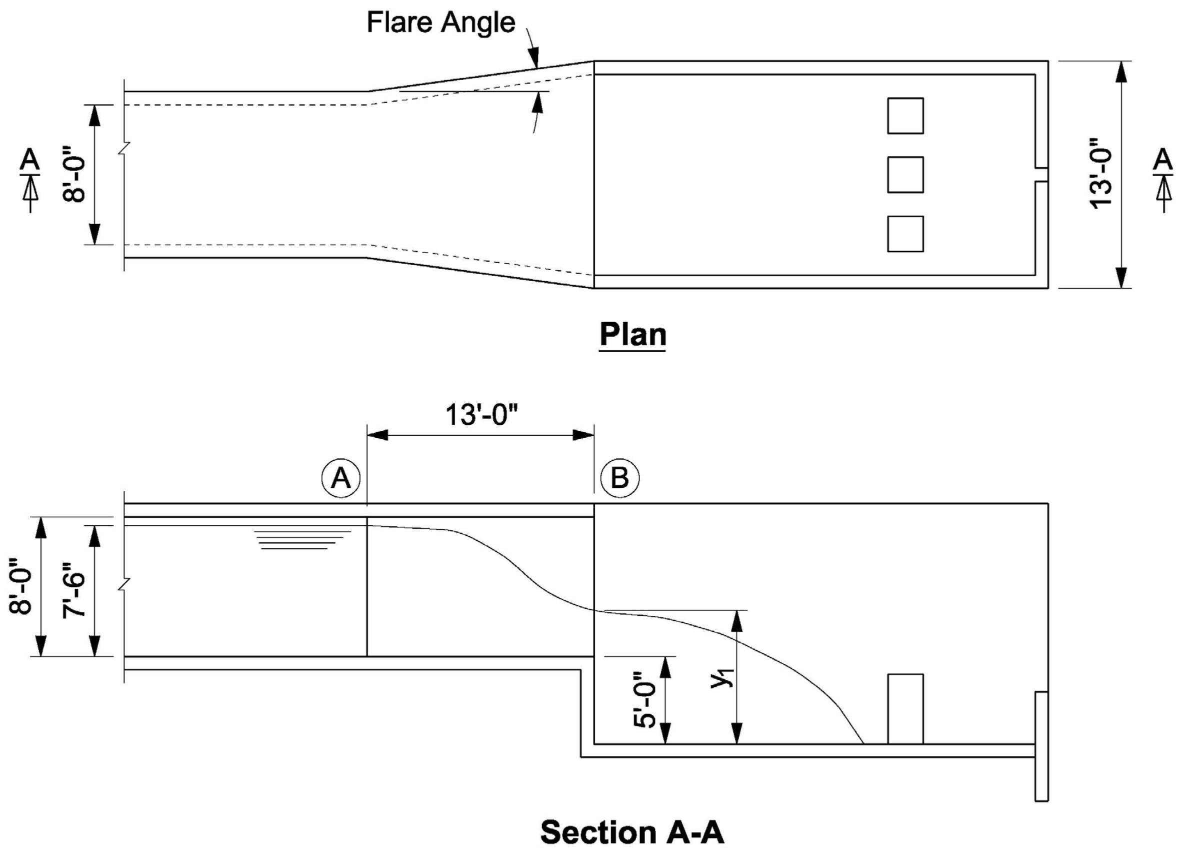


Figure 8.4-11
Drop Outlet Example

Assumptions:

- That the specific head of "A" is approximately equal to the specific head at "B". Therefore, the elevation head + velocity head at "A" = elevation head + velocity head at "B".
- The end sill height should be less than or equal to 2'-0".

If the drop were placed at "A":

$$d_c = 0.315 \cdot \sqrt[3]{\left(\frac{Q}{B}\right)^2} = 0.315 \cdot (100)^{2/3} = 6.78$$

And end sill = 0.4dc = 2'-9" which exceeds 2'-0, therefore flare outlet.

To obtain a 2'-0" sill, set d_c = 2'-0"/0.4 = 5 ft

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

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 height required to cause the hydraulic jump,
F_1	=	Froude number = $v_1 /(gy_1)^{1/2}$
g	=	acceleration of gravity,
y_1	=	velocity at beginning of jump.

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

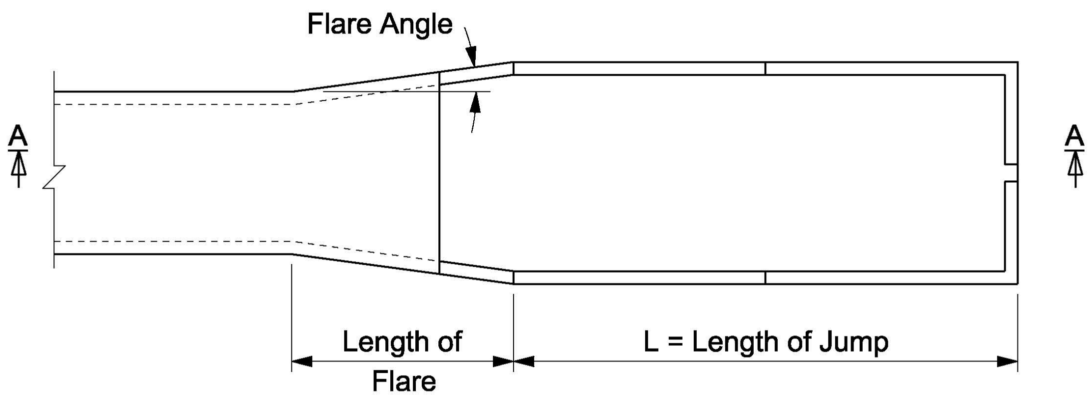
Length of jump is assumed to be 6 times the depth change (y_2-y_1).

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.

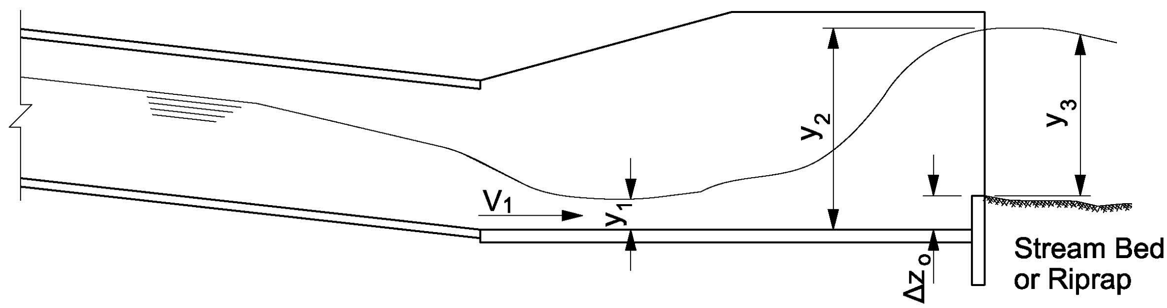
The problem of scour on the downstream side of the end sill can be overcome by providing riprap in the stream bottom. If riprap is used, it starts from the top of the sill at a maximum slope of 6:1 up from end sill to original streambed. If no riprap is used, the streambed begins at the top of the end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



Plan



Section A-A

Figure 8.4-12
Hydraulic Jump Stilling Basin

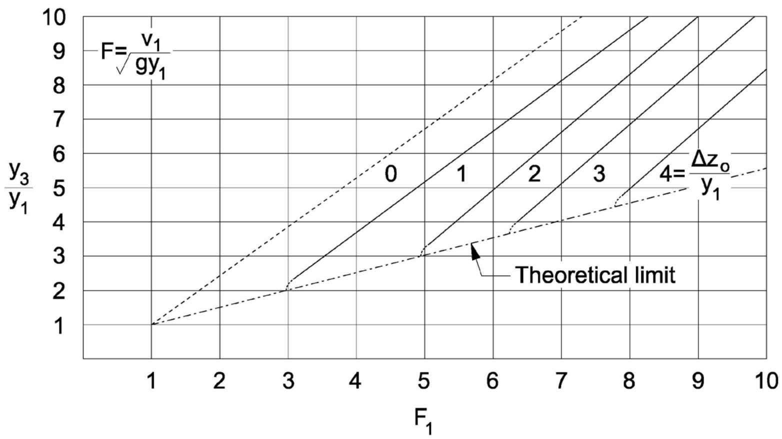


Figure 8.4-13
Characteristics of a Hydraulic Jump at an Abrupt Rise

8.4.2.7.3.1 Hydraulic Jump Stilling Basin Example Calculations

Given:

A discharge of 600 cfs flows through a 7'x6' box culvert at 16 fps and a depth of 5.8'. Normal tailwater depth in the outlet channel is 5.0 feet.

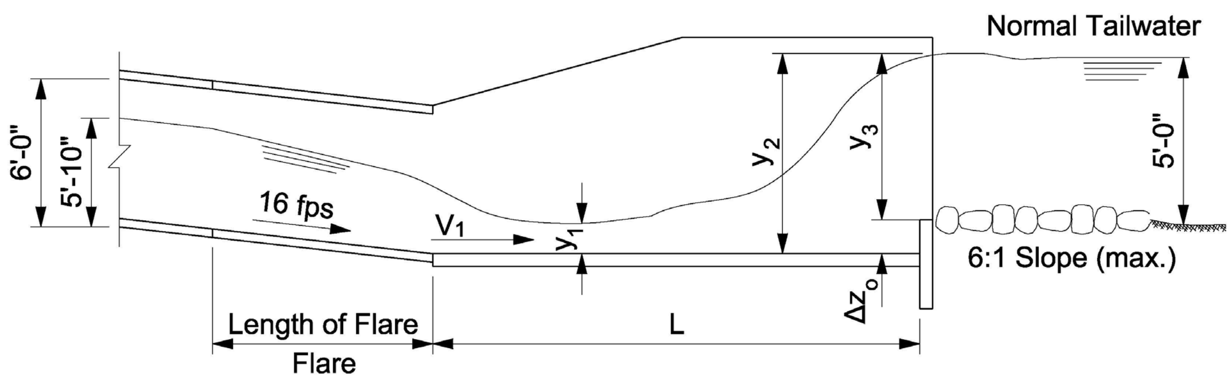


Figure 8.4-14
Hydraulic Jump Stilling Basin Example

Flare of wings $= \dfrac{150}{16} \approx 9^0$

$$H = 5.8 + \frac{16^2}{2 \times 32.2} = 5.8 + 3.975 = 9.775$$

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 x width x 22.1, width = 12.36

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, $\Delta Z_o/y_1 = 0.5$

ΔZ_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. U. S. Geological Survey, *Flood-Frequency Characteristics of Wisconsin Streams*. Water-Resources Investigations Report 03-4250, 2003. This report can be found on the USGS web site using the following link:

   http://wi.water.usgs.gov/publications/flood/currentreport.html

3. U. S. Geological Survey, *Guidelines for Determining Flood Flow Frequency, Bulletin #17B* Revised September 1981, Editorial Corrections, March 1982.

4. U.S. Department of Agriculture, Soil 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 4.0, March 2008.

8. U.S. Army Corps of Engineers, *HEC-RAS River Analysis System Hydraulic Reference Manual* (CPD-69), Hydrologic Engineering Center, Davis CA, Version 4.0, March 2008.

9. U.S. Army Corps of Engineers, *HEC-RAS River Analysis System Applications Guide* (CPD-70), Hydrologic Engineering Center, Davis CA, Version 4.0, March 2008.

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.

13. U.S. Department of Transportation, FHWA, Hydraulic Design Series (HDS), Number 5, *Hydraulic Design of Highway Culverts*, September 2001, Revised May 2005.

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,* Report No. FHWA-HI-90-016, February 1990.

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 Dam*, 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, recommended scour prevention measures and assigned scour code. (Scour calculations shall be submitted with the hydraulic site report).

- 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, but will be updated in the future.

Code	Title	Year	Publication #	NTIS #
HDS 01	Hydraulics of Bridge Waterways	1978	FHWA-EPD-86-101	PB86-181708
HDS 02	Highway Hydrology Second Edition	2002	FHWA-NHI-02-001	
HDS 03	Design Charts for Open-Channel Flow	1961	FHWA-EPD-86-102	PB86-179249
HDS 04	Introduction to Highway Hydraulics	2001	FHWA-NHI-01-019	
HDS 05	Hydraulic Design of Highway Culverts	2005	FHWA-NHI-01-020	
HDS 06	River Engineering for Highway Encroachments	2001	FHWA-NHI-01-004	
HEC 09	Debris Control Structures Evaluation and Countermeasures	2005	FHWA-IF-04-016	
HEC 11	Design of Riprap Revetment	1989	FHWA-IP-89-016	PB89-218424
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 17	The Design of Encroachments on Flood Plains Using Risk Analysis	1981	FHWA-EPD-86-112	PB86-182110
HEC 18	Evaluating Scour at Bridges, Fifth Edition	2012	FHWA-HIF-12-003	
HEC 20	Stream Stability at Highway Structures Fourth Edition	2012	FHWA-NIF-12-004	
HEC 21	Bridge Deck Drainage Systems	1993	FHWA-SA-92-010	PB94-109584
HEC 22	Urban Drainage Design Manual Second Edition	2001	FHWA-NHI-01-021	
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 (cover)	2001	FHWA-NHI-01-007	
HEC 24	Highway Stormwater Pump Station Design	2001	FHWA-NHI-01-007	
HEC 25	Tidal Hydrology, Hydraulics, and Scour at Bridges	2004	FHWA-NHI-05-077	
HEC 25	Highways in the Coastal Environment - 2nd edition	2008	FHWA-NHI-07-096	
HRT	Assessing Stream Channel Stability at Bridges in Physiographic Regions	2006	FHWA-HRT-05-072	
HRT	Effects of Inlet Geometry on Hydraulic Performance of Box Culverts	2006	FHWA-HRT-06-138	
HRT	Junction Loss Experiments: Laboratory Report	2007	FHWA-HRT-07-036	
HRT	Hydraulics Laboratory Fact Sheet	2007	FHWA-HRT-07-054	

Code	Title	Year	Publication #	NTIS #
Other	Geosynthetic Design and Construction Guidelines	1995	FHWA-HI-95-038	PB95-270500
Other	Underwater Evaluation And Repair of Bridge Components	1998	FHWA-DP-98-1	
Other	Best Management Practices for Erosion and Sediment Control	1995	FHWA-FLP-94-005	
Other	Underwater Inspection of Bridges	1980	FHWA-DP-80-1	
Other	Culvert Management Systems User Manual	2001	FHWA-02-001	
Other	FHWA Hydraulics Library on CD-ROM FHWA Hydraulics Library on CD-ROM (Updated Browser)	2002		
Other	Hydraulic Performance of Curb and Gutter Inlets	1999	FHWA-KU-99-1	
Other	Culvert Management Systems Source Code	2001		
Other	NCHRP Report 25-25 (04) Environmental Stewardship Practices, Procedures, and Policies for Highway Construction and Maintenance	2004		
Other	New England Transportation Consortium: Performance Specs for Wood Waste Materials as an Erosion Control Mulch and as a Filter Berm	2001	FHWA-NETC 25	
Other	Bridge Scour Protection Systems Using Toskanes	1994	FHWA-PA-94-012	PB95-266318
Other	Structural Design Manual for Improved Inlets and Culverts	1983	FHWA-IP-83-6	PB84-153485
Other	Culvert Inspection Manual	1986	FHWA-IP-86-2	PB87-151809
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	

Code	Title	Year	Publication #	NTIS #
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	
RD	Performance Curve for a Prototype of Two Large Culverts in Series Dale Boulevard, Dale City, Virginia	2001	FHWA-RD-01-095	
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	PB-2204-104690
RD	Enhanced Abutment Scour Studies For Compound Channels	2004	FHWA-RD-99-156	
RD	Field Observations and Evaluations of Streambed Scour at Bridges	2005	FHWA-RD-03-052	
RD	South Dakota Culvert Inlet Design Coefficients	1999	FHWA-RD-01-076	

Figure 8.7-1
FHWA Hydraulic Engineering Publications

FHWA Hydraulics Engineering Software		
Software	Title	Year
HY 7	Bridge Waterways Analysis Model (WSPRO)	2005
HY 7	WSPRO User's Manual (Version 061698) (pdf 2.1 MB)	1998
HY 8	Culvert Hydraulic Analysis Program, Version 7.0	2007
HDS 5	HDS 5 Hydraulic Design of Highway Culverts (pdf, 9.25 mb)	2001
HDS 5	HDS 5 Chart Calculator	2001
HY 11	Preliminary Analysis System for WSP	1989
HY 11	PAS USERS MANUAL (ISDDC)	1989
HY 11	Accuracy of Computed Water Surface Profiles (ISDDC)	1986
FESWMS	FESWMS (Version 3.1.5)	2003
FESWMS	FESWMS User's Manual	2003
HY 22	Visual Urban	2002
HY 22	HEC 22 - Urban Drainage Manual	2001
BRI-STARS	Bridge Stream Tube for Alluvial River Sim	2000
BRI-STARS	BRI-STARS Users Manual	2000
HYRISK	HYRISK Setup (zip, 13 mb)	2002
Hydraulics Software by Others		
Software	Title	Year
BCAP	Broken-back Culvert Analysis Program (Version 3.0)	2002
CAESAR	Cataloging And Expert evaluation of Scour risk And River stability at bridge sites	2001
CHL	Coastal & Hydraulics Laboratory USACE	
FishXing	Fish Passage through Culverts USFS	
HEC	Hydrologic Engineering Center USACE	
HyperCalc	HyperCalc Plus	2002
NSS	National Streamflow Statistics Program	
PEAKFQ	PEAKFQ	1995
SMS	Surface-Water Modeling System (SMS)	2001
StreamStats	StreamStats	
USGS	Water Resources Applications Software USGS	
WMS	Watershed Modeling System (WMS)	

Figure 8.7-2

FHWA Hydraulics Software List

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Table of Contents

11.1 General

11.1.1 Overall Design Process

The overall foundation support design process requires an iterative collaboration to provide cost-effective constructible substructures. Input is required from multiple disciplines including, but not limited to, structural, geotechnical and design. For a typical bridge design, the following four steps are required (see 6.2):

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

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

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

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

11.1.2 Foundation Type Selection

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

- Magnitude and direction of loads.

- Depth to suitable bearing material.

- Potential for liquefaction, undermining or scour.

- Frost potential.

11.3 Deep Foundations

When competent bearing soil is not present near the base of the proposed foundation, structure loads must be transferred to a deeper stratum by using deep foundations such as piles or drilled shafts (caissons). Deep foundations can be composed of piles, drilled shafts, micropiles or augered cast-in-place piles.

The primary functions of a deep foundation are:

- To transmit the load of the structure through a stratum of poor bearing capacity to one of adequate bearing capacity.
- To eliminate objectionable settlement.
- To transfer loads from a structure through erodible soil in a scour zone, to stable underlying strata.
- To anchor structures subjected to hydrostatic uplift or overturning forces.
- To resist lateral loads from earth pressures, as well as external forces.

11.3.1 Driven Piles

Deep foundation support systems have been in existence for many years. The first known pile foundations consisted of rows of timber stakes driven into the ground. Timber piles have been found in good condition after several centuries in a submerged environment. Several types of concrete piles were devised at the turn of the twentieth century. The earliest concrete piles were cast-in-place, followed by reinforced, precast and prestressed concrete piling. The requirement for longer piles with higher bearing capacity led to the use of concrete-filled steel pipe piles in about 1925. More recently, steel H-piles have also been specified due to ease of fabrication, higher bearing capacity, greater durability during driving and the ability to easily increase or decrease driven lengths.

11.3.1.1 Conditions Involving Short Pile Lengths

WisDOT policy generally requires piles to penetrate a minimum of 10 feet through the original ground. Concern exists that short pile penetration in foundation materials of variable consistency may not adequately restrain lateral movements of substructure units. Pile penetrations of less than 10 feet are allowed if prebored at least 3 feet into solid rock. If conditions detailed in the Site Investigation Report clearly indicate that minimum pile penetration cannot be achieved, preboring should be included as a pay quantity. If there is a potential that preboring may not be necessary, do not include it in the plan documents. Piles which are not prebored into rock must not only meet the 10-foot minimum pile penetration criteria but must also have at least 5 feet of penetration through material with a blow count of at least 7 blows per foot. Piling should be “firmly seated” on rock after placement in prebored holes. The annular space between the cored holes in bedrock and piling should then be filled with concrete. Some sites may require casing during the preboring operation. If casing is required, it should be clearly indicated in the plan documents. Refer to 11.3.1.6 for additional information on preboring.

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

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

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

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

11.3.1.2 Pile Spacing

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

WisDOT policy item:

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

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

Geotechnical Site Investigation Report must be used as a guide in determining the nominal geotechnical resistance for the pile.

Any structural strength contribution associated with the steel shell is neglected in driven CIP concrete pile design. Therefore, environmentally corrosive sites do not affect driven CIP concrete pile designs. An exception is that CIP should not be used for exposed pile bents in corrosive environments as shown in the *Facilities Development Manual,* Procedure 13-1-15.

Based on the above equation, current WisDOT practice is to design driven cast-in-place concrete piles for factored (ultimate structural) axial compression resistances as shown in Table 11.3-5. See 6.3.2.1 for the typical style of plan notes showing axial resistance as well as required driving resistance on plans. **If less than the maximum axial resistance is required by design, state only the required corresponding driving resistance on the plans.** The minimum shell thickness is 0.219 inches for straight steel tube and 0.1793 inches for fluted steel shells, unless otherwise noted in the Geotechnical Site Investigation Report and stated in the project plans. Exposed piling (e.g. open pile bents) should not be less than 12 inches in diameter.

When cobbles or other difficult driving conditions are present, the minimum wall thickness for steel shells of driven cast-in-place concrete piles should be increased to 0.25 inches or thicker to facilitate driving without damaging the pile. A drivability analysis should be completed in design, to determine the required wall thickness based on site conditions and an assumed driving equipment.

Driven cast-in-place concrete pile is generally the most favorable displacement pile type since inspection of the steel shell is possible prior to concrete placement and more reliable control of concrete placement is attainable.

11.3.1.12.2.2 Precast Concrete Piles

Precast concrete pile can be divided into two primary types – reinforced concrete piles and prestressed concrete piles. These piles have parallel or tapered sides and are usually of rectangular or round cross section. Since the piles are usually cast in a horizontal position, the round cross section is not common because of the difficulty involved in filling a horizontal cylindrical form. Because of the somewhat variable subsurface conditions in Wisconsin and the need for variable length piles, these piles are currently not used in Wisconsin.

11.3.1.12.3 Steel Piles

Steel pile generally consist of either H-pile or pipe pile types. Both open-end and closed-end pipe pile are used. Pipe piles may be left open or filled with concrete, and can also have a structural shape or reinforcement steel inserted into the concrete. Open-end pipe pile can be socketed into bedrock with preboring.

Steel pile is typically top driven at the pile butt. However, closed-end pipe pile can also be bottom driven with a mandrel. Mandrels are generally not used in Wisconsin.

Steel pile can be used in friction, point-bearing, a combination of both, or rock-socketed piles. One advantage of steel pile is the ease of splicing or cutting to accommodate differing final constructed lengths.

Steel pile should not be used for exposed pile bents in corrosive environments as show in the *Facilities Development Manual, Procedure 13.1.15*.

The nominal (ultimate) axial structural compressive resistance of steel piles is designed in accordance with **LRFD [10.7.3.13.1]** as either non-composite or composite sections. Composite sections include concrete-filled pipe pile and steel pile that is encased in concrete. The nominal structural compressive resistance for non-composite and composite steel pile is further specified in **LRFD [6.9.4 and 6.9.5]**, respectively. The effective length of horizontally unsupported steel pile is determined in accordance with **LRFD [10.7.3.13.4]**. Resistance factors for the structural compression limit state are specified in **LRFD [6.5.4.2]**.

WisDOT policy item:

For steel H-piles, 50 ksi yield strength material shall be used. For steel pipe piles, 45 ksi yield strength material shall be used. Plans shall note specified yield strength.

11.3.1.12.3.1 H-Piles

Steel piles are generally used for point-bearing piles and typically employ what is known as the HP-section (often called H-piles for brevity). Steel H-piles are rolled sections with wide flanges such that the depth of the section and the width of the flanges are approximately equal. The cross-sectional area and volume displacement are relatively small and as a result, H-piles can be driven through compact granular materials and slightly into soft rock. Also, steel piles have little or no effect in causing ground swelling or raising of adjacent piles. Because of the small volume of H-piles, they are considered “non-displacement” piling.

H-piles are available in many sizes and lengths. Unspliced pile lengths up to 140 feet and spliced pile lengths up to 230 feet have been driven. Typical pile lengths range from 40 to 120 feet. Common H-pile sizes vary between 10 and 14 inches.

The current WisDOT practice is to design driven H-piles for the factored (ultimate structural) axial compression resistance as shown in Table 11.3-5. These values are based on ϕ_c = 0.5 for severe driving conditions **LRFD [6.5.4.2]**. See 6.3.2.1 for the typical style of plan notes showing axial resistance as well as required driving resistance on plans. **If less than the maximum axial resistance is required by design, state only the required corresponding driving resistance on the plans.**

Since granular soil is largely incompressible, the principal action at the tip of the pile is lateral displacement of soil particles. Although it is an accepted fact that steel piles develop extremely high loads per pile when driven to point-bearing on rock, some misconceptions still remain that H-piles cannot function as friction piles. Load tests indicate that steel H-piles can function quite satisfactorily as friction piles in sand, sand-clay, silt-and-sand or hard clay. However, they are not as efficient as displacement piles in these conditions and typically drive to greater depths.

Pile Size	Shell Thickness (inches)	Concrete or Steel Area (A_g or A_s) (in^2)	Nominal Resistance (Pn) (tons) (2)(3)(6)	(ϕ)	Maximum Factored Resistance (Pr) (tons) (4)	Modified Gates Driving Criteria: Factored Resistance (Pr) (ϕ = 0.50) (tons)	Modified Gates Driving Criteria: Required Driving Resistance (Rn_{dyn}) (tons) (5)	PDA/CAPWAP Driving Criteria: Factored Resistance (Pr) (ϕ = 0.65) (tons)	PDA/CAPWAP Driving Criteria: Required Driving Resistance (Rn_{dyn}) (tons) (5)
Cast in Place Piles									
10 ¾"	0.219	83.5	99.4	0.75	75	55 (8)	110 (11)	72 (8)	110 (11)
10 ¾"	0.250	82.5	98.2	0.75	74	65 (8)	130 (11)	75 (9)	115
10 ¾"	0.365	78.9	93.8	0.75	70	75 (9)	150	75 (9)	115
10 ¾"	0.500	74.7	88.8	0.75	67	75 (9)	150	75 (9)	115
12 ¾"	0.250	118.0	140.4	0.75	105	80 (8)	160 (11)	104 (8)	160 (11)
12 ¾"	0.375	113.1	134.6	0.75	101	105 (9)	210	104 (9)	160
12 ¾"	0.500	108.4	129.0	0.75	97	105 (9)	210	104 (9)	160
14"	0.250	143.1	170.3	0.75	128	85 (8)	170 (11)	111 (8)	170 (11)
14"	0.375	137.9	164.1	0.75	123	120 (8)	240 (11)	120	185
14"	0.500	132.7	158.0	0.75	118	120 (9)	240	120 (9)	185
16"	0.375	182.6	217.3	0.75	163	145 (8)	290 (11)	159	245
16"	0.500	176.7	210.3	0.75	158	160 (9)	320	159 (9)	245
H-Piles									
10 x 42	NA(1)	12.4	310.0	0.50	155	90	180 (10)	117	180 (10)
12 x 53	NA(1)	15.5	387.5	0.50	194	110	220 (10)	143	220 (10)
14 x 73	NA(1)	21.4	535.0	0.50	268	125	250 (10)	162	250 (10)

Table 11.3-5
Typical Pile Axial Compression Resistance Values

Notes:

1. NA – not applicable

2. For CIP Piles: Pn = 0.8 (k_C * f'c * Ag + fy * As) **LRFD [Eq'n 5.6.4.4-3]**. k_C = 0.85 (for $f'_C \leq$ 10.0 ksi). Neglecting the steel shell, equation reduces to 0.68 * f'c * Ag.

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

3. For H-Piles: Pn = (0.66^{λ} * Fe * As) **LRFD [Eq'n 6.9.5.1-1]** (λ = 0 for piles embedded in the ground below the substructure, i.e. no unsupported lengths)

$Fe = fy$ = yield strength of steel = 50,000 psi

4. $Pr = \phi * Pn$

   $\phi = 0.75$ (**LRFD [5.5.4.2]** for axial compression concrete)

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

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

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

     $\varphi_{dyn} = 0.50$ for construction driving criteria using modified Gates

     $\varphi_{dyn} = 0.65$ for construction driving criteria using PDA/CAPWAP

   - The nominal required driving resistance is based on past experience. For H-Piles, refer to note 10. For CIP Piles, refer to note 11.

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

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

8. The Factored Axial Compression Resistance is controlled by the maximum allowable driving resistance based on 70 percent of the specified yield strength of steel rather than concrete capacity. Refer to note 11 for additional information.

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

10. Rn_{dyn} values given for H-Piles are representative of past Departmental experience (rather than Pn x ø) and are used to avoid problems associated with overstressing during driving. These Rn_{dyn} values utilize 46 to 58 percent of the specified yield strength, which is less than the drivability limit **[LRFD 10.7.8]**. If other H-Piles are utilized that are not shown in the table, values should be held to approximately this same range.

11. Rn_{dyn} values given for CIP piles are representative of past Departmental experience of using 35 ksi yield strength material and are used to avoid problems associated with overstressing during driving. These Rn_{dyn} values utilize 70 percent (90% x 35ksi/45ksi) of specified yield strength, which is less than the drivability limit **[LRFD 10.7.8]**. If other

CIP Piles are utilized that are not shown in the table, values should be held to the same limit.

11.3.1.18 Construction Considerations

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

11.3.1.18.1 Pile Hammers

Pile driving hammers are generally powered by compressed air, steam pressure or diesel units. The diesel hammer, a self-contained unit, is the most popular due to its compactness and adoption in most construction codes. Also, the need for auxiliary power is eliminated and the operation cost is nominal. Vibratory and sonic type hammers are employed in special cases where speed of installation is important and/or noise from impact is prohibited. The vibrating hammers convert instantly from a pile driver to a pile extractor by merely tensioning the lift line.

Pile hammers are raised and allowed to fall either by gravity or with the assistance of power. If the fall is due to gravity alone, the hammer is referred to as single-acting. The single-acting hammer is suitable for all types of soil but is most effective in penetrating heavy clays. The major disadvantage is the slow rate of driving due to the relatively slow rate of blows from 50 to 70 per minute. Wisconsin construction specifications call for a minimum hammer weight depending on the required final bearing value of the pile being driven. In order to avoid damage to the pile, the fall of the gravity hammer is limited to 10 feet.

If power is added to the downward falling hammer, the hammer is referred to as double-acting. This type of hammer works best in sandy soil but also performs well in clay. Double-acting hammers deliver 100 to 250 blows per minute, which increases the rate of driving considerably over the single-acting hammers. Wisconsin construction specifications call for a rated minimum energy of 15 percent of the required bearing of the pile. A rapid succession of blows at a high velocity can be extremely inefficient, as the hammer bounces on heavy piles.

Differential-acting hammers overcome the deficiencies found with both single- and double-acting hammers by incorporating higher frequency of blows and more efficient transfer of energy. The steam cycle, which is different from that of any other hammer, makes the lifting area under the piston independent of the downward thrusting area above the piston. Sufficient force can be applied for lifting and accelerating these parts without affecting the dead weight needed to resist the reaction of the downward acceleration force. The maximum delivered energy per blow is the total weight of the hammer plus the weight of the downward steam force times the length of the stroke.

The contractor’s selection of the pile hammer is generally dependent on the following:

- The hammer weight and rated energy are selected on the basis of supplying the maximum driving force without damaging the piles.
- The hammer types dictated by the construction specification for the given pile type.

- The hammer types available to the contractor.

- Special situations, such as sites adjacent to existing buildings, that require consideration of vibrations generated from the driving impact or noise levels. In these instances, reducing the hammer size or choosing a double-acting hammer may be preferred over a single-acting hammer. Impact hammers typically cause less ground vibration than vibratory hammers.

- The subsurface conditions at the site.

- The required final resistance capacity of the pile.

WisDOT specifications require the heads of all piling to be protected by caps during driving. The pile cap serves to protect the pile, as well as modulate the blows from the hammer which helps eliminate large inefficient hammer forces. When penetration-per-blow is used as the driving criteria, constant cap-block material characteristics are required. The cap-block characteristics are also assumed to be constant for all empirical formula computations to determine the rate of penetration equivalent to a particular dynamic resistance.

11.3.1.18.2 Driving Formulas

Formulas used to estimate the bearing capacity of piles are of four general types – empirical, static, dynamic and wave equation.

Empirical formulas are based upon tests under limited conditions and are not suggested for general use.

Static formulas are based on soil stresses and try to equate shaft resistance and point resistance to the load-bearing capacity of the piles.

Dynamic pile driving formulas assume that the kinetic energy imparted by the pile hammer is equal to the nominal pile resistance plus the energy lost during driving, starting with the following relationship:

$$\text{Energy input} = \text{Energy used} + \text{Energy lost}$$

The energy used equals the driving resistance multiplied by the pile movement. Thus, by knowing the energy input and estimating energy losses, driving resistance can be calculated from observed pile movement. Numerous dynamic formulas have been proposed. They range from the simpler Engineering News Record (ENR) Formula to the more complex Hiley Formula. A modified Engineering News Formula was previously used by WisDOT to determine pile resistance capacity during installation. All new designs shall use the FHWA-modified Gates dynamic pile formula (modified Gates) or WAVE equation for determining the required driving resistance.

The following modified Gates formula is used by WisDOT:

$$R_R = \varphi_{dyn} R_{ndr} = \varphi_{dyn}\left(0.875\left(E_d\right)^{0.5}\log_{10}\left(10/s\right) - 50\right)$$

Where:

R_R	=	Factored pile resistance (tons)
φ_{dyn}	=	Resistance factor = 0.50, as specified in Table 11.3-1
R_{ndr}	=	Nominal pile resistance measured during pile driving (tons)
E_d	=	Energy delivered by the hammer per blow (lb-foot)
s	=	Average penetration in inches per blow for the final 10 blows (inches/blow)

Because of the difficulty of evaluating the many energy losses involved with pile driving, these dynamic formulas can only approximate pile driving resistance. These approximate results can be used as a safe means of determining pile length and bearing requirements. Despite the obvious limitations, the dynamic pile formulas take into account the best information available and have considerable utility to the engineer in securing reasonably safe and uniform results over the entire project.

The wave equation can be used to set driving criteria to achieve a specified pile bearing capacity (contact the Bureau of Technical Services, Geotechnical Engineering Unit prior to using the wave equation to set the driving criteria). The wave equation is based upon the theory of longitudinal wave transmission. This theory, proposed by Saint Venant a century ago, did not receive widespread use until the advent of computers due to its complexity. The wave equation can predict impact stresses in a pile during driving and estimate static soil resistance at the time of driving by solving a series of simultaneous equations. An advantage of this method is that it can accommodate any pile shape, as well as any distribution of pile shaft resistance and point resistance. The effect of the hammer and cushion block can be included in the computations.

Dynamic monitoring is performed by a Pile Driving Analyzer (PDA). WisDOT uses the PDA to evaluate the driving criteria, which is set by a wave equation analysis, and in an advisory capacity for evaluating if sufficient pile penetration is achieved, if pile damage has occurred or if the driving system is performing satisfactorily.

The PDA provides a method of dynamic pile testing both for pile design and construction control. Testing is accomplished during pile installation by attaching reusable strain transducers and accelerometers directly on the pile. Piles can be tested while being driven or during restrike. The instrumentation mounted on the pile allows the measurement of force and acceleration signals for each hammer blow. This data is transmitted to a small field computer for processing and recording. Calculations made by the computer based upon one-dimensional wave mechanics provide an immediate readout of maximum stresses in the pile, energy transmitted to the pile and a prediction of the nominal axial resistance of the pile for each hammer impact. Monitoring of the force and velocity wave traces with the computer during driving also enables detection of any structural pile damage that may have occurred. Review of selected force and velocity wave traces are also available to provide additional testing documentation. The PDA can be used on all types of driven piles with any impact type of pile-driving hammer.

11.3.1.18.3 Field Testing

Test piles are employed at a project site for two purposes:

- For test driving, to determine the length of pile required prior to placing purchasing orders.
- For load testing, to verify actual pile capacity versus design capacity for nominal axial resistance.

11.3.1.18.3.1 Installation of Test Piles

Test piles are not required for spliceable types of piles. Previous experience indicates that contractors typically order total plan quantities for cast-in-place or steel H-piling in 60-foot lengths. The contractor uses one of the driven structure piles as a test pile at each designated location.

Test piling should be driven near the location of a soil boring where the soil characteristics are known and representative of the most unfavorable conditions at the site. The test pile must be exactly the same type and dimension as the piles to be used in the construction and installed by the same equipment and manner of driving. A penetration record is kept for every 1 foot of penetration for the entire length of pile. This record may be used as a guide for future pile driving on the project. Any subsequent pile encountering a smaller resistance is considered as having a smaller nominal resistance capacity than the test pile.

11.3.1.18.3.2 Static Pile Load Tests

A static pile load test is usually conducted to furnish information to the geotechnical engineer to develop design criteria or to obtain test data to substantiate nominal resistance capacity for piles. A static pile load test is the only reliable method of determining the nominal bearing resistance of a single pile, but it is expensive and can be quite time consuming. The decision to embark on an advance test program is based upon the scope of the project and the complexities of the foundation conditions. Such test programs on projects with large numbers of displacement piling often result in substantial savings in foundation costs, which can more than offset the test program cost. WisDOT has only performed a limited number of pile load tests on similar type projects.

Static pile load testing generally involves the application of a direct axial load to a single vertical pile. However, static pile load testing can involve uplift or axial tension tests, lateral tests applied horizontally, group tests or a combination of these applied to battered piles. Most static test loads are applied with hydraulic jacks reacting against either a stable loaded platform or a test frame anchored to reaction piles.

The basic information to be developed from the static pile load test is usually the deflection of the pile head under the test load. Movement of the head is caused by elastic deformation of the piles and the soil. Soil deformation may cause undue settlement and must be guarded against. The amount of deformation is the significant value to be obtained from load tests, rather than the total downward movement of the pile head. Static pile load tests are typically performed by loading to a given deflection value.

It is impractical to test every pile on a project. Therefore, test results can be applied to other piles or pile groups providing that the following conditions exist:

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

11.3.1.19 Construction Monitoring for Economic Evaluation of Deep Foundations

The goal of the foundation design is to provide the most efficient and economical design for the subsurface conditions. The design of pile-supported foundations is influenced by the resistance factor, which is generally a function of pile resistance determination during installation. The discussion in 11.3.1.14 presents the definition of resistance factors.

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

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

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

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

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

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

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

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

The following two examples use Method 1 to illustrate the potential cost savings/expenses for PDA with CAPWAP:

Pier
Pier Example: 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $40/foot. (**Note:** It is realized that for pier design the number of piles is not exclusively related to the vertical load, but this example is simplified for illustrative purposes).
Modified Gates: RDR = 220 tons, FACR = 110 tons, Total Load on Pier = 3,500 tons, Number of Piles = 3,500 tons / 110 tons = 32 piles <u>Pile Cost = 32 piles x 100 feet x $40/ft = $128,000</u> Total Cost = $128,000
PDA/CAPWAP: RDR = 220 tons, FACR = 143 tons, Total Load on Pier = 3,500 tons, Number of Piles = 3,500 tons / 143 tons = 25 piles Pile Cost = 25 piles x 100 feet x $40/ft = $100,000 PDA Testing Cost = 2 piles/sub. x $700/pile = $1,400 PDA Restrike Cost = 2 piles/sub. x $600/pile = $1,200 <u>CAPWAP Evaluation = 1 eval./sub. x $400/eval. = $400</u>

Total Cost = $103,000
PDA/CAPWAP Savings = $25,000/pier
Abutment
Abutment Example: 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $40/foot.
Modified Gates: RDR = 220 tons, FACR = 110 tons, Total Load on Abut = 980 tons, Number of Piles = 980 tons / 110 tons = 9 piles Total Cost = 9 piles x 100 feet x $40/ft = $36,000
PDA/CAPWAP: RDR = 220 tons, FACR = 143 tons, Load on Abut = 980 tons, Number of Piles = 980 tons / 143 tons = 7 piles, however because of maximum spacing requirements the design will need 8 piles. Pile Cost = 8 piles x 100 feet x $40/ft = $32,000 PDA Testing Cost = 2 piles/sub. x $700/pile = $1,400 PDA Restrike Cost = 2 piles/sub. x $600/pile = $1,200 CAPWAP Evaluation = 1 eval./sub. x $400/eval. = $400 Total Cost = $35,000
PDA/CAPWAP Cost = $1000/abutment **Note:** For a three span bridge, with 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $40/foot, PDA/CAPWAP would provide an estimated structure savings of $52,000. For a two span bridge, with 12 x 53 H-piles to an estimated length of 40 feet at a unit cost of $40/foot, PDA/CAPWAP would provide an estimated structure savings of $5,400. Bid prices based on 2014-2015 cost data.

Table 11.3-6

Economical Evaluation for Deep Foundations with Two Construction Monitoring Methods

11.3.2 Drilled Shafts

11.3.2.1 General

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

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

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

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

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

The minimum drilled shaft spacing shall be 3.0 shaft diameters center-to-center (3D). When drilled shafts are spaced less than 6D, group effects shall be evaluated for possible reductions to axial and lateral resistances. See 11.3.2.3.3 for more information.

Drilled shafts have been used on only a small number of structures in Wisconsin. For unusual site conditions, the use of drilled shafts may be advantageous. Design methodologies for drilled shafts can be found in **LRFD [10.8]** Drilled Shafts and *Drilled Shafts: Construction Procedures and Design Methods*. FHWA Publication NHI-18-024, FHWA GEC 010. 2018.

Strength limit states for drilled shafts are evaluated in the same way as for driven piles. Drivability is not required to be evaluated. The structural resistance of drilled shafts is evaluated in accordance with **LRFD [5.6 and 5.7]**. This includes evaluation of axial resistance, combined axial and flexure, shear and buckling. It is noted that the critical load case for combined axial and flexure may be a load case that results in the minimum axial load or tension.

11.3.2.2 Resistance Factors

Resistance factors for drilled shafts are presented in Table 11.3-7 and are selected based on the method used to determine the nominal (ultimate) resistance capacity of the drilled shaft. The design intent is to adjust the resistance factor based on the reliability of the method used to determine the nominal shaft resistance. As with driven piles, the selection of a geotechnical resistance factor should be based on the intended method of resistance verification in the field. Because of the cost and difficulty associated with testing drilled shafts, much more reliance is placed on static analysis methods.

<table>
<tr><th colspan="4">Condition/Resistance Determination Method</th><th>Resistance Factor</th></tr>
<tr><td rowspan="16">Static Analysis – Used in Design Phase</td><td rowspan="10">Nominal Resistance of Single-Drilled Shaft in Axial Compression, φ_{stat}</td><td>Shaft Resistance in Clay</td><td>Alpha Method</td><td>0.45</td></tr>
<tr><td>Point Resistance in Clay</td><td>Total Stress</td><td>0.40</td></tr>
<tr><td>Shaft Resistance in Sand</td><td>Beta Method</td><td>0.55</td></tr>
<tr><td>Point Resistance in Sand</td><td>O’Neill and Reese</td><td>0.50</td></tr>
<tr><td>Shaft Resistance in IGMs</td><td>O’Neill and Reese</td><td>0.60</td></tr>
<tr><td>Point Resistance in IGMs</td><td>O’Neill and Reese</td><td>0.55</td></tr>
<tr><td rowspan="2">Shaft Resistance in Rock</td><td>Horvath and Kenney
O’Neill and Reese</td><td>0.55</td></tr>
<tr><td>Carter and Kulhawy</td><td>0.50</td></tr>
<tr><td>Point Resistance in Rock</td><td>Canadian Geotech. Soc.
Pressuremeter Method
O’Neill and Reese</td><td>0.50</td></tr>
<tr><td colspan="2" style="display:none"></td></tr>
<tr><td>Block Failure, φ_{bl}</td><td colspan="2">Clay</td><td>0.55</td></tr>
<tr><td rowspan="3">Uplift Resistance of Single-Drilled Shaft, φ_{up}</td><td>Clay</td><td>Alpha Method</td><td>0.35</td></tr>
<tr><td>Sand</td><td>Beta Method</td><td>0.45</td></tr>
<tr><td>Rock</td><td>Horvath and Kenney
Carter and Kulhawy</td><td>0.40</td></tr>
<tr><td>Group Uplift Resistance, φ_{ug}</td><td colspan="2">Sand and Clay</td><td>0.45</td></tr>
<tr><td>Horizontal Geotechnical Resistance of Single Shaft or Pile Group</td><td colspan="2">All Soil Types and Rock</td><td>1.0</td></tr>
</table>

Table 11.3-7
Geotechnical Resistance Factors for Drilled Shafts **LRFD [Table 10.5.5.2.4-1]**

For drilled shafts, the base geotechnical resistance factors in Table 11.3-7 assume groups containing two to four shafts, which are slightly redundant. For groups containing at least five elements, the base geotechnical resistance factors in Table 11.3-7 should be increased by 20%.

WisDOT policy item:

When a bent contains at least 5 columns (where each column is supported on a single drilled shaft) the resistance factors in Table 11.3-7 should be increased up to 20 percent for the Strength Limit State.

For piers supported on a single drilled shaft, the resistance factors in Table 11.3-7 should be decreased by 20 percent for the Strength Limit State. Use of single drilled shaft piers requires approval from the Bureau of Structures.

Resistance factors for structural design of drilled shafts are obtained from **LRFD [5.5.4.2]**.

11.3.2.3 Bearing Resistance

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

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

Because these resistances mobilize at different rates, it may be more appropriate to add the ultimate end bearing to that portion of the side resistance remaining at the end of bearing failure. Or it may be more appropriate to add the ultimate side resistance to that portion of the end bearing mobilized at side resistance failure. Note that consideration of deflection, which is a service limit state, may control over the axial geotechnical resistance since displacements required to mobilize the ultimate end bearing can be excessive.

11.3.2.3.1 Shaft Resistance

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

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

11.3.2.3.2 Point Resistance

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

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

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

11.3.2.3.3 Group Capacity

Group effects for axial and lateral resistances shall be evaluated in accordance with **LRFD [10.8.3.6]** and **LRFD [10.8.3.8]**, respectively. In general, reductions to individual nominal resistances are limited to drilled shafts spaced less than 6D and are based on spacing, soil type, and soil contact.

11.3.2.4 Lateral Load Resistance

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

11.3.2.5 Other Considerations

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

11.3.3 Micropiles

11.3.3.1 General

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

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

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

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

11.3.3.2 Design Guidance

Micropiles shall be designed in conformance with the current *AASHTO LRFD* and in accordance with the WisDOT Bridge Manual. Design guidelines for micropiles are provided in FHWA Publication No. FHWA-NHI-05-039.

11.3.4 Augered Cast-In-Place Piles

11.3.4.1 General

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

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

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

11.3.4.2 Design Guidance

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

11.4 References

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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17.8 Bridge Deck Protective Systems

17.8.1 General

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

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

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

17.8.2 Design Guidance

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

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

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

- High Performance Concrete (HPC) – This is typically used within the bridge superstructure (deck, diaphragms, parapets, structural approach slabs, etc.) on urban interchange projects

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

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

17.9 Bridge Approaches

The structure approach slab, or approach pavement, is part of the roadway design plans. Structure approach standards are provided in the Facilities Development Manual (FDM).

Guidance for the selection of pavement types for bridge approaches is as shown in FDM 14-10-15.

Considerations for site materials, drainage and backfill are provided in Chapter 12 – Abutments. Most approach pavement failures are related to settlement of embankment or foundation materials. Past experience shows that significant settlement is most likely to occur where marginal materials are used. Designers are encouraged to provide perforated underdrains wrapped in geotextile fabric placed in a trench filled with crushed stone. Also, abutment backfill material should be granular in nature and consolidated under optimum moisture conditions.

17.10 Design of Precast Prestressed Concrete Deck Panels

17.10.1 General

An advantage of stay-in-place forms is that they can be placed in less time than it takes to place the forms for a conventional deck. There is also a labor savings because the extra step of removing deck forms is not required. Stay-in-place forms are often the preferred system for shallow box girders because of the difficulty of removing forms in a confined space.

When determined ideal for a project, precast concrete deck panels should be detailed in the contract documents. Include the Standardized Special Provision *Precast Prestressed Concrete Deck Panels*. The contractor is responsible for the shop plans of the panels and any other changes that may be required to the reinforcing steel in the cast-in-place portion of the deck. Contract documents should also include an option for the contractor to use a conventional deck. Contact the Bureau of Structures Design Section for other considerations.

When a conventional deck is detailed in the contract documents and the contractor is interested in utilizing precast deck panels, the department may consider their usage on a project-specific basis. The contractor would be responsible for the shop plans of the panels and any other changes that may be required to the reinforcing steel in the cast-in-place portion of the deck. Payment to a contractor who chooses to use stay-in-place forms is based on the contract prices bid for the conventional cast-in-place deck.

Deck panels are only used between the inside faces of the exterior girders. The overhangs outside the exterior girders are formed and the concrete placed in the same way as in a conventional cast-in-place deck. On skewed decks, the contractor may form and cast the skewed portion of the deck full depth or they may use skewed end deck panels which may be individually precast or saw-cut from square end planks.

One potential issue with decks formed using concrete deck panels is that cracks often form in the cast-in-place concrete over the transverse joints between panels and along the edges of the panels parallel to the girders. Reflection cracking is less of a problem when these panels are used on prestressed concrete girders than on steel girders. Simple-span prestressed concrete girder bridges have less reflective cracking than continuous-span prestressed concrete girder bridges.

17.10.2 Deck Panel Design

The design of precast prestressed concrete deck panels shown in Table 17.10-1 is based on *AASHTO LRFD* design criteria. These panels were designed for flexure due to the HL-93 design truck live load, dead load of the plastic concrete supported by the panels, a construction load of 50 psf, dead load of the panels and a future wearing surface of 20 psf. The live load moments were obtained from **LRFD [Table A4-1]**.

At the request of precast deck panel fabricators, only two strand sizes are used – 3/8 inch and 1/2 inch. Strand spacing is given in multiples of 2 inches.

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24.15 Box Girders

Box girders present a smooth, uncluttered appearance under the bridge deck due to the lack of transverse bracing and due to their closed section. Enhanced torsional rigidity can make box girders a favorable choice for horizontally curved bridges. However, due to redundancy concerns, use of single-box and two-box girder bridges should be avoided unless absolutely necessary.

In the design of box girders, the concrete slab is designed as a portion of the top flange and also as the support between the two girder webs which satisfies the requirement for being considered a closed box section.

Current experience shows that box girders may require more material than conventional plate girders. On longer spans, additional bracing between girders is required to transfer lateral loads.

Several requirements in *AASHTO LRFD* are specific to box girders. For box girders, sections in positive flexure shall not have a yield strength in excess of 70 ksi. The following web slenderness requirement from **LRFD [6.11.6.2.2]** must also be satisfied:

$$\frac{2D_{cp}}{t_w} \le 3.76\sqrt{\frac{E}{F_{yc}}}$$

Where:

D_{cp} = Depth of web in compression at plastic moment (in.)

t_w = Web thickness (in.)

F_{yc} = Specified minimum yield strength of the compression flange (ksi)

Other requirements for positive flexure in box girders are presented in **LRFD [6.11.6.2.2]**. Steel sections in negative flexure must not use the provisions in Appendices A or B of the *AASHTO LRFD* specifications.

When computing effective flange widths for closed-box sections, the distance between the outside of the webs at the tops is to be used in lieu of the web thickness in the general requirements. For closed box sections, the spacing should be taken as the spacing between the centerlines of the box sections.

When computing section properties for closed-box sections with inclined webs, the moment of inertia of the webs about a horizontal axis at the mid-depth of the web should be adjusted for the web slope by dividing by the cosine of the angle of inclination of the web plate to the vertical. Also, inspection manholes are often inserted in the bottom flanges of closed-box sections near supports. These manholes should be subtracted from the bottom-flange area when computing the elastic section properties for use in the region of the access hole. If longitudinal flange stiffeners are present on the closed-box section, they are often included when computing the elastic section properties.

When investigating web bend-buckling resistance for closed-box sections, **LRFD [6.11.3.2]** states that the maximum longitudinal flange stress due to the factored loads, calculated without consideration of longitudinal warping, must not exceed $\phi_f F_{crw}$ at sections where non-composite box flanges are subject to compression during construction. For more information about the web bend-buckling resistance of box girders, refer to 24.12.1. In *AASHTO LRFD*, a box flange is defined as a flange connected to two webs.

Torsion in structural members is generally resisted through a combination of St. Venant torsion and warping torsion. For closed cross-sections such as box girders, St. Venant torsion generally dominates. Box girders possess favorable torsional characteristics which make them an attractive choice for horizontally curved bridges. However, due to redundancy concerns, use of single-box and two-box girder bridges should be avoided unless absolutely necessary.

WisDOT policy item:

Certain criteria must be met to consider a trapezoidal steel box girder bridge to be a System Redundant Member (SRM), as outlined in *A Simplified Approach for Designing SRMs in Composite Continuous Twin-Tub Girder Bridges* (as summarized in Appendix A – the full report is available upon request from BOS) by Robert J. Connor, et. al., Purdue University. A summary of these steps required by WisDOT are outlined below this policy item box.

It is required to design twin-tub girders to meet SRM criteria. BOS approval is required for all box girders.

Summary of Appendix A

Approach
For a multi-span twin-tub girder bridge to be considered an SRM, the bridge must meet certain screening criteria. If the criteria are met, design must be in accordance with the provisions set forth in the subject report. Figure A-1 is a flowchart for describing the proposed guideline steps.

Screening
To consider a twin-tub girder an SRM, certain criteria must be met, which require continuous spans, composite section with specific shear stud design, maximum bridge width, maximum girder spacing, web depth range, interior span length limits, exterior span length limits, ratio of unfractured to fractured span length limits, ratio of radius of curvature to longest span length limit, skew limit, maximum number of design lanes, and maximum dead load displacement limit at both interior and exterior spans.

Design Methodology
If the screening criteria are met, the design then needs to meet specific design requirements for shear studs, intermediate diaphragms, bottom flange buckling resistance, and positive moment flexural resistance.

Additional information regarding design and rating includes:

New twin steel tub girder designs should continue to include the redundancy load factor (LRFD 1.3.4) for nonredundant members, η_R = 1.05 under the strength limit state, regardless of the structure's final redundant related classification (e.g. FCM or SRM). The continued use of this load factor, even if a structure is determined to be redundant via system redundant classification is to maintain consistency in design with the original group of structures evaluated and documented in the report by Purdue University.

However, the load redundancy factor shall not be considered when checking the *Redundancy I and II* limit states described in the aforementioned report.

For load ratings, the Manual for Bridge Evaluation, section 6A.4.2.4 applies a system factor ϕ_S = 0.85 to the resistance of welded members in two-girder systems (i.e. twin steel tub girders). If a twin steel tub girder bridge has achieved SRM classification the system factor should be taken as 1.0 for load rating purposes.

24.16 Design Examples

E24-1 2-Span Continuous Steel Plate Girder Bridge, LRFD

E24-2 Bolted Field Splice, LRFD

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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.

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29.1 General

Wherever practical, bridge drainage should be carried off the structure along the curb or gutter line and collected with roadway catch basins. Floor drains are not recommended for structures less than 200' long and floor drain spacing is not to exceed 500' on any structure. However, additional floor drains are required on some structures due to flat grades, superelevations and the crest of vertical curves. The drains are spaced according to the criteria as set forth in 29.2, which includes acceptable spread of water measured from gutterline as a function of design speed, design storm frequency and duration of rainfall. Additional drains should not be provided other than what is required by design. Utilizing blockouts in parapets to facilitate drainage is not allowed.

Superelevation on structures often creates drainage problems other than at the low point especially if a reverse curve is involved. Water collects and flows down one gutter and as it starts into the superelevation transition it spreads out over the complete width of roadway at the point of zero cross-slope. From this point the water starts to flow into the opposite gutter. Certain freezing conditions can cause traffic accidents to occur in the flat area between the two transitions. To minimize the problem, locate the floor drain as close to the cross over point as practical. Floor drains are installed as near all joints as practical to prevent gutter flow from passing over and/or through the joints.

The Bureau of Structures recommends the Type “GC” floor drain for new structures. Type "GC" floor drains are gray iron castings that have been tested for hydraulic efficiency. Where hydraulic efficiency or girder flange to edge of deck geometry dictates the use of a different floor drain configuration, BOS recommends the Type “WF” floor drain. Steel fabricated floor drains Type “H” provide an additional 6” of downspout clearance and are retained for maintenance of structures where floor drain size modifications are necessary.

All of the floor drains shown on the Standards have grate inlets. When the longitudinal grade exceeds 1 percent, hydraulic flow testing indicates grates with rectangular longitudinal bars are more efficient than grates having transverse rectangular bars normal to flow. However, grates with bars parallel to the direction of traffic are hazardous to bicyclists and even motorcyclists as bar spacing is increased for hydraulic efficiency. As a result, transverse bars sloped toward the direction of flow are detailed for the cast iron floor drains.

Downspouts are to be fabricated from reinforced thermosetting resin (fiberglass) pipe having a diameter not less than 6” for all new structures. Galvanized standard pipe or reinforced fiberglass material may be used for downspouts when adjusting or rehabilitating existing floor drains. Downspouts are required on all floor drains to prevent water and/or chlorides from getting on the girders, bearings, substructure units, etc. Downspouts should be detailed to extend a minimum of 6” below low prestressed girder bottom flange or 1' below low steel to prevent flange or web corrosion. A downspout collector system is required on all structures over grade separations. Reinforced fiberglass pipe is recommended for all collector systems due to its durability and economy. In the design of collector systems, elimination of unnecessary bends and provision for an adequate number of clean outs is recommended.

29.2 Design Criteria

The flow of water in an open channel depends on its cross section, grade, and roughness. Generally, the gutter cross section on a structure is right triangular in shape with the curb, median or parapet forming the vertical leg. For design speeds 45 mph or less, floor drains are spaced at a distance such that the maximum gutter flow is restricted to a spread width of the shoulder plus one-half the adjacent through driving lane for a given design frequency storm. This defines the hypotenuse of the triangle if the shoulder and driving lane slope are equal. For design speeds greater than 45 mph, floor drains are spaced at a distance such that the maximum gutter flow is restricted to a spread width of the shoulder. An increase in longitudinal and transverse slope increases inlet capacity. In design, it is assumed that all of the water passing over the width of the inlet is taken by that inlet, the remaining water (Q bypass) continues to the next inlet.

For design, a storm frequency of 10 years with a duration of 5 minutes is used. This gives a rainfall intensity (i) in inches/hour that can be found for each county in Wisconsin in the *Facilities Development Manual (FDM)* (Sect. 13-10, Attachment 5.4). A run-off coefficient (C) of 0.9 is used for concrete surfaces.

The Rational Method (English Units) converts rainfall intensity for a given design frequency storm to run-off by the following equation:

$$Q = C\,i\,A$$

Where:

Q	=	peak rate of run-off in cfs.
C	=	run-off coefficient for surface type.
i	=	rainfall intensity in inches/hour.
A	=	drainage area in acres = $\frac{LW}{43560}$

Where:

L	=	floor drain spacing in feet.
W	=	contributing structure width in feet.

The Manning equation modified for triangular flow is used to compute Q and Q_{bypass} for the given gutter section. The modified equation is:

$$Q = 0.56\left(\frac{Z}{n}\right)(S_o)^{\frac{1}{2}}(d)^{\frac{8}{3}}$$

Where:

Q	=	discharge in cfs.
Z	=	reciprocal of cross slope.
n	=	Manning's coefficient of roughness, use n = 0.014 for concrete.
S_o	=	longitudinal slope in feet/foot.
d	=	depth of flow at the deepest point (gutter line) in feet.

Refer to Table 29.2-1, Table 29.2-2 and Table 29.2-3 for values of (Z/n) and to Figure 29.2-1 for a nomographic solution to the Manning equation.

29.3 Design Example

The following method is used to compute floor drain spacing by equating net discharge to the Rational Method:

Given: Structure 1200 feet long on a 0.3% grade having a cross slope of 0.02 feet/foot with a contributing structure width of 23'-6". Use Type "GC" floor drain. For a structure in Marathon County, the rainfall intensity (i) from the *FDM* (Sect. 13-10, Attachment 5.4) is 6.60 in./hr.

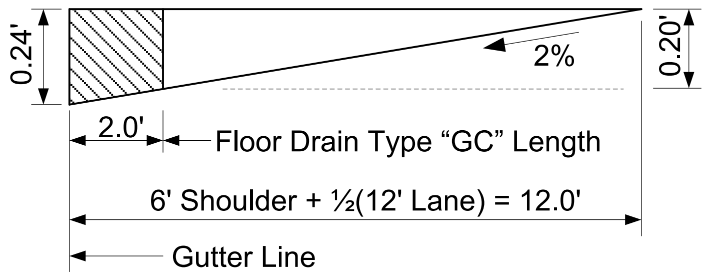

Figure 29.3-1
Cross Section of Flow

Compute: Floor drain spacing

From Table 29.2-1 with a cross slope of 0.02 feet/foot

$(Z/n) = 3571.$

From Figure 29.2-1, Q = 2.44 cfs and Q_{bypass} = 1.50 cfs.

$$L = (Q - Q_{bypass})\frac{43560}{CiW}$$

L = (2.44 – 1.5) · 43569 / (0.9 · 6.60 · 23.5}

L = 293 ft

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36.1 Design Method

36.1.1 Design Requirements

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

36.1.2 Rating Requirements

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

36.1.3 Standard Permit Design Check

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

36.11 Plan Detailing Issues

36.11.1 Weep Holes

Investigate the need for weep holes for culverts in cohesive soils. These holes are to relieve the hydrostatic pressure on the sides of the culverts. Where used, place the weep holes 1 foot above normal water elevation but a minimum of 1 foot above the lower sidewall construction joint. Do not place weep holes closer than 1 foot from the bottom of the top slab.

36.11.2 Cutoff Walls

Where dewatering the cutoff wall in sandy terrain is a problem, the concrete may be poured in the water. Place a note on the plans allowing concrete for the cutoff wall to be placed in the water.

36.11.3 Nameplate

Designate a location on the wingwall for placement of the nameplate. Locate nameplate on the first right wing traveling in the Cardinal direction (North/East).

36.11.4 Plans Policy

If cast-in-place concrete box sections or aprons are used, full plans shall be provided and sealed by a professional engineer. The plans shall be in accordance with the *Bridge Manual* and Standards.

If precast concrete box sections are allowed in lieu of cast-in-place concrete, a noted allowance shall be provided on the plans. Precast details are not required for box sections following ASTM Specification C1577. The design and fabrication shall be in accordance with ASTM Specification C1577, AASHTO LRFD Specifications, and the Bridge Manual.

If precast only concrete box sections are justified, precast details are required for box sections following ASTM Specification C1577. The design and fabrication shall be in accordance with ASTM Specification C1577, AASHTO LRFD Specifications, and the Bridge Manual.

If precast concrete apron elements are allowed, a noted allowance shall be provided on the plans and precast details shall be provided in accordance with the *Bridge Manual* and Standards. The design may deviate (e.g. use a precast apron floor) from the precast alternatives shown in the Standards provided the engineer submits design calculations, sealed by a professional engineer, to the Bureau of Structures for acceptance. The design and fabrication shall be in accordance with AASHTO LRFD Specifications and the Bridge Manual.

If the contractor selects a precast alternative, the contractor is to submit shop drawings, sealed by a professional engineer, to the Bureau of Structures for acceptance. If precast concrete elements (e.g. apron wingwalls) are prohibited by the designer, the plans shall be noted accordingly.

36.11.5 Rubberized Membrane Waterproofing

When required by the Standard Details, place the bid item "Rubberized Membrane Waterproofing" on the final plans. The quantity is given square yards.

36.12 Precast Four-Sided Box Culverts

Typically, precast concrete box culverts can reduce construction time, but may also cost more than cast-in-place concrete construction. As such, it is often difficult to determine if a contractor will choose to use precast or cast-in-place sections. To provide greater flexibility, projects can provide options (alternatives) for the contractor to determine if precast would be beneficial based on the project's needs.

In general, there are two options for preparing concrete box culvert plans. The most common and recommended option is to provide a complete cast-in-place concrete design with a noted allowance for the contractor to substitute the cast-in-place design with precast box sections in accordance with ASTM C1577. This option provides project flexibility while maintaining historically lower cast-in-place concrete costs. The designer shall determine if a noted precast allowance is appropriate on a project-by-project basis. In some cases, the precast option may not be suitable and should be noted accordingly on the plans. The following are several conditions where a noted allowance for precast may not be suitable for a project:

- Structure openings not covered by ASTM Specification C1577, which will require a separate analysis.
- Structure skew is greater than 30 degrees and the depth of cover is less than 5 feet. This condition is beyond the design tables shown in ASTM C1577 and requires a separate analysis.
- Depth of cover is less than 2 ft while supporting traffic loads. Cast-in-place sections are preferred due to performance concerns at the top slab and joint locations.
- Pedestrian underpasses - Cast-in-place sections are preferred for improved serviceability.
- Unique hydraulic conditions or other factors may also warrant not allowing precast sections, such as differential settlement concerns.

A precast concrete only plan delivery method may be considered when cast-in-place concrete usage is highly unlikely. This option would simplify plan preparation and may provide design savings. Use of precast only culverts, that are assigned a structure number, are subject to prior-approval by the Bureau of Structures.

If precast concrete box sections are allowed, the designer shall also determine if precast aprons should be allowed as well. Use of precast aprons may not be as beneficial as concrete box sections since these elements are located beyond the construction staging limits and may not require an accelerated schedule.

Refer to 36.11.4 for additional information on plan detail requirements.

36.13 Other Buried Structures

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

36.13.1 General

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

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

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

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

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

36.13.2 Three-Sided Concrete Structures

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

36.13.2.1 Cast-In-Place Three-Sided Structures

To be developed

36.13.2.2 Precast Three-Sided Structures

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

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

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

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

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

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

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

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

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

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

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

36.13.2.2.1 Precast Three-Sided Structure Span Lengths

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

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

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

36.13.2.2.2 Segment Configuration and Skew

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

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

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

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

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

36.13.2.2.2.1 Minimum Fill Height

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

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

36.13.2.2.2.2 Rise

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

36.13.2.2.2.3 Deflections

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

36.13.2.3 Plans Policy

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

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

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

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

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

The following plan requirements shall be followed:

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

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

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

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

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

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

36.13.2.4 Foundation Requirements

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

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

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

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

36.13.2.5 Precast Versus Cast-in-Place Wingwalls and Headwalls

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

36.13.3 Metal Buried Structures

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

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

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

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

Design Life: The minimum service life shall be 75 years.

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

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

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

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

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

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

36.13.3.1 Metal Pipes and Pipe-Arches

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

36.13.3.2 Other Shapes

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

36.14 References

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

36.15 Design Example

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39.2 Materials

WisDOT uses the following for round, hollow structural sections (HSS) for truss chord members, vertical support members and horizontal monotube members.

<table>
<tr><th>Member Type</th><th colspan="2">Material Requirements</th></tr>
<tr><td rowspan="3">HSS Chords, Vertical Supports, & Horizontal Monotubes</td><td>Wall Thickness ≤ ½”</td><td>ASTM A500, Grade C (Fy = 46 ksi)</td></tr>
<tr><td>Wall Thickness > ½”
and
Pipe Diameter ≤ 20”</td><td>ASTM A1085 (round HSS)
Or
API 5L Grade 46 PSL-2 (round pipe)</td></tr>
<tr><td>Pipe Diameter > 20”
(Any Wall Thickness)</td><td>API 5L Grade 46 PSL-2 (round pipe)</td></tr>
<tr><td>Plates, Bars, and Structural Angles</td><td colspan="2">ASTM A709, Grade 36</td></tr>
<tr><td>Round or Multi-Sided Tapered Poles</td><td colspan="2">ASTM A595, Grade A (Fy = 55 ksi)
Or
ASTM A572, Grade 55</td></tr>
</table>

Galvanized ASTM F3125 A325 bolts with DTI washers are to be used in all primary structural connections, including those that are fully tensioned. A449 bolts are not allowed in fully tensioned connections and are only allowed in full span chord to column saddle or full span post to chord clamp connections. More details can be found in the OSS Standard Design Drawings and Standard Specifications Section 532.

WisDOT policy item:

Installation of flat washers in between faying surfaces of mast arm connection plates is not allowed.

When selecting members sizes for individually designed OSS, it is important to select members that are regularly produced and domestically available. Specifying members that are infrequently produced may result in higher bid prices, longer fabrication lead time, and/or member substitution requests that may delay the fabrication and production process. A general rule of thumb is to select HSS round tube members that match standard (Schedule 40) outside pipe diameters and thickness. The Steel Tube Institute provides current information on their website regarding domestic availability of HSS sections at:

https://steeltubeinstitute.org/hss/availability-tool/.

Designers can also consult the Bureau of Structures.

39.3 Specifications

39.3.1 LRFD Design

WisDOT has transitioned the design of all roadside standard Type 1 breakaway sign supports and foundations to be in accordance with the *AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals, 1st Edition (2015) (LRFDLTS-1)* with current Interim revisions.

WisDOT is currently transitioning the design of OSS to be in accordance with the AASHTO LRFDLTS-1 with current Interim revisions. Use of the AASHTO LRFDLTS-1 specification for OSS is currently optional and will be required beginning with the December 2020 letting.

39.3.2 Other Specifications and Manuals

The following manuals and specifications provide further guidance and requirements for the design and construction of OSS:

- Wisconsin Department of Transportation "*Bridge Manual*" (BM)
- Wisconsin Department of Transportation "*Geotechnical Manual*"
- Wisconsin Department of Transportation *"Facilities Development Manual"* (FDM)
- State of Wisconsin *"Standard Specifications for Highway and Structure Construction"*
- State of Wisconsin "*Construction and Materials Manual*" (CMM)
- AASHTO "*LRFD Bridge Design Specifications"* (Current Edition and Interim Specifications)
- American Society for Testing and Materials Standards (ASTM)
- American National Standards Institute / American Petroleum Institute 5L Specification for Line Pipe. (ANSI / API 5L)
- AWS D1.1 Structural Welding Code (Steel)
- AWS D1.2 Structural Welding Code (Aluminum)

39.5 Geotechnical Guidelines

39.5.1 General

For full span and cantilever 4-chord trusses, the typical preferred foundation is comprised of two cylindrical drilled shafts connected by a concrete cross-girder, as detailed in the OSS Standard Design Drawings. The top of the cross-girder is set 3 feet above the highest ground elevation at the foundation. For all other types, the typical preferred foundation is comprised of a single cylindrical drilled shaft directly supporting the column vertical support. Occasionally, some columns are mounted directly on top of modified bridge parapets, pier caps and concrete towers instead of footings.

There are several potential challenges regarding subsurface exploration for OSS foundations:

- The development and location of these structures are typically not known at the onset of the preliminary design stage, when the most subsurface exploration typically occurs. This creates the potential need for multiple drilling mobilizations for the project.

- OSS are often located in areas of proposed fill soils. The source and characteristics of fill soil is unknown at the time of design.

- OSS foundations are often located on the shoulder or median directly adjacent to high-volume roadways. Obtaining boings in these locations typically requires significant traffic control, night work, and working in a potentially hazardous work zone.

- If a consultant is involved in the project, the unknowns associated with these structures in the project scoping stage complicate the consultant contracting process. It is often difficult to determine the need for OSS specific subsurface investigation at the time the consultant contract is normally being scoped. In cases where the need for a specific subsurface investigation is known or anticipated, an assumption must be made regarding the level of subsurface investigation to include in the consultant design contract. Alternatively, a decision can be made to assume use of standard OSS and foundation designs. If the need for specific subsurface investigation is later determined to be necessary, this may require a change to add it to the consultant contract.

39.5.2 Standard Foundations for OSS

39.5.2.1 General

WisDOT has created standard full span and cantilever 4-chord truss designs that include fully designed and detailed drilled shaft foundations as part of the overall standard design. The standard foundation details are incorporated with the OSS Standard Design Drawings for these structures and are available on the BOS website.

Single drilled shaft OSS Standard Design Drawings for use with contractor designed full span and cantilever 2-chord truss and monotube OSS are also available on the BOS website.

WisDOT has no standard foundation design details for alternate foundation types and the selected alternative foundations would be required to be individually designed and reviewed by BOS.

39.5.2.2 Design Parameters Used for Standard Foundation Design

Standard dual and single drilled shaft foundation designs were developed in accordance with applicable requirements of Section 10 of the *AASHTO LRFD Bridge Design Specifications.*

The standard foundation designs are based on the following design parameters:

- Total Unit Weight = 125 pcf
- Granular Soil Profile: Internal Angle of Friction = 24 degrees, or
- Cohesive Soil Profile: Undrained Shear Strength = 750 psf
- Soil and drilled shaft downward resistance factor ϕ =1.0 [1]
- Drilled shaft uplift resistance factor ϕ =0.8 [1]
- Depth of water table assumed 10 feet below the ground surface
- Soil side resistance is considered fully effective to the top of the drilled shaft or top of ground surface, whichever is the lower elevation.
- Lateral deflection at the top of the foundation limited to 1-inch at the Service I Limit State

Note 1: Resistance factors per AASHTO 10.5.3.3 assuming the drilled shaft design is governed by the wind load combination which is an Extreme Event load combination.

WisDOT policy item:
Design of standard sign structure foundations assumes soil side resistance is fully effective to the top of the drilled shafts for full span 4-chord OSS foundations and to within 3 feet below the lowest ground surface for all other OSS foundations. This is a deviation from AASHTO 10.8.3.5 1b.

Use of the standard foundations requires that the in-situ soils parameters at the site meet or exceed the assumed soil design parameters noted above. Soil parameters were selected to be sufficiently conservative to cover most sites across the state. Designers should contact the Region Soils Engineer or the Geotechnical Consultant to assist in the evaluation of the subsurface conditions compared to the assumed soil parameters. An assessment can also be made by checking nearby borings and as-built drawings of nearby existing structures, and similar sources. If there is reason to suspect weaker soils or that shallow bedrock is present, OSS specific soil borings should be obtained to confirm in-situ soil properties meet or exceed the assumed parameters used for the standard designs. If these site-specific soil properties do not meet the above minimums, a special individual foundation design will be required using actual soil parameters determined from a subsurface investigation per 39.5.3.

39.5.3 Standard Base Reactions for Non-Standard Foundation Design

There may be instances when a Standard Design sign structure is used in conjunction with a non-standard foundation, for reasons detailed in 39.4.5. Contact Bureau of Structures to obtaining the Standard Design or Contractor Designed sign structure base reactions that were used in developing the standard foundations.

39.5.4 Subsurface Investigation and Information

No subsurface investigation/information is necessary for the use of WisDOT standard foundations. Appropriate subsurface information is necessary for any non-standard OSS or situation that is outside any of the standard design ranges of applicability which requires an individual foundation design to be performed.

There may be several methods to obtain the necessary subsurface soil properties for a custom, individual foundation design, as described below:

- In areas of fill soils, the borrow material is usually unknown. The designer should use their best judgment as to what the imported soils will be. Standard compaction of this material can be assumed.

- The designer may have a thorough knowledge of the general soil conditions and properties at the site and can reasonably estimate soil design parameters.

- The designer may be able to use information from nearby borings. Judgment is needed to determine if the conditions present in an adjacent boring(s) are representative of those of the site in question.

- If the designer cannot reasonably characterize the subsurface conditions by the above methods, a soil boring and Geotechnical report (Site Investigation Report) should be completed. Necessary soil design information includes soil unit weights, cohesions, phi-angles and location of water table.

Designers, both internal and consultant, should also be aware of the potential of high bedrock, rock fills and the possible conflict with utilities and utility trenches. Conservative subsurface design parameters are encouraged.

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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.

40.2 History

40.2.1 Concrete

Prior to 1975, all concrete structures were designed by the Service Load Method. The allowable design stress was 1400 psi based on an ultimate strength of 3500 psi except for deck slabs. In 1965, the allowable stress for deck slabs on stringers was reduced from 1400 to 1200 psi. The reason for the change was to obtain thicker concrete decks by using lower strength. The thicker deck was assumed to have more resistance to cracking and greater durability. During this time no changes were made in the physical properties of the concrete or the mixing quantities until 1974 when more cement was added to the mix for concrete used in bridge decks.

In 1980, the use of set retarding admixtures was required when the atmospheric temperature at the time of placing the concrete is 70°F or above; or when the atmospheric temperature is 50°F or above and it is expected that the time to place the concrete of any span or pour will require four or more hours. Retarding admixtures reduce concrete shrinkage cracking and surface spalling. Also, during the early 1980's a water reducing admixture was provided for Grades A and E concrete mixes to facilitate workability and placement.

40.2.2 Steel

Prior to 1975, Grade 40 bar steel was used in the design of reinforced concrete structures. This was used even though Grade 60 bars may have been furnished on the job. Allowable design stress was 20 ksi using the Service Load Method and 40 ksi using the Load Factor Method.

40.2.3 General

In 1978, Wisconsin Department of Transportation discovered major cracking on a two-girder, fracture critical structure, just four years after it was constructed. In 1980, on the same structure, major cracking was discovered in the tie girder flange of the tied arch span.

This is one example of the type of failures that transportation departments discovered on welded structures in the 1970's and '80's. The failures from welded details and pinned connections led to much stricter standards for present day designs.

All areas were affected: Design with identification of fatigue prone details and classifications of fatigue categories (AASHTO); Material requirements with emphasis on toughness and weldability, increased welding and fabrication standards with licensure of fabrication shops to minimum quality standards including personnel; and an increased effort on inspection of existing bridges, where critical details were overlooked or missed in the past.

Wisconsin Department of Transportation started an in-depth inspection program in 1985 and made it a full time program in 1987. This program included extensive non-destructive testing. Ultrasonic inspection has played a major role in this type of inspection. All fracture critical structures, pin and hanger systems, and pinned connections are inspected on a 72-month cycle.

40.2.4 Funding Eligibility and Asset Management

Nationally, MAP-21 (2012) and the FAST Act (2015) have moved structures asset management to a more data-driven approach. Funding restrictions with regards to Sufficiency Rating, Structural Deficiency, and Functional Obsolescence have been removed or significantly revised. In place of these past restrictions, MAP-21 requires the development and approval of a statewide Transportation Asset Management Plan (TAMP). A key part of the WisDOT TAMP is the Wisconsin Structures Asset Management System (WiSAMS).

WiSAMS is being developed as a planning tool, which analyzes current structure inspection data, projects future deteriorated structure condition, and applies Chapter 42-Bridge Preservation to recommend appropriate structure work actions at the optimal time. WiSAMS is a tool for regional and statewide programming, and is not designed as an in-depth scoping tool. WiSAMS may provide an estimate of the appropriate work action, but an in-depth evaluation of the actual structure condition and appropriate scope of work (SSR) and consideration of other non-structural project factors (e.g. cost and functionality) is still required.

In Wisconsin, the Local Bridge Program, through State Statute 84.18 and Administrative Rule Trans 213, is still tied to historic FHWA classifications of Sufficiency Rating, Structural Deficiency, and Functional Obsolescence.

40.3 Bridge Replacements

Bridge preservation and rehabilitation is preferred over bridge replacement if the final structure provides adequate serviceability. Ideal bridge preservation strategy is explained in Chapter 42-Bridge Preservation. This guide should be followed as closely as possible, considering estimated project costs and funding constraints.

See Faculties Design Manual (FDM) 11-40-1.5 for policies regarding necessary bridge width* and structural capacity.

* If lane widening is planned as part of the 3R project, the usable bridge width should be compared to the planned width of the approaches after they are widened.

40.4 Rehabilitation Considerations

As a structure ages, rehabilitation is a necessary part of ensuring some level of acceptable serviceability; however, structure preservation as explained in Chapter 42-Bridge Preservation should be followed as closely as possible, considering estimated project costs and funding constraints.

The first consideration for any bridge rehabilitation decision is whether its geometry and load carrying capacity are adequate to safely carry present and projected traffic. Information which is helpful in determining structure adequacy includes structure inspection history, inventory data, traffic projections, maintenance history, capacity and route designations. The methods of rehabilitation are based on the type of structures, existing condition or rating information, the preliminary details of rehabilitation, and traffic control costs. These are important factors in considering rehabilitation options such as either a deck protection system or a deck replacement.

WisDOT policy item:

Rate the bridge using LFR provided it was designed using ASD or LFD. There are instances, however, where the LRFR rating of an existing bridge is beneficial (e.g. There is no M/M_u reduction to shear capacity using LRFR, which can affect longer-span steel bridges). Please contact the Bureau of Structures Development Section if using LRFR to rate bridges designed using ASD or LFD. Bridges designed using LRFD must be rated using LRFR.

The Regions are to evaluate bridge deficiencies when the bridge is placed in the program to ensure that rehabilitation will remove all structural deficiencies. Bureau of Structures (BOS) concurrence with all proposed bridge rehabilitation is required. See FDM 11-40-1.5 for policies regarding bridge rehabilitation.

Deck removal on prestressed girders is a concern as the contractors tend to use jackhammers to remove the deck. Contractors have damaged the top girder flanges using this process either by using too large a hammer or carelessness. With the wide-flange sections this concern is amplified. It is therefore suggested that the contractor saw cut the slab to be removed longitudinally close to the shear connectors. With the previously applied bond breaker, the slab should break free and then the contractor can clear the concrete around the shear connectors. Saw cutting needs to be closely monitored as contractors have sawn through steel girder flanges by not watching their blade depth.

In the rehabilitation or widening of bridge decks, it is often necessary to place concrete while traffic uses adjacent lanes. There has been considerable concern over the effects of traffic-induced vibrations on the quality of the concrete and its bond to existing concrete and its embedded reinforcing steel. Wisconsin bridge construction experience indicates that there are many cases where problems have occurred during deck pours with adjacent traffic. Consideration should be given to prohibiting traffic during the deck pour until concrete has taken its initial set.

40.5 Deck Overlays

As a bridge deck ages, preservation and rehabilitation techniques are necessary to maximize the life of the deck and ensure a level of acceptable serviceability. Overlays can be a useful tool to extend the service life of structures. This section discusses several overlay methods, considerations, and guidelines for deck overlays. The provided information is intended for deck-girder structures and may be applicable for slab structures. Slab structures may have different condition triggers and may warrant additional considerations.

The following criteria should be met when determining if an overlay should be used:

- The structure is capable of carrying the overlay dead load
- The deck and superstructure are structurally sound
- The desired service life can be achieved with the considered overlay and existing structure
- The selected option is cost effective based on the anticipated structure life and funding constraints

Decks deteriorate at different rates depending on many factors, including deck materials, material quality, construction quality, structure geometry, exposure to deicing agents, and traffic demands. Additionally, there is a wide variance in the amount of structure preservation techniques utilized by different regions. While the deck age can be a useful parameter, it should not be the primary consideration for determining the eligibility of overlays. Recommended preservation techniques should rely heavily on quality inspection data to determine the appropriate course of action. For more information related to preservation techniques and practices, refer to Chapter 42-Bridge Preservation.

Overlays can be an effective tool to maximize the life of the deck. Figure 40.5-1 illustrates a possible preservation scenario using deck deterioration curves showing approximate deck NBI ratings at which the overlays would occur, and the benefit of performing these overlays. This scenario assumes that the underside of deck deterioration is significantly reduced due to the preservation techniques performed on the top side of the deck.

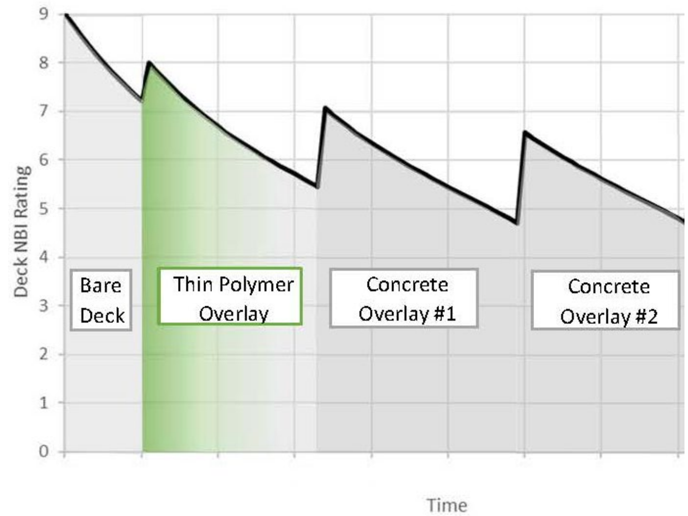

Figure 40.5-1
Deck Deterioration Curve

40.5.1 Overlay Methods

There are several commonly used overlay methods for the preservation and rehabilitation of decks. Generally, thin polymers overlays are recommended as preventative maintenance for decks with a minimal amount of deck distress. Ideally, thin polymer overlays are applied within the first couple of years to limit chloride infiltration. For decks with distress, the existing deck is typically milled and repaired with a low-slump concrete overlay as part of a more extensive bridge rehabilitation effort. For decks nearing replacement, asphaltic overlays maybe a cost effective option to improve ride quality. Refer to the following sections and Table 40.5-1 and Table 40.5-2 for a list of common overlay methods and additional information.

40.5.1.1 Thin Polymer Overlay

A thin polymer overlay (TPO) is expected to extend the service life of a bridge deck for 7 to 15 years. This overlay adds minimal dead load to the existing structure while providing an impermeable surface to prevent chlorides from infiltrating the deck. It can also be used to improve or restore friction on bridge decks.

In general, thin polymer overlays are defined as 1-inch thick or less overlays consisting of a polymer binder with aggregates and can be placed either as a multi-layer, slurry, or premixed system. Typical polymer binders are either epoxy, polyester, or methacrylate based. For WisDOT applications, TPO’s consist of a two-layer, two-component epoxy polymer in conjunction with natural or synthetic aggregates for a 1/4-inch minimum total thickness. For dead load purposes, use 5 psf for thin polymer overlays. Refer to the approved products list for a list of pre-qualified polymer liquid binders.

The following items are associated with repairing distressed deck areas as shown in Figure 40.5-3:

Preparation Decks Type 1 – The removal of existing patches and unsound concrete only to a depth that exposes 1/2 of the peripheral area of the top or bottom bar steel in the top mat of reinforcement. Care should be taken to limit damaging sound concrete.

Preparation Decks Type 2 – The removal of existing unsound concrete below the limit of the type 1 removal described above. One inch below the bottom of the top or bottom bar steel in the top mat of reinforcement is the minimum depth of type 2 removal.

Full-Depth Deck Repair – The complete removal of existing concrete.

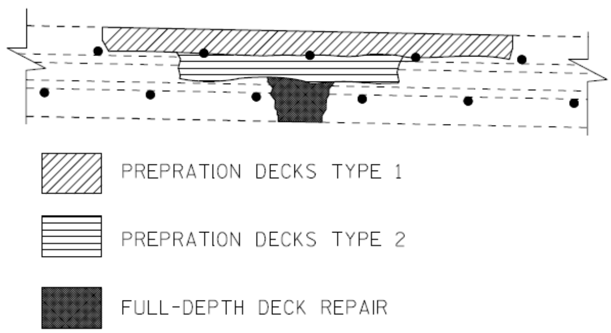

Figure 40.5-3
Deck Repairs

Deck Patches

Portland cement concrete is the preferred patch material. This material is easy to work with and very economical. When traffic impacts warrants, other materials may be considered. For concrete overlays, Type 1 and Type 2 deck patch repairs should be filled during the concrete overlay placement. Full-depth deck repairs should not be filled during the concrete overlay placement, but rather filled and curing a minimum of 24 hours before placing the concrete overlay. For other overlays, concrete repairs are usually properly cured prior to placing the overlay.

For minimal traffic impacts, a rapid-set material may be used for deck patches on asphaltic and thin polymer overlays. When repair quantities are minimal, distress areas less than 5% of the entire deck area, PPC overlays may use PPC to fill deck repairs prior overlay placement. See Table 40.5-1 for typical deck patch materials. Refer to the approved products list for a list of pre-qualified rapid setting concrete patch materials and their associated restrictions.

Surface Removal and Surface Preparation

Overlays require a properly prepared deck to achieve the desired bond strength. The following techniques are used for deck surface removal and preparations for an overlay:

Air cleaning – A preparation process to remove loose materials with compressed air. This process is intended to remove any material that may have gathered after the use of surface or concrete removal processes. This process is performed just prior to installing the overlay.

Water blasting (pressure or power washing) - A preparation process used to remove loose materials using low to high pressure water (5,000 psi to 10,000 psi). This process is beneficial as it keeps down dust and can remove loose particles.

Sand blasting – A surface removal process to remove loose material, foreign material, and loose concrete with sand material.

Shot blasting – A surface removal process to remove loose material, foreign material, and loose concrete by propelling steel shot against the concrete surface. This process also provides a roughen surface texture for improved bonding for overlays. Note: TPO's and PPC overlays provisions required a concrete surface profile meeting CSP-5 prior to overlay placement. This surface profile can be achieved using medium to medium-heavy shot blast.

40.5.5 Preservation Techniques

The following are some of the common activities being used to preserve decks and overlays:

- Deck cleaning (sweeping and power washing)
- Deck sealing/crack sealing
- Joint cleaning
- Joint repairs
- Deck patching

For additional preservation techniques and information refer to Chapter 42-Bridge Preservation.

40.5.5.1 Deck Sealing

Deck sealing has been found to be a cost-effective tool in preserving decks and overlays. In general, deck treatments should be applied as early as possible and re-applied thereafter. The frequency of deck sealing is dependent on the roadway traffic volume. Decks are to be sealed at initial construction and then resealed at the frequency shown in Table 40.5-3**.** Decks are to be resealed twice prior to applying a thin polymer overlay. Crack sealing should be considered as a potential combined treatment when deck sealing.

Roadway ADT	Deck Sealing Frequency
ADT < 2,500	4 – 5 years
2,500 <= ADT < 6,500	4 years
6,500 <= ADT < 15,000	3 years
ADT >= 15,000	2 years*

*In place of deck sealing, a thin polymer overlay is recommended within 2 years of deck construction. Use of the thin polymer overlay at this time will help minimize traffic impacts related to deck preservation work.

Table 40.5-3
Deck Sealing Frequency

Thin polymer overlays can be used in lieu of resealing the deck on a project-to-project basis with BOS approval. Approval occurs through the structure certification process. Some examples where TPOs might be used instead of deck sealing are where heavy snowmobile traffic is expected or when the safety certification provides justification for enhanced friction surface treatment.

40.5.6 Other Considerations

- Bridges with Inventory Ratings less than HS10 after rehabilitation shall not be considered for overlays, unless approved by the Bureau of Structures Design Section.
- Inventory and Operating Ratings shall be provided on the bridge rehabilitation plans.
- Verify the desired transverse cross slope with the Regions as they may want to use current standards.
- On continuous concrete slab bridges with extensive spalling in the negative moment area, not more than 1/3 of the top bar steel should be exposed if the bar ends are not anchored. This is to maintain the continuity of the continuous spans and should be stated on the final structure plans. If more than 1/3 of the steel is exposed and the bar ends are not anchored, either adjacent spans must be shored or a special analysis and removal plan are required. Reinforcement shall be anchored using Portland cement concrete.
- Asphaltic overlays should not be considered on a bridge deck which has a longitudinal grade in excess of four percent or an extensive amount of stopping and starting traffic.
- All full-depth repairs shall be made with Portland cement concrete.
- Joints and floor drains should be modified to accommodate the overlay
- Concrete chloride thresholds – Chloride content tests measure the chloride ion concentrations at various depths. Generally, research has shown initiation of corrosion

is expected when the chloride content is between 1 to 2 lbs/CY in concrete for uncoated bars and 7 to 12 lbs /CY for epoxy coated bars at the reinforcement. These limits are referred to as the threshold for corrosion. Threshold limits do not apply to stainless steel rebar.

When the chloride ion content is greater than 0.8 lbs/CY in concrete for uncoated bars and 5 lbs /CY for epoxy coated bars at the reinforcement depth, measures should be considered to limit additional chloride infiltration.

- See Chapter 6-Plan Preparation and Chapter 40 Standards for additional guidance.

- Refer the standard details for the most current bid items.

- Overlay transitional areas should be used and coordinated when accommodating profile differences. These transitions are intended to improve ride quality and protect against snowplow damage. Ideally, transitions are placed such that the overlay thickness remains constant, which requires a tapered removal of the existing surface over a sufficient distance. For profile adjustments 1 1/2-inch or greater, transitional areas should consider a minimum taper rate of 1:250 for low-speed applications (RSD< 50 mph) and for high-speed applications up to a 1:400 taper rate. Typically, thicker profile adjustments are provided off the bridge deck and are coordinated by the roadway designer. For profile adjustments less than 1 1/2-inch, a minimum rate of 1:250 may be used regardless of the roadway design speed. For a 3/4-inch minimum PPC overlay, provide a 16-feet minimum transition length. For a 1/4-inch TPO overlay, a 3-feet minimum transition length is sufficient. See Chapter 40 Standards for additional guidance.

40.5.7 Past Bridge Deck Protective Systems

In the past, several bridge deck protective systems have been employed on the original bridge deck or while rehabilitating the existing deck as described in 17.8. The following systems have been used to protect bridge decks:

- Epoxy coated deck reinforcement – Prior to the 1980's, uncoated (black) bars were used throughout structures, including bridge decks. Criteria for epoxy coated reinforcement was first introduced in 1981 as a deck protective system. At first, usage was limited to the top mat of deck reinforcement. By 1987, coated bars were required in the top and bottom mats for high volume roadways (ADT > 5000). By 1991, coated bars were required for all State bridges and on some local bridges (ADT > 1000). Currently, use of epoxy coated deck reinforcement is required on all bridge decks.

- Asphaltic overlay with Membranes – Use of this overlay system was largely discontinued in the 1990's.

- High Performance Concrete (HPC) - Use of HPC has been limited to Mega Projects.

- Thin Polymer Overlays – Use of this overlay system is currently being used.

40.21 References

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Prestressed Girder Superstructure

Primary Work Concept	Secondary Work Concept	Structure Year
New Construction	---	Year 0
Reseal Deck	---	Year 4
Reseal Deck	---	Year 8
Thin Polymer Overlay	---	Year 12
Thin Polymer Overlay	---	Year 22
Concrete Overlay and New Joints	• Substructure repair • Superstructure repair	Year 47
Deck Replacement	• Substructure repair • Superstructure repair	Year 67
Reseal Deck	---	Year 71
Reseal Deck	---	Year 75
Thin Polymer Overlay	---	Year 79
Thin Polymer Overlay	• Substructure repair • Superstructure repair	Year 89
Bridge Replacement	---	Year 100

Steel Girder Superstructure

Primary Work Concept	Secondary Work Concept	Structure Year
New Construction	---	Year 0
Reseal Deck	---	Year 4
Reseal Deck	---	Year 8
Thin Polymer Overlay	---	Year 12
Thin Polymer Overlay	---	Year 22
Concrete Overlay and New Joints	• Spot/zone painting • Substructure repair • Superstructure repair	Year 47
Deck Replacement	• Complete painting • Substructure repair • Superstructure repair	Year 67
Reseal Deck	---	Year 71
Reseal Deck	---	Year 75
Thin Polymer Overlay	---	Year 79
Thin Polymer Overlay	• Spot/zone painting • Substructure repair • Superstructure repair	Year 89
Bridge Replacement	---	Year 100

Concrete Slab Superstructure

Primary Work Concept	Secondary Work Concept	Structure Year
New Construction	---	Year 0
Reseal Slab	---	Year 4
Reseal Slab	---	Year 8
Thin Polymer Overlay	---	Year 12
Thin Polymer Overlay	---	Year 22
Concrete Overlay and New Joints	• Substructure repair • Superstructure repair	Year 47
Concrete Overlay and New Joints	• Substructure repair • Superstructure repair	Year 67
Bridge Replacement	---	Year 87

For all other superstructure types or in-service structures, consult BOS Bridge Management Unit for direction.

41.6.6.2 Discount Rate

WisDOT policy item:

A discount rate of 5% shall be used for cost-benefit analysis.

This value was determined based on analysis conducted by DTIM and is Department policy.

41.6.7 User Delay

WisDOT policy item:

For the purposes of cost-benefit analysis, user delay shall be addressed per direction in the WisDOT Facilities Development Manual (FDM).

User delay can have a dramatic impact on the results of a cost-benefit analysis and must be considered based on Department policy.

Table of Contents

42.1 Overview

This chapter provides goals, objectives, measures, and strategies for the preservation of bridges. This chapter contains criteria that is used to identify condition based and cyclical preservation, maintenance, and improvement work actions for bridges. Bridge preservation is defined as actions or strategies that prevent, delay, or reduce deterioration of bridges or bridge elements; restore the function of existing bridges; keep bridges in good or fair condition; and extend their service life. Preservation actions may be cyclic or condition-driven. (1)

A successful bridge program will seek a balanced approach to preservation, rehabilitation, and replacement. One measure of success is to maximize the life of structures while minimizing the life cycle cost. Preservation of structures is one of the strategies in maximizing the effectiveness of the overall bridge program by retarding the rate of overall deterioration of the bridges.

Bridges are key components of our highway infrastructure. Wisconsin has over 14,000 bridges, of which about 37% are owned by WisDOT. The average age of these bridges in 2019 is 38 years. The aging infrastructure is expected to deteriorate faster in the coming decades with increased operational demand unless concerted efforts are taken to preserve and extend their life. In addition, the state bridge infrastructure is also likely to see an increased funding competition among various highway assets. As a result, WisDOT must emphasize a concerted effort to preserve and extend the life of bridge infrastructure while minimizing long-term maintenance costs.

This chapter provides WisDOT personnel and partners with a framework for developing preservation programs and projects using a systematic and consistent process that reflects the environment and conditions of bridges and reflects the priorities and strategies of the Department.

A well-defined bridge preservation program will also help WisDOT use federal funding (2) for Preventative Maintenance (PM) activities by using a systematic process of identifying bridge preservation needs and its qualifying parameters as identified in FHWA's Bridge Preservation Guide (1). This chapter will promote timely preservation actions to extend and optimize the life of bridges in the state.

Objective	Target/Goals	Performance Measure
Maintain bridges in good or fair condition	95% of bridges	Percentage of bridge in good or fair condition (NBI rating 5 or higher)
Maintain bridge decks in good or fair condition	95% of bridge decks	Percentage of bridge decks in good or fair condition (NBI Rating 5 or higher)
Maintain effective expansion joints that do not leak	85% of bridges with strip seal joints that are effective in stopping leakage	Percent of a bridges with 90% of their strip seal expansion joints in condition state 2 or better (effective joint)
Maintain coated steel surfaces in condition state 2 or better	90% of coated steel surfaces	Percentage of coated steel surfaces in condition state 2 or better (effective)
Maintain bearings in condition state 2 or better	95 % of bearings in condition state 2 or better	Percentage of bearings in condition state 2 or better
Seal eligible concrete decks (NBI rating 6 or higher) with sealant every 3-5 years	Seal 20% eligible concrete decks	Number of decks sealed (sq. ft of deck area) each year during a 5-year period

Table 42.4-1
Objectives and Performance Measures

42.4.4 Preservation Program Benefits

Each objective and measure proposed in Table 42.4-1 is aimed at extending the life of the main bridge components by performing timely cyclical or condition-based (corrective) preservation actions. The cost of performing preservation actions is minor when compared to premature replacement or rehabilitation of bridge components. The benefits of each objective are discussed below:

- Maintaining 95% of bridge decks in good or fair condition is an asset management approach that should extend the service life of bridges and promote the MAP21 objectives. Experience has shown that bridges designed for a 100-year life expectancy should have decks that last 55 with progressive preservation activities though the life of the bridge deck. Appropriate corrective actions taken as part of deck preservation extends the bridge deck life significantly. The costs of such corrective actions are substantially less than the costs of prematurely replacing the decks.

- The objective of maintaining 85% of strip seals in good or fair condition will focus on a program that will help in minimizing the damage on bridge superstructure and substructure components. Leaking joints cause significant deterioration and damage to bridge components that include girders, bearings, and substructures. There is significant cost each year in repairing structural elements that have deteriorated prematurely as a result of leaking joints. Maintaining effective (non-leaking) strip seals can delay superstructure and substructure deterioration.

- Maintaining protective paint systems is important. The structural components of the steel bridges will corrode and lose load carrying capacity if left unprotected or partially-protected. Protective paint coatings systems should have a service life of 25-40 years for the protection of structural steel. The objective of maintaining 90% of coated steel surfaces in good or fair condition will aim at creating a paint program for extending the life of steel components up to 100 years.

- Bridge bearings are a key component. Bearings support bridge super structures and allow for expansion of the superstructure. Experience has shown that loss of lubrication, tipping, or corrosion of bearings can cause harm to the deck and superstructure. The proposed measure of keeping 95% of bearings in good or fair condition will help WisDOT maintain bridges in a state of good repair.

- Objective of sealing 20% of all eligible concrete decks at 5-year intervals will help delay deck deterioration and prolong deck life. Sealing decks every 3 to 5 years at a minor cost can delay deck deterioration by 10-12 years that will promote increased deck life.

42.5 Bridge Preservation Activities, Eligibility and Need Assessment Criteria

The bridge preservation activities shown below relate to deck, superstructure and substructure elements. Table 42.5-1 shows the most common bridge preservation activities that are considered cost effective when applied to the appropriate bridge at the appropriate time, as well as considered eligible for bridge preservation funding. Additionally, these activities together with the eligibility and prioritization criteria discussed in this section will form a basis to generate an eligibility list of bridges that are candidates for cyclical and condition based PM actions.

Bridge Component	Bridge Preservation Type	Activity Description	Preventive Maintenance Type	Action Frequency (years)
All	Preventive Maintenance	Sweeping, power washing, cleaning	Cyclical	1-2
Deck	Preventive Maintenance	Deck washing	Cyclical	1
		Deck sweeping		1
		Deck sealing/crack sealing		3-5
		Thin polymer (epoxy) overlays		7-15
		Drainage cleaning/repair	Condition Based	As needed
		Joint cleaning		
		Deck patching		1- 2
		Chloride extraction		1 -2
		Asphalt overlay with membrane		5-15
		Polymer modified asphalt overlay		10-15
		Joint seal replacement		10
		Drainage cleaning/repair		1
	Repair or Rehab Element	Rigid concrete overlays	Condition Based	As needed
		Structural reinforced concrete overlay		
		Deck joint replacement		
		Eliminate joints		
Super	Preventive Maintenance	Bridge approach restoration	Cyclical	2
		Seat and beam ends washing		2
	Repair or Rehab Element	Bridge rail restoration	Condition Based	As needed
		Retrofit rail		
		Painting		
		Bearing restoration (replacement, cleaning, resetting)		
		Superstructure restoration		
		Pin and hanger replacement		
		Retrofit fracture critical members		
Sub	Preventive Maintenance	Substructure restoration	Condition Based	As needed
		Scour counter measure		
		Channel restoration		

Table 42.5-1
Bridge Preservation Activities

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The first step is to perform a rating analysis to determine inventory and operating ratings. Based on the results of the rating analysis, a posting analysis should be performed when:

- The inventory rating factor is less than or equal to 1.0 (HS-20) – Emergency Vehicles (EVs) only, see Figure 45.10-5; or
- The operating rating factor is less than or equal to 1.2 (HS-24) – Specialized Hauling Vehicles (SHVs) only, see Figure 45.10-2; or
- The operating rating factor is less than or equal to 1.0 (HS-20) for all other posting vehicles.

An emergency vehicle analysis is performed to determine whether a bridge can safely carry emergency vehicles, which may exceed legal weight limits in place for other vehicles. A posting analysis is performed to determine whether a bridge can safely carry other legal-weight traffic. Both analyses are performed at the operating level. See 45.10 for more information.

Permit analysis is used to determine whether or not over legal-weight vehicles may travel across a bridge. See 45.11 for more information on over-weight vehicle permitting.

A flow chart outlining this approach is shown in Figure 45.3-2. The procedures are structured to be performed in a sequential manner, as needed.

45.3.8.1 Load Factors for Load Factor Rating

See Table 45.3-5 for load factors to be used when rating with the LFR method. The nominal capacity, C, is the same regardless of the rating level desired.

For emergency vehicles, alternate live load factors determined in accordance with NCHRP Project 20-07 / Task 410 may be used. If alternate live load factors are used, this shall be noted in the Load Rating Summary Form, along with assumptions of one-way ADTT and emergency vehicle crossings per day.

LFR Load Factors		
Rating Level	A_1	A_2
Inventory	1.3	2.17
Operating	1.3	1.3

Table 45.3-5
LFR Load Factors

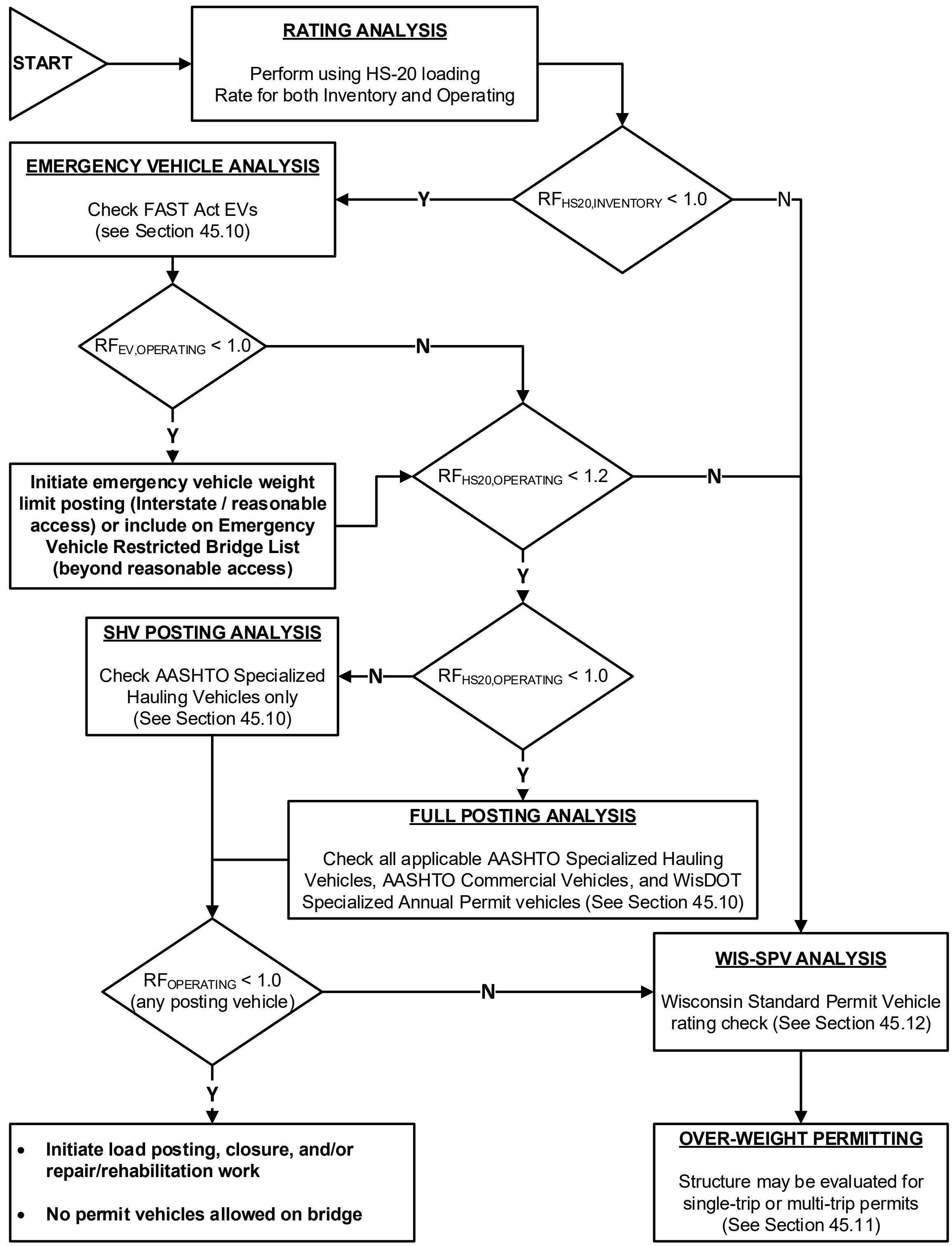

Figure 45.3-2

Load Factor Rating and Allowable Stress Rating Flow Chart

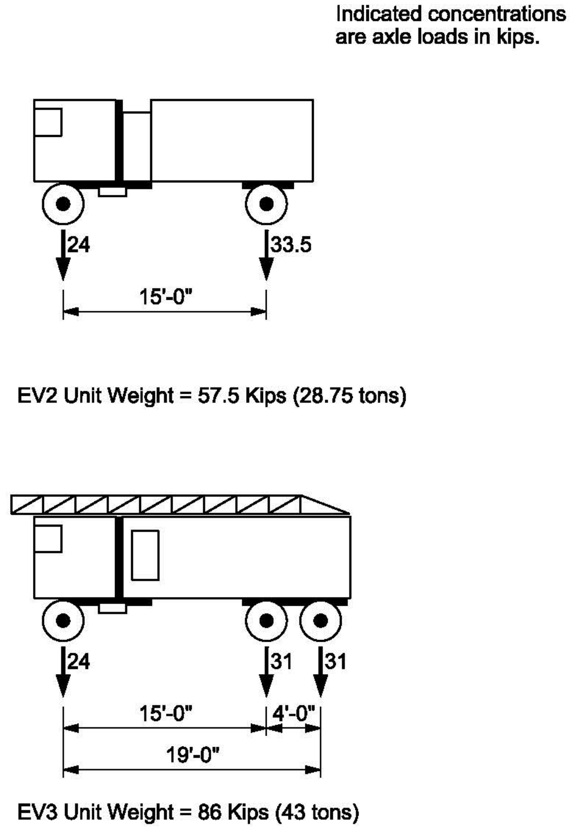

Figure 45.10-5
Emergency Vehicle Load Models

45.10.3 Load Posting Analysis

All posting vehicles shall be analyzed at the operating level. A load posting analysis is required when the calculated rating factor at operating level for a bridge is:

- Less than 1.0 for HL-93 loading using LRFR methodology.
- Less than 1.0 for HS-20 loading using LFR/ASR methodology; or
- Less than or equal to 1.2 for LFR/ASR methodology (SHV analysis only)

A load posting analysis is very similar to a load rating analysis, except the posting live loads noted in 45.10.2 are used instead of typical LFR or LRFR live loading.

If the calculated rating factor at operating is less than 1.0 for a given load posting vehicle, then the bridge shall be posted, with the exception of the Wis-SPV. For State Trunk Highway Bridges, current WisDOT policy is to post structures with a Wisconsin Standard Permit Vehicle (Wis-SPV) rating of 120 kips or less. If the RF ≥ 1.0 for a given vehicle at the operating level, then a posting is not required for that particular vehicle.

A bridge is posted for the lowest restricted weight limit of any of the standard posting vehicles. To calculate the capacity, in tons, on a bridge for a given posting vehicle utilizing LFR, multiply the rating factor by the gross vehicle weight in tons. To calculate the posting load for a bridge analyzed with LRFR, refer to 45.10.3.2.

Posting or weight limit analysis for emergency vehicles occurs separately; it is required when the calculated rating factor at inventory level for a bridge is:

- Less than 0.9 for HL-93 loading using LRFR methodology; or
- Less than 1.0 for HS-20 loading using LFR/ASR methodology.

If the calculated rating factor at operating rating is less than 1.0 for a given emergency vehicle, then the bridge shall have an emergency vehicle-specific weight limit restriction, as follows:

- If RF_{EV2} < 1.0 and RF_{EV3} < 1.0
  - Single Axle = Minimum (RF_{EV2} x 16.75 tons, RF_{EV3} x 31 tons)
  - Tandem = Minimum (RF_{EV2} x 28.75 tons, RF_{EV3} x 31 tons)
  - Gross = Minimum (RF_{EV2} x 28.75 tons, RF_{EV3} x 43 tons)
- If only RF_{EV2} < 1.0
  - Single Axle = RF_{EV2} x 16.75 tons
  - Tandem = RF_{EV2} x 28.75 tons
  - Gross = RF_{EV2} x 28.75 tons
- If only RF_{EV3} < 1.0
  - Single Axle = Minimum (16 tons, RF_{EV3} x 31 tons)
  - Tandem = RF_{EV3} x 31 tons
  - Gross = RF_{EV3} x 43 tons

Sign postings may or may not be required for emergency vehicles, depending on their location. Refer to 45.10.4.

45.10.3.1 Limit States for Load Posting Analysis

For LFR methodology, load posting analysis should consider strength-based limit states only.

For LRFR methodology, load posting analysis should consider strength-based limit states, but also some service-based limit states, per Table 45.3-1.