Table of Contents

5.1 Factors Governing Bridge Costs

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

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

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

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

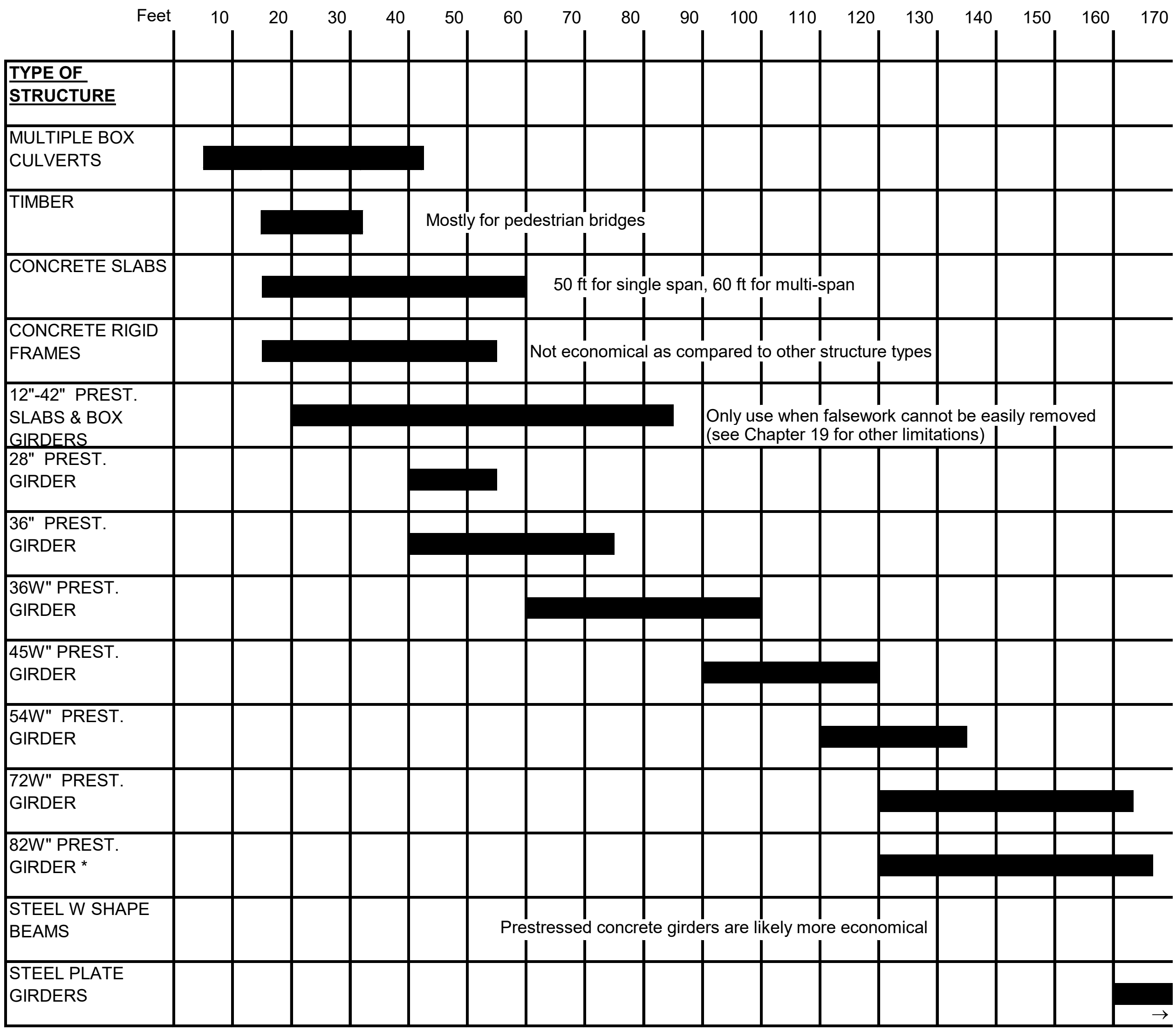
In these reports prestressed girder costs are grouped together because there is a small cost difference between girder sizes. Refer to unit costs. Concrete slab costs are also grouped together for this reason.

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

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

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

5.2 Economic Span Lengths



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

Figure 5.2-1
Economic Span Lengths

5.3 Contract Unit Bid Prices

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

- Bid Express
- AASHTOWare Project Estimator
- Estimating Tools website

5.4 Bid Letting Cost Data

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

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

5.4.1 2021 Year End Structure Costs

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

Table 5.4-1
Stream Crossing Structures

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

Table 5.4-2
Grade Separation Structures

5.4.2 2022 Year End Structure Costs

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

Table 5.4-3
Stream Crossing Structures

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

Table 5.4-4
Grade Separation Structures

5.4.3 2023 Year End Structure Costs

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

Table 5.4-5
Stream Crossing Structures

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

Table 5.4-6
Grade Separation Structures

5.4.4 2024 Year End Structure Costs

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

Table 5.4-7
Stream Crossing Structures

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

Table 5.4-8
Grade Separation Structures

5.4.5 2025 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	15	85,589	18,478,491	98.27	215.90
Reinf. Conc. Slabs (Flat)	69	105,062	25,357,417	92.80	241.36
Reinf. Conc. Slabs (Haunched)	15	63,291	14,438,374	96.61	228.13
Prestressed Box Girder	1	1,515	449,174	147.02	296.48
Buried Slabs	1	3,928	726,304	81.26	184.90
Rolled Steel Girders	1	7,997	3,570,794	176.10	446.52

Table 5.4-9
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	24	263,429	45,433,499	85.39	172.47
Reinf. Conc. Slabs (Flat)	1	1,989	418,223	95.77	210.27
Reinf. Conc. Slabs (Haunched)	0	--	--	--	--
Steel Plate Girders	1	35,139	12,365,280	205.26	351.90

Table 5.4-10
Grade Separation Structures

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Table of Contents

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

All bridges and box culverts covered under this chapter shall be evaluated for the 100-year (Q100) or the 1% Annual Exceedance Probability (AEP) event. In floodplain management, this is also referred to as the Regional or Base flood. Design of bridges and box culverts over stream crossings is guided by requirements in Federal Highway Administration (FHWA) directives and the Cooperative Agreement between the Wisconsin Department of Transportation (DOT) and the 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 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.

All hydraulic sizing reports are required to be stamped and signed by a Professional Engineer (PE) registered within the State of Wisconsin. Supporting hydrologic and hydraulic computations and models must also have documentation of the PE in responsible charge of the analysis recorded.

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 criterion 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 superstructure replacement provided that the substructure and berm configuration remain unchanged, and the low chord elevation is not significantly lowered.

The designer should consider historical high-water elevations, Flood Insurance Studies and inundation potential 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 *Estimating Flood Magnitude and Frequency for Unregulated Streams in Wisconsin*[2] which considers the flooding potential 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 relate flood discharges to physical basin characteristics, namely, drainage area, wetland area, forest cover, herbaceous upland area, open water, precipitation intensity index, and soil hydraulic conductivity. 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 Project Site at Streamgage

An attachment to reference (2) above includes flood frequency discharges for 299 gaged sites computed using flood records through water year 2020. These flood frequency discharge estimates were generated using the Log-Pearson Type III (LP3) distribution method as described in Bulletin 17C 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]. Additional years of data are available from the USGS for some gaged watersheds. Flood frequency discharge estimates for these watersheds can be updated beyond water year 2020 using the same methodology as described above.

In addition to the LP3 method, reference (2) describes a theoretically improved estimate of flood discharge that combines the LP3 discharge estimate with the regression estimate for the gaged site. More details on this method can be found under the section titled "Estimating the Weighted Flood Discharge at a Streamgage."

8.2.3 Project Site at Ungaged Location on a Gaged Stream

If a project site is located on a stream with an existing streamgage (but is not at the gage itself), results obtained from the above regression equations can be combined with the flood discharge estimate at the gage to produce an improved peak flow estimate. More details are provided in the section titled "Estimating the Flood Discharge at an Ungaged Location on a Gaged Stream" in reference (2) as discussed above. This method is applicable only if the drainage area associated with the ungaged location is between 50 and 150 percent of the drainage area associated with the streamgage.

8.2.4 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

8.2.5 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. Velocities with potential to compromise slope or streambed stability are not acceptable and should be avoided. This threshold will vary depending on site geometry and local stream geomorphology.

8.3.1.2 Roadway Overflow

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

8.3.1.3 Bridge Skew

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

8.3.1.4 Backwater and High-water Elevation

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

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

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

8.3.1.5 Freeboard

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

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

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

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

- 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 scour potential at a 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 and shear stress through the bridge may be the greatest. This is known as the “incipient overtopping” event. Scour should be computed for the incipient overtopping flow event if the magnitude of flow is less than the scour design flow (typically this is the 100 year flood or greater).

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 outside the immediate project area may be developed using LiDAR or publicly available contour mapping. Stream portions of the cross sections should be developed based on inspection of photos and field survey data collected near the structure.

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 the most widely used computer tool 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.

   HEC-RAS supporting documentation can be found in section 8.5 references (7), (8) and (9). The HEC-RAS program and manuals can be downloaded from the U.S. Army Corps of Engineers web site: http://www.hec.usace.army.mil/software/hec-ras/. All hydraulic models using HEC-RAS software must be submitted in digital form in the public domain version of the model.

2. HY8

HY8 is a computer program that uses the FHWA culvert hydraulic analysis methods 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. HY-8 also includes the capabilities to analyze more complex situations, including buried inverts, tapered inlets, "broken-back" culverts, and energy dissipators. HY-8 can also be used to evaluate low-flow conditions for aquatic organism passage/stream connectivity. 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

Prior to developing a hydraulic model, the designer should determine whether the project is located within a regulatory floodplain, also referred to as FEMA Special Flood Hazard Area (SFHA). This determination can be made by viewing regulatory Flood Insurance Rate Maps (FIRMs) on FEMA Map Service Center Viewer at https://msc.fema.gov/portal/home and https://msc.fema.gov/portal/search

Regulatory floodplain information can also be found on WDNR's Storm Water, Waterway, and Wetland Permit Viewer (SW4P) at https://dnrmaps.wi.gov/H5/?Viewer=SW4P

Hydraulic models developed for structure analysis and design are expected to be one-dimensional, standard step-backwater, steady state flow, employing public domain software.

Projects Not Located in SFHA

Existing Conditions Model:

An existing conditions model is developed for the bridge/culvert site. The model will consist of a stream reach, cross sections, roadway embankment/bridge data. The existing conditions model and its results are used as the basis of comparison of water surface elevations for proposed conditions. It is uncommon for models to be available for use/modification for projects not located within regulated floodplains.

When practical, the hydraulic model should be geo-referenced to a known coordinate system--preferably the coordinate system used for development of roadway and structure plans.

Model extents: The model should include cross sections far enough upstream and downstream of the bridge such that the flow is fully effective flow. Flow contraction ratios are limited to 1:1 upstream of the bridge and flow expansion ratios are limited to 1:2 downstream of the bridge. Deviations from typical contraction and expansion ratios may be accepted but will require justification from the designer.

Flow information: All models should evaluate the 2-year and 100-year peak discharge rates as approved by the Bureau of structures. Boundary conditions should be based on either normal depth based on the slope water surface (friction slope) or a known downstream water surface elevation (most often the 10-yr water surface elevation of the receiving stream). The 5-year discharge rates should be included when a temporary bypass structure is planned for construction.

Cross sections: Hydraulic cross sections should be developed based on the best available topographic information, which may include a combination of LiDAR derived digital elevation model for overbanks, and site-specific survey data for the streambed elevations and existing structure elevations and bridge opening details. All cross sections should be normal (perpendicular) to the direction of flow, which may require "dog-legs" in the cross sections. Cross sections should fully contain effective flow, as truncating cross sections may artificially increase water surface elevations.

Roadway embankment: The roadway should be modeled to reflect the roadway profile along the highest point in the roadway cross section. The embankment needs to span the entire length of the upstream and downstream cross sections, not leaving any gaps. The roadway width is also required to determine the appropriate weir coefficient.

Bridge opening: The model should include the effective opening length normal to the direction of flow for all bridges in pressure flow, and for any bridge with a skew of 20 degrees or more for low flow conditions. The bridge model should include any sloping abutments and pier sizes and locations. Internal geometry editor may be used to represent stream channel elevations at the upstream and downstream faces of the bridge as appropriate.

Culvert: Information to be included for modeling a culvert includes culvert size, shape, number of barrels, material/roughness, length, invert elevations, inlet type, depth blocked (if any), roughness value to be used for blocked portion. When physical characteristics of multiple barrel culverts differ, they should be modeled independently.

Proposed Conditions Model:

It is recommended to duplicate the existing conditions model and make changes that correspond to the proposed conditions to the duplicated model. Only parameters that change between existing and proposed conditions should be modified.

In general, the flow parameters and cross section data should remain consistent between existing and proposed conditions. When the changes in bridge length and/or roadway overtopping conditions are minor, it is recommended to keep ineffective flow limits and elevations consistent between existing and proposed conditions.

Likely changes between existing and proposed conditions include culvert physical parameters, bridge opening geometry, pier type and location, sloping abutment protection, roadway profile and width.

<u>Projects Located in SFHA</u>

Design of all projects located within a regulatory floodplain (SFHA) shall meet the commitments made in the DOT/DNR Cooperative Agreement (link) and the spirit and intent of State Administrative Code NR 116. Primary considerations are:

1. No rise in base flood elevation (BFE)/100-year water surface elevation is allowed unless "appropriate legal arrangements" are made with the affected landowners and impacted municipality. A Conditional Letter of Map Revision (CLOMR) application may be required for projects that result in an increase in Base Flood Elevation.

2. Temporary structures used for carrying traffic and causeways used for construction access located in regulatory floodplains must be evaluated for the 100-year flood. Potential risks to insurable structures must be determined.

Zone A SFHA

A Zone A SFHA shows the boundaries of regulated floodplains that were determined through "approximate methods," meaning there are no detailed engineering hydrologic or hydraulic models. Modeling should be completed using the same methods and techniques as for projects not located within a SFHA. Modeling results will be communicated to DNR and Floodplain Zoning Administrators consistent with policy in FDM Chapter 20, and modeling information will be made available through the Highway Structure Information System (HSIS).

Zone AE SFHA

Zone AE SFHAs are regulated floodplains that are determined through detailed engineering and analysis methods, meaning that they are based on the results of hydrologic and hydraulic models. Commitments in the DOT/DNR Cooperative Agreement state that WisDOT projects located in Zone AE SFHA's use the FEMA model of record as the basis for hydraulic modeling. The most common modeling software used for Flood Insurance Models (FIS) models are HEC-RAS, HEC-2, WSPRO, and E431.

FEMA approved FIS models can be obtained from WDNR on their mapping application at https://dnrmaps.wi.gov/H5/?Viewer=SW4P. In some cases, models may need to be obtained from FEMA's Engineering Library. WDNR Region Water Management Engineers should be contacted with any questions related to existing effective models. A contact list can be found at https://dnr.wisconsin.gov/topic/FloodPlains/staff_flood.html.

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. Note that older models use the NGVD29 datum, but that the mapping on DNR's and FEMA's websites may be shown in NAVD88 datum. FIS reports may contain an average conversion factor used to convert elevations county-wide.

When a regulatory hydraulic model is in a legacy format (HEC-2, WSPRO, E431, etc.), a good-faith effort must be made to convert the model to a usable format. HEC-RAS can directly import HEC-2 data. Contact BOS with questions regarding model conversions.

When the design 100-year flow deviates from the regulatory 100-year flow, both values need to be analyzed. All hydraulic results on the plan sheets will be based on the approved design 100-year flow.

To meet the commitments made in the DOT/DNR Cooperative Agreement, analyses using regulatory models must include the following:

1. Duplicate Effective Model. This model re-runs the effective model with no changes or the minimum changes necessary for the model to run. When converting from HEC-2 to HEC-RAS, bridge widths may need to be shortened slightly and half the shortened width entered as the distance to the upstream cross section.

2. Corrected Effective Model. This model is the existing conditions model. Edits to the regulatory model should be limited to the immediate area around the project. Expected modifications may include roadway profile, bridge opening information, and streambed and overbank elevations. Manning's roughness values, expansion and contraction coefficients, and ineffective flow limits and elevations should remain the same as those in the regulatory model. The Corrected Effective Model may require elevation datum conversion depending on the age and the source of the regulatory model.

3. Proposed Conditions Model. This model modifies the existing conditions model to represent the proposed structure. Expected modifications may include roadway profile and bridge geometry.

All changes made to the models should be documented in the hydraulic sizing report and in the hydraulic model comment sections. The regulatory model should not be truncated.

Modeling results will be communicated to DNR and Floodplain Zoning Administrators consistent with guidelines in FDM Sections 20-55-40 and 20-55-75, and modeling information will be made available through the Highway Structure Information System (HSIS).

8.3.2.6.1 Bridge Hydraulics

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

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

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

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

8.3.2.6.2 Roadway Overflow

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

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

The weir discharge equation is:

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

Where:

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.

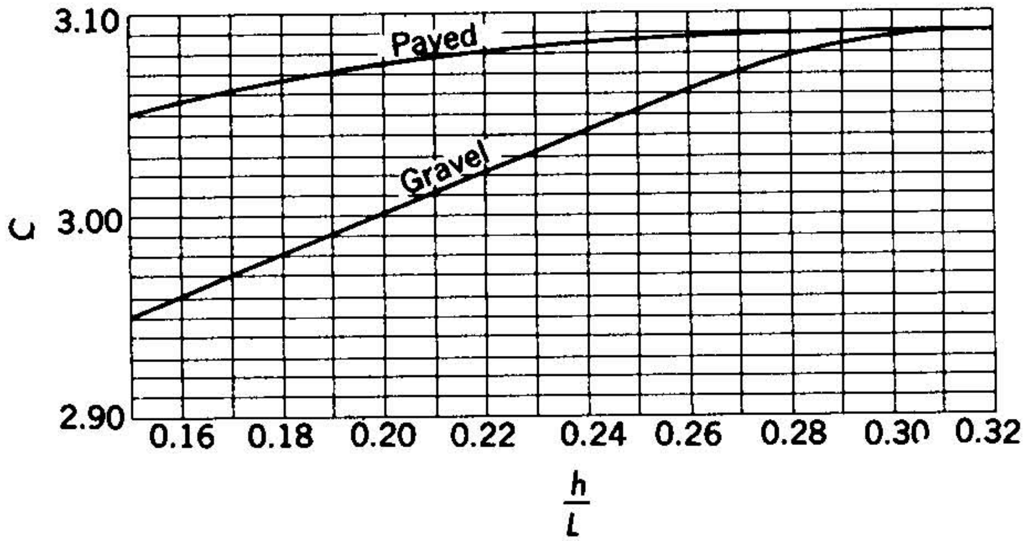


Figure 8.3-1

Discharge Coefficients, C_f, for Highway Embankments for H/L Ratios > 0.15

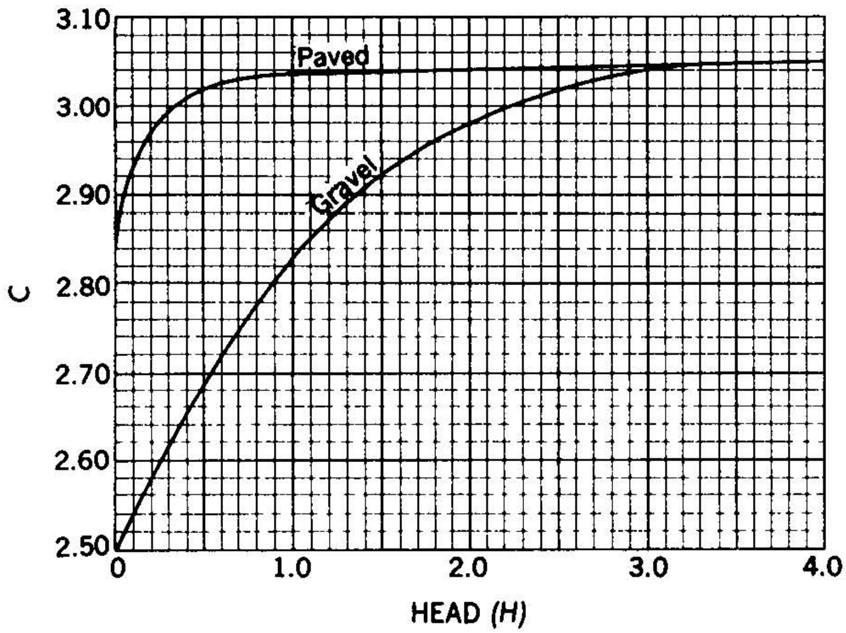


Figure 8.3-2
Discharge Coefficients, C_f, for Highway Embankments for H/L Ratios < 0.15

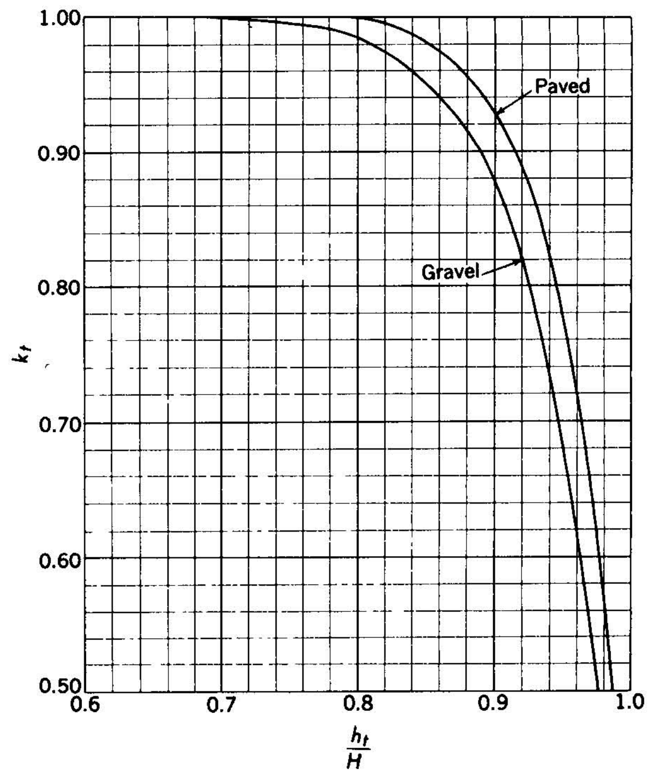


Figure 8.3-3
Definition of Adjustment Factor, k_t, for Submerged Highway Embankments

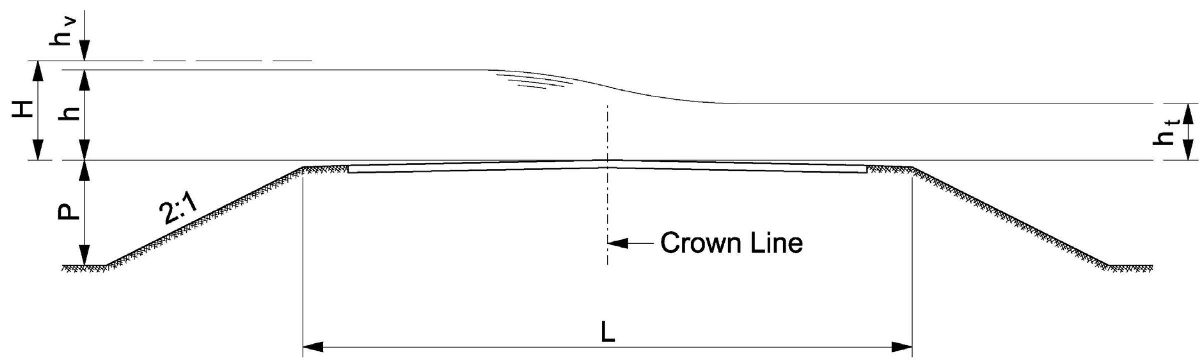


Figure 8.3-4
Definition Sketch of Flow Over Highway Embankment

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. Similarly, all bridges should also be assigned a Scour Vulnerability Code as described under Item B.AP.03 in the Specifications for the National Bridge Inventory: https://www.fhwa.dot.gov/bridge/snbi/snbi_march_2022_publication.pdf.

Hydraulic variables needed to complete a bridge scour analysis should be extracted from a 1D or a 2D hydraulic model. Water surface profile modeling tables highlighting pertinent variables should be included in documentation of the analysis. Scour calculations performed using the HEC RAS built-in calculators are unacceptable.

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 placed such that the hydraulic properties of the cross section are not influenced by flow contraction at 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. Long-term Degradation (LTD) 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.

Contraction scour should be computed using live bed, clear water or cohesive material equations. The appropriate equation is dependent on sediment properties in the scour zone and hydraulic characteristics of the bridge crossing. See 8.5 references (14) and (15) for discussion and methods of analysis.

If a pressure flow condition exists at the bridge opening, then vertical contraction scour must be evaluated. Pressure flow scour depth is the greater of either LTD + Contraction Scour (Live

Bed or Clear Water) or Vertical Contraction Scour. Vertical Contraction Scour should not be added to Live Bed or Clear Water Scour. 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 7.1)	
Shape of Pier Nose	K_1
(a) Square Nose	1.1
(b) Round Nose	1.0
(c) Circular Cylinder	1.0
(d) Group of Cylinders	1.0
(e) Sharp Nose	0.9

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

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

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

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

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

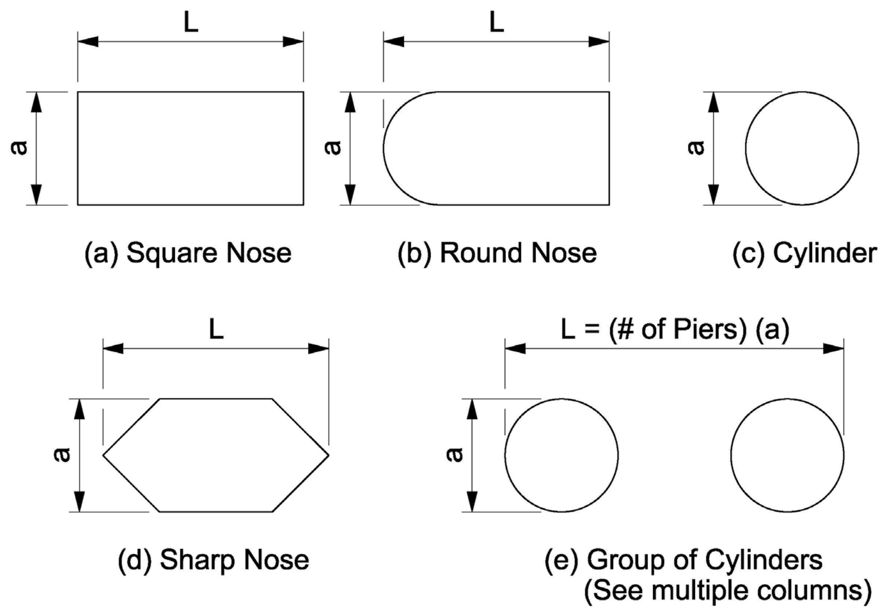


Figure 8.3-5
Common Pier Shapes

2. Abutment Scour Equations

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

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

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

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

Standard slope protection at stream crossings as detailed in WisDOT Bridge Manual (Chapter 15) is not considered a designed scour countermeasure and should not be factored into scour calculations. In general, new bridges should have foundations designed to withstand calculated scour without the need for designed scour countermeasures.

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.

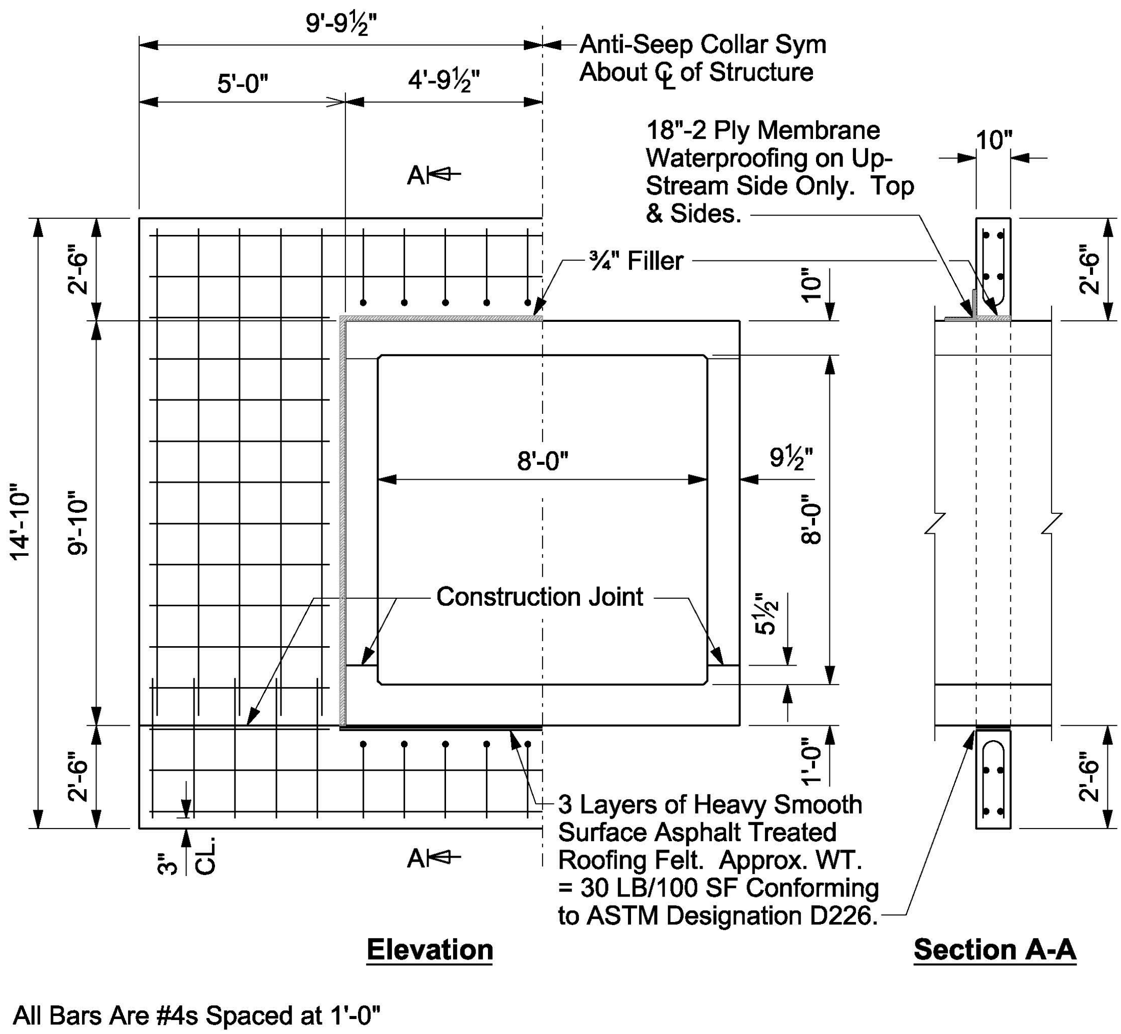


Figure 8.4-1
Anti-Seepage Collar

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 HDS No. 5, see 8.5 reference (13). It is highly recommended the designer first thoroughly study the methodologies presented in that publication.

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

8.4.2.5 Develop Hydraulic Model

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

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

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

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

1. Outlet Control Problem.

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

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

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

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

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

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

2. Inlet Control Problem.

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

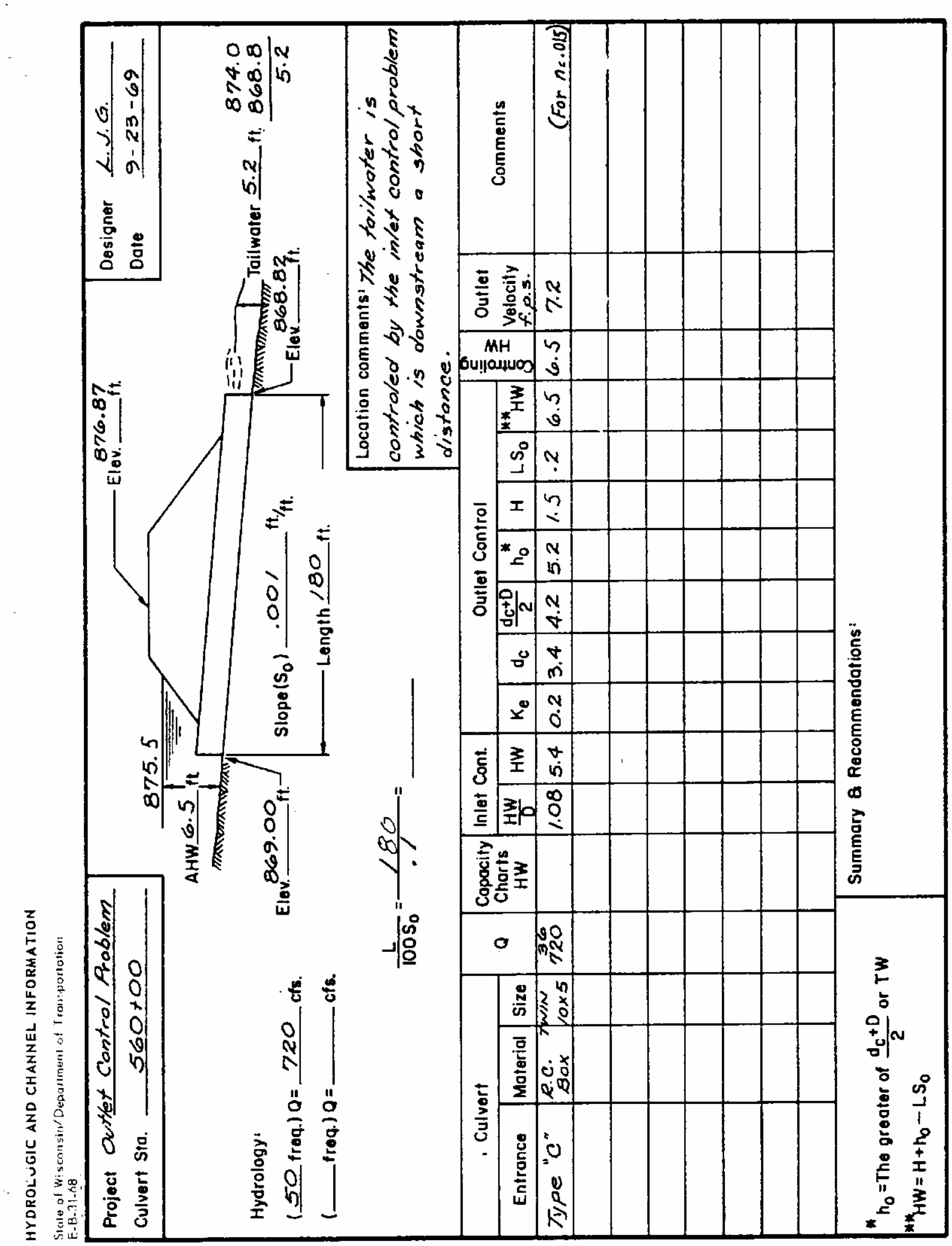
   Check for Inlet Control: For a Q/B value of 36 and a twin 10 x 5 box with type “C” inlet; HW/D = 1.08 from Figure 8.4-3.

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

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

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

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



HYDROLOGIC AND CHANNEL INFORMATION

State of Wisconsin/Department of Transportation
E-B-31-68

Project Outlet Control Problem
Culvert Sta. 560+00

Designer L.J.G.
Date 9-23-69

Hydrology:
(50 freq.) Q = 720 cfs.
(___ freq.) Q = ___ cfs.

$\frac{L}{100 S_0} = \frac{180}{.1} =$ ___

Location comments: The tailwater is controled by the inlet control problem which is downstream a short distance.

Culvert: Entrance	Culvert: Material	Culvert: Size	Q	Capacity Charts HW	Inlet Cont.: $\frac{HW}{D}$	Inlet Cont.: HW	Outlet Control: K_e	Outlet Control: d_c	Outlet Control: $\frac{d_c+D}{2}$	Outlet Control: h_o*	Outlet Control: H	Outlet Control: LS_o	Outlet Control: **HW	Controling 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)

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

Summary & Recommendations:

Figure 8.4-2
Culvert Computation Form

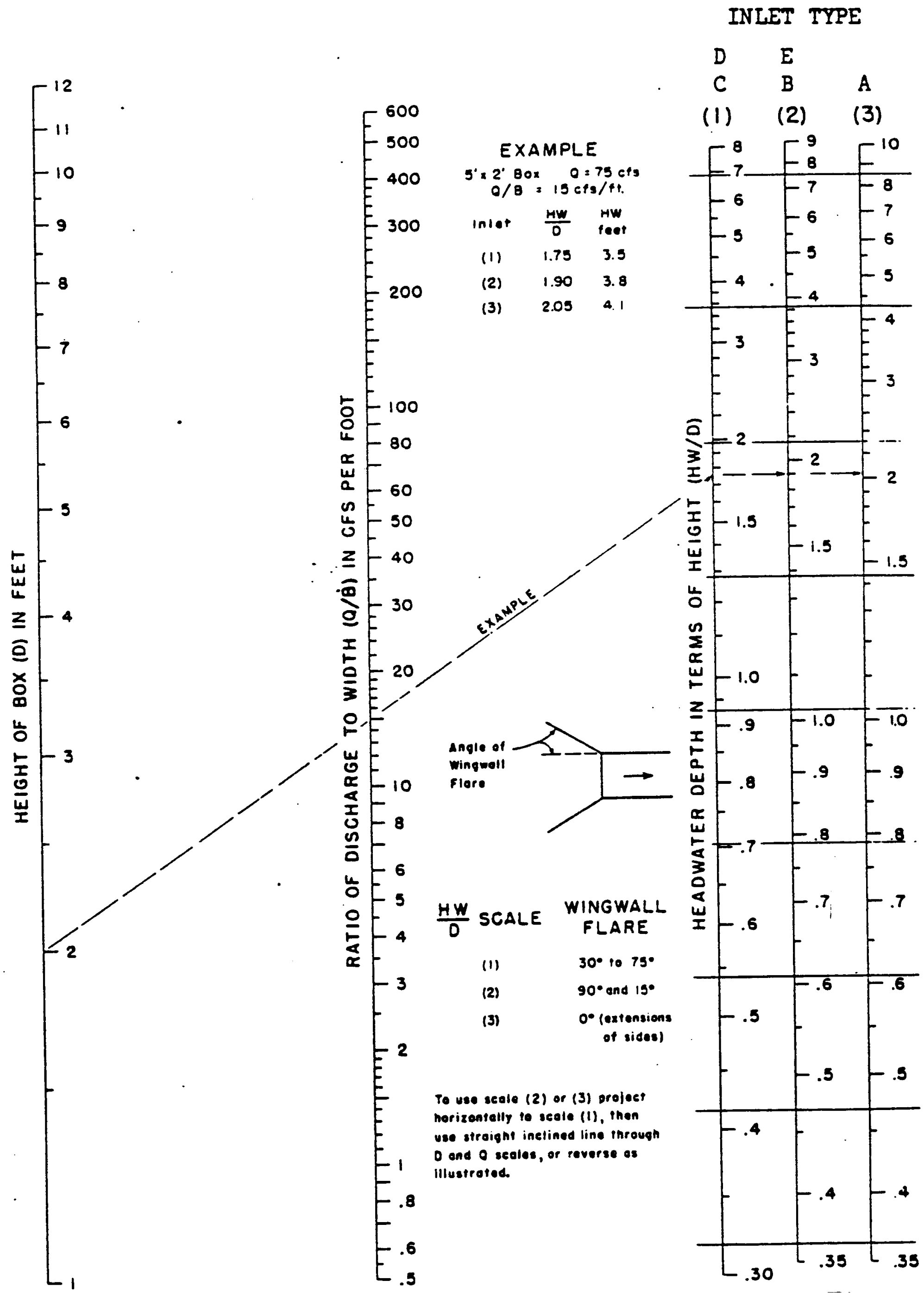


Figure 8.4-3

Headwater Depth for Box Culverts with Inlet Control

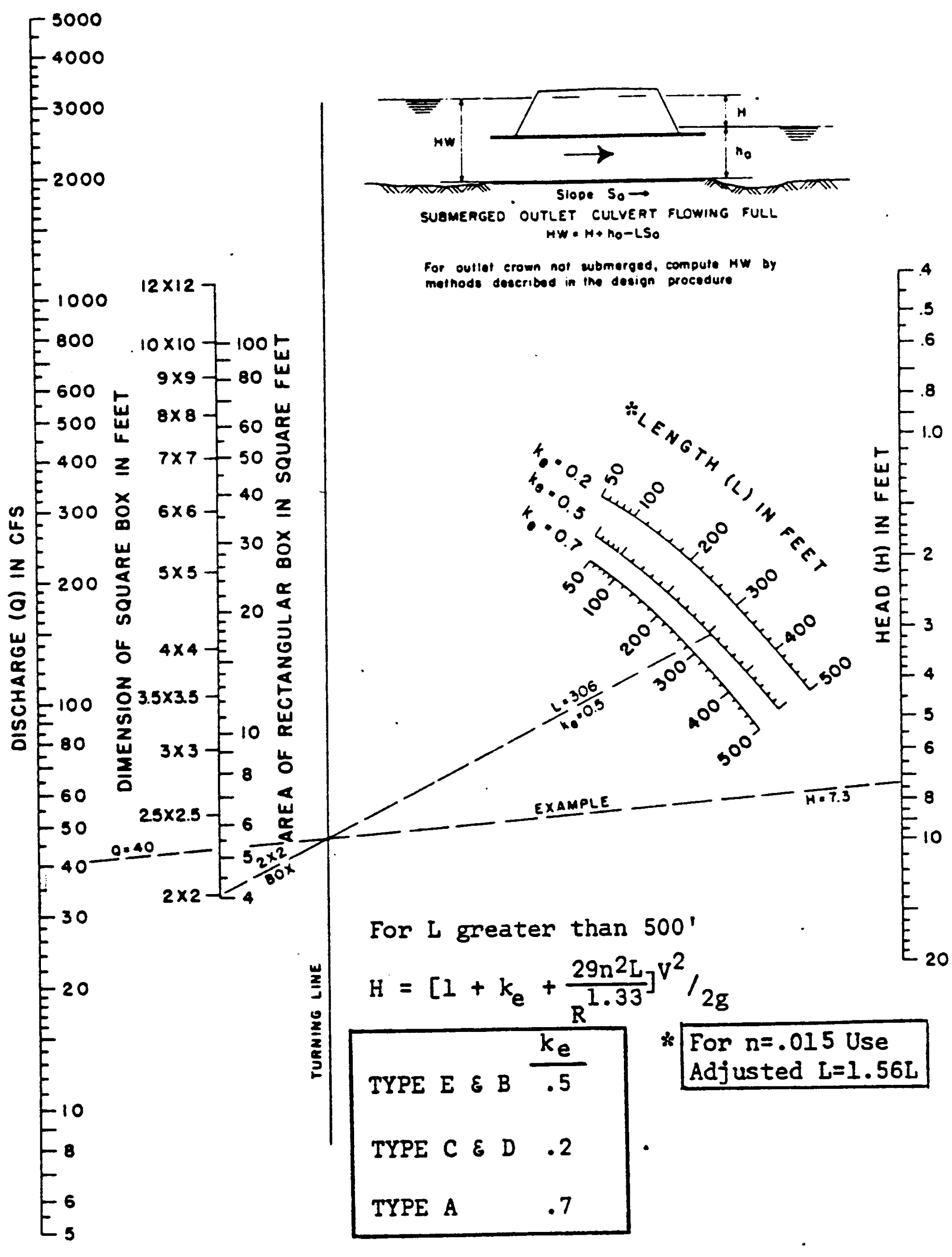


Figure 8.4-4

Head for Concrete Box Culverts Flowing Full, n = 0.012

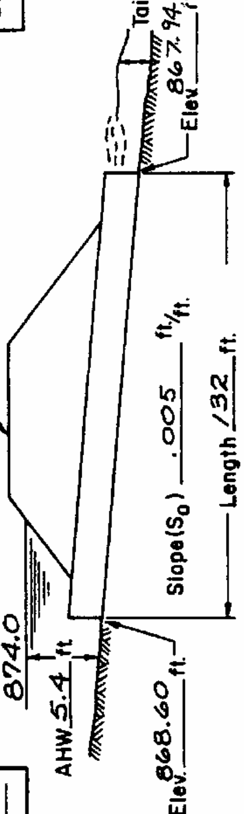
HYDROLOGIC AND CHANNEL INFORMATION

State of Wisconsin Department of Transportation
F-B-31-68

Project *Inlet Control Problem*
Culvert Sta. *432+00*

Designer *L.J.G.*
Date *9-23-69*

Hydrology:
(*50* freq.) Q= *720* cfs.
(____ freq.) Q= ____ cfs.



$\frac{L}{100 S_o}$ = ____ = ____

Location comments: *This structure is located a short distance downstream of the outlet control problem.*

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)

Summary & Recommendations:

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

** $HW = H + h_o - LS_o$

Figure 8.4-5
Culvert Computation Form

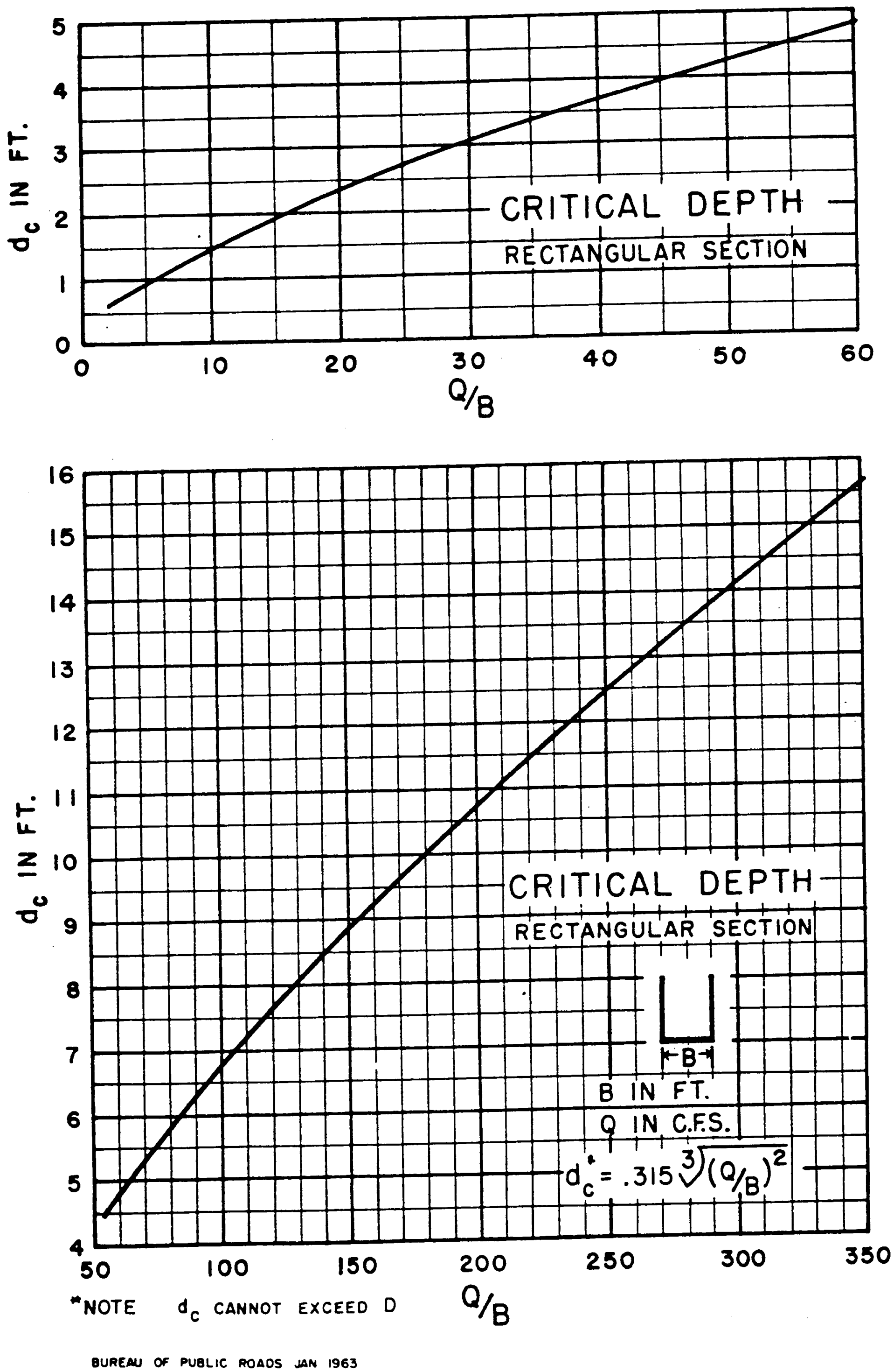


Figure 8.4-6
Critical Depth – Rectangular Section

8.4.2.6 Roadway Overflow

See 8.3.2.6.2

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.

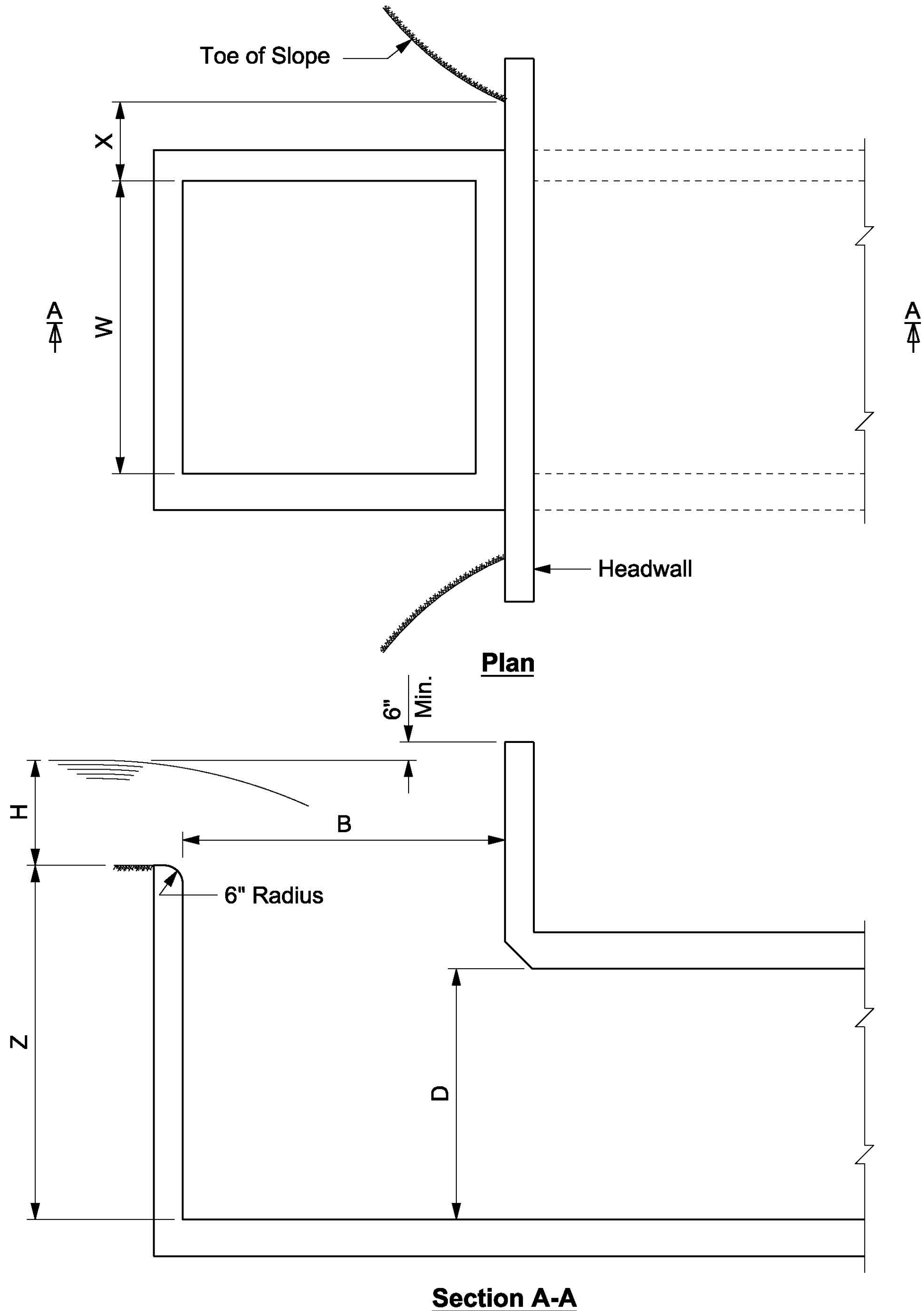


Figure 8.4-7
Box Drop Inlet

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

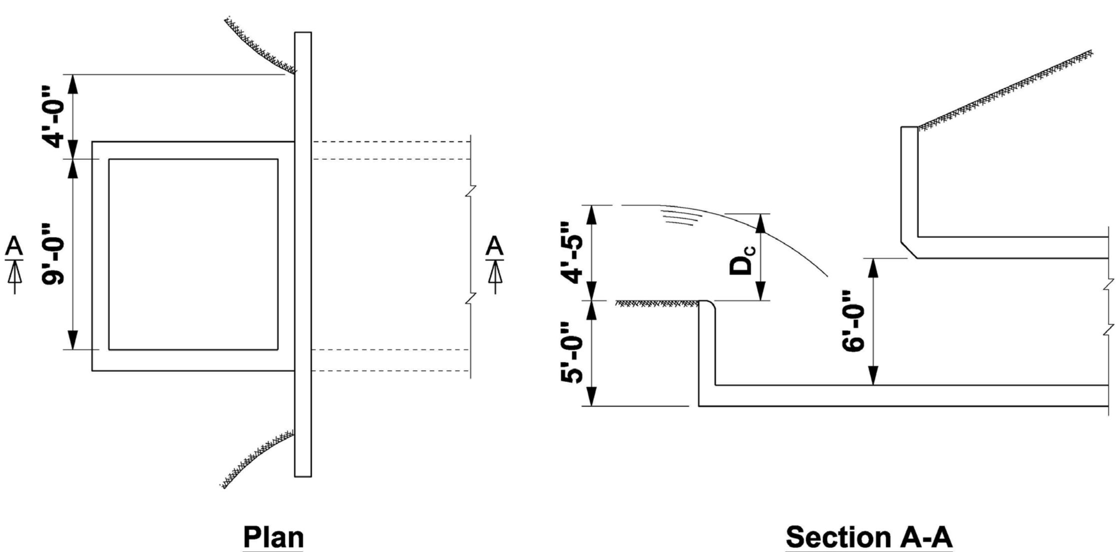


Figure 8.4-8
Drop Inlet Example

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

Corrections:

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

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

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

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

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.

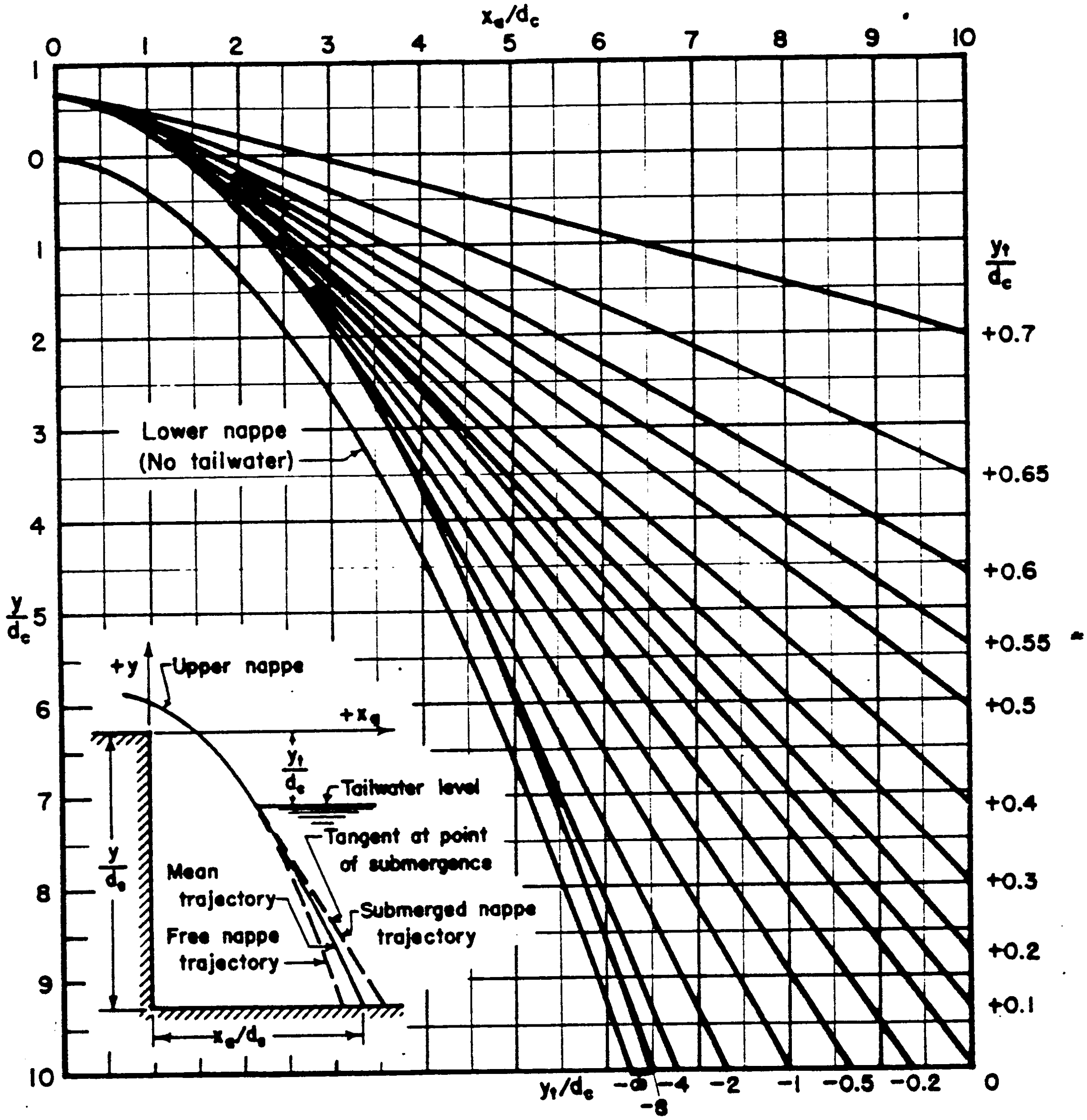


Figure 8.4-9

Design Chart for Determination of "X_a"

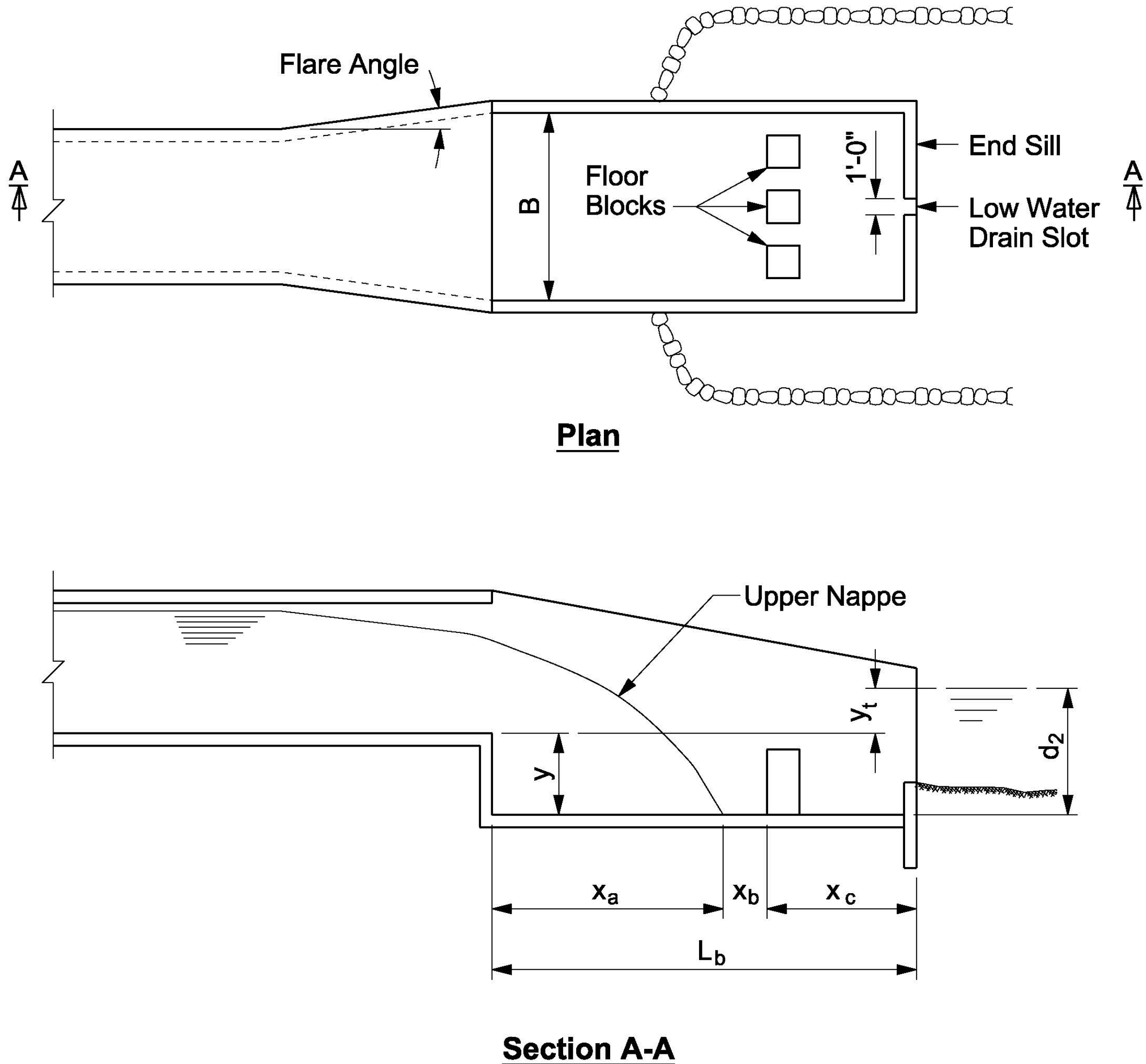


Figure 8.4-10
Straight Drop Outlet Stilling Basin

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

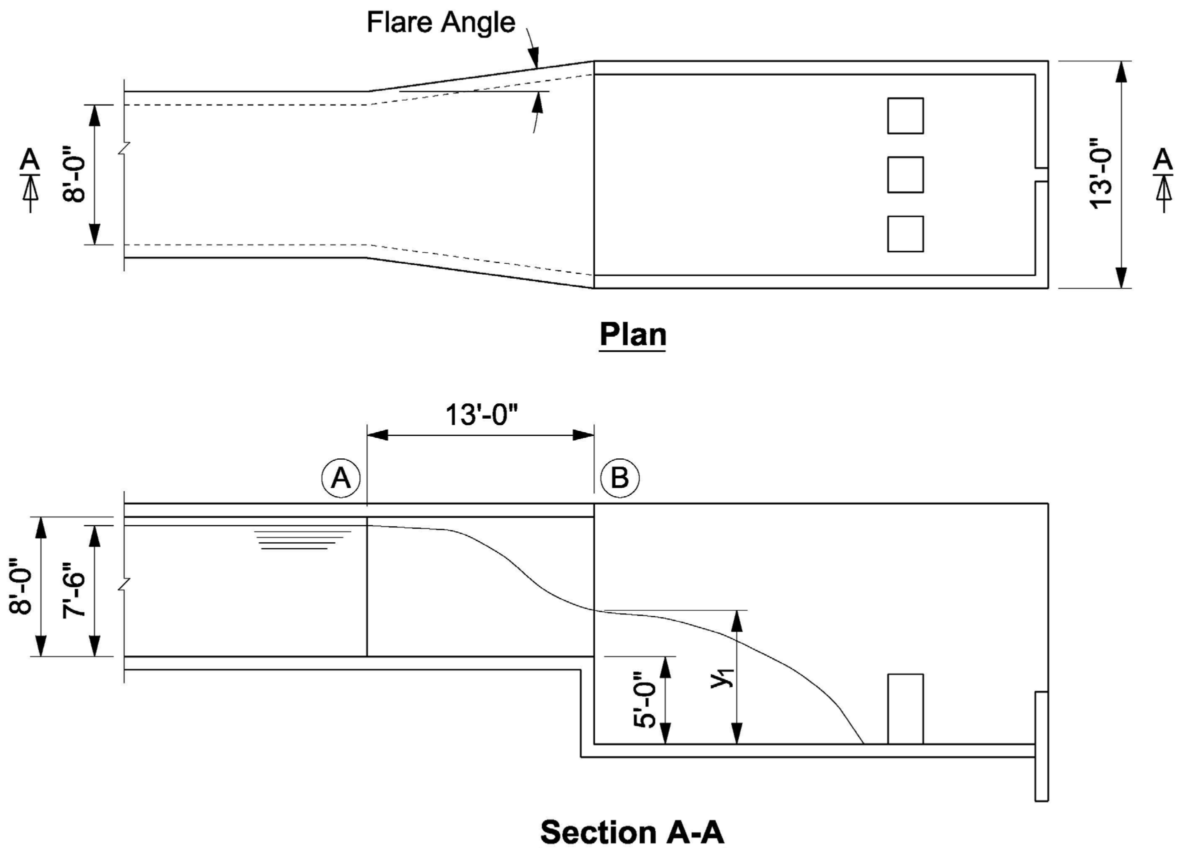


Figure 8.4-11
Drop Outlet Example

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:

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

Where:

y_2	=	tailwater depth required to cause the hydraulic jump
y_1	=	water depth at beginning of hydraulic jump
F_1	=	Froude number = $v_1 /(gy_1)^{1/2}$
g	=	acceleration of gravity
v_1	=	velocity at beginning of hydraulic jump

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

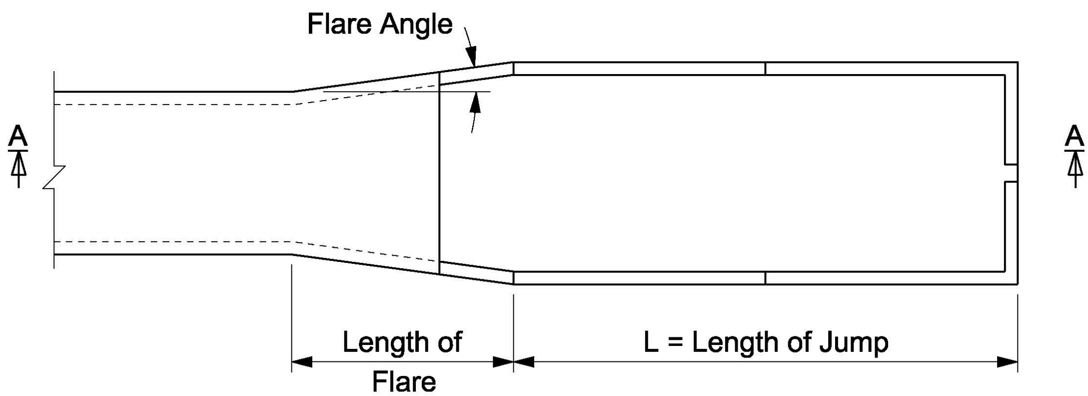
Length of jump is assumed to be 6 times the depth change (y_2-y_1).

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

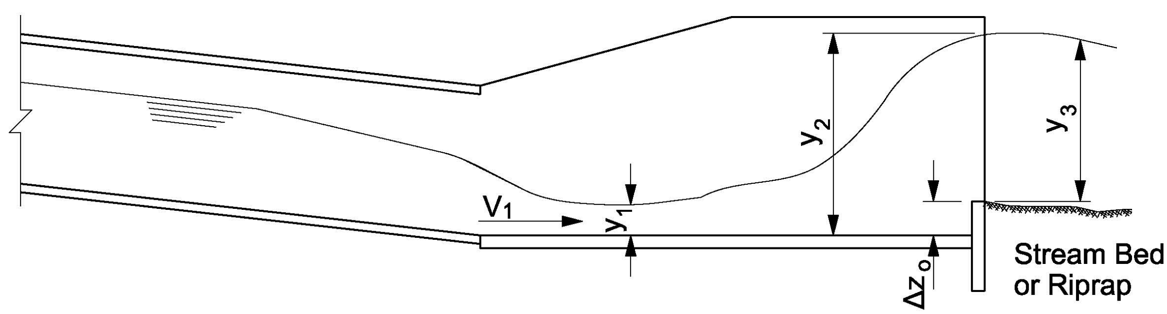
The problem of scour on the downstream side of the end sill can be overcome by providing riprap in the stream bottom. If riprap is used, it starts from the top of the sill at a maximum slope of 6:1 up from end sill to original streambed. If no riprap is used, the streambed begins at the top of the end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

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

Sample computations are shown in 8.4.2.7.3.1.



Plan



Section A-A

Figure 8.4-12
Hydraulic Jump Stilling Basin

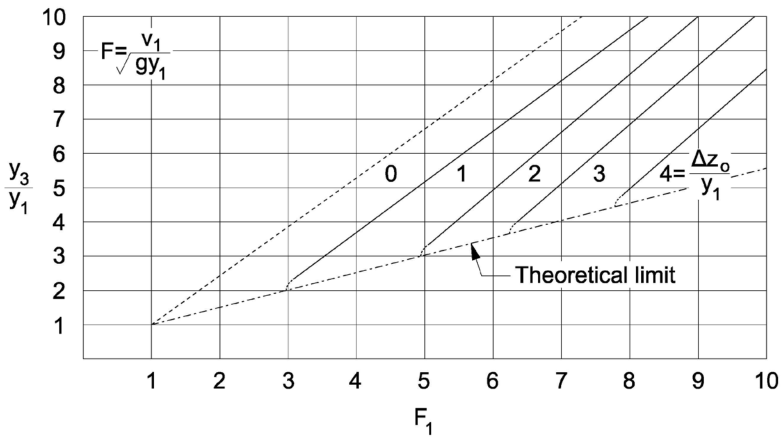


Figure 8.4-13
Characteristics of a Hydraulic Jump at an Abrupt Rise

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.

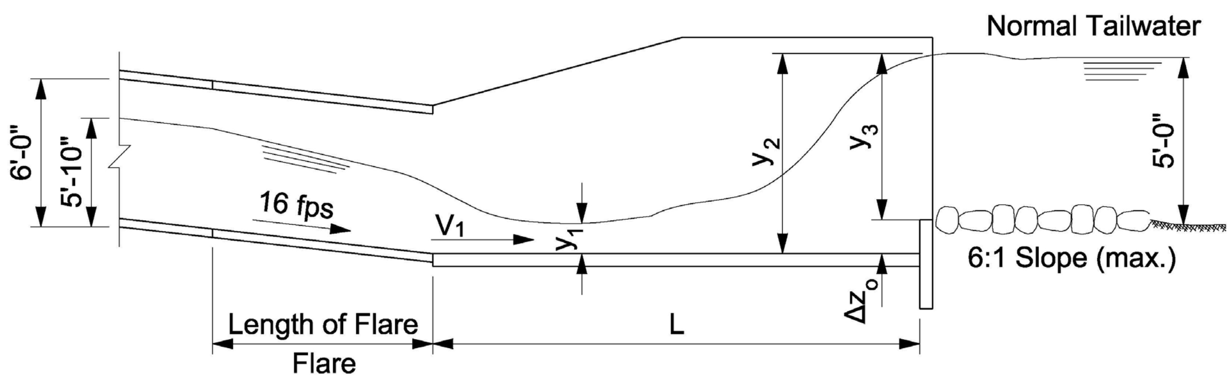


Figure 8.4-14
Hydraulic Jump Stilling Basin Example

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

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

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

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

$Y_1 = 2.20$

$V_1 = 22.1$

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

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

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

Assume $y_3 = 5'$

$y_3/y_1 = 5/2.2 = 2.27$

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

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

8.4.2.7.4 Riprap Stilling Basins

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

8.4.2.8 Select Culvert Design Alternatives

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

8.5 References

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

A hydraulic and site report shall be prepared for all stream crossing bridge and culvert projects that are completed by consultants. The report shall be submitted to the Bureau of Structures for review along with the "Stream Crossing Structure Survey Report" and preliminary structure plans (see WisDOT Bridge Manual, 6.2.1). The hydraulic and site report needs to include information necessary for the review of the hydraulic analysis and the type, size and location of proposed structure. All hydraulic sizing reports are required to be stamped and signed by a Professional Engineer (PE) registered within the State of Wisconsin. Supporting hydrologic and hydraulic computations and models must also have documentation of the PE in responsible charge of the analysis recorded. The following is a list of the items that need to be included in the hydraulic site report:

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

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

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

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

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

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

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

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

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

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

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

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

8.7 Appendix 8-B, FHWA Hydraulic Engineering Publications

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

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

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

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

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

Figure 8.7-1
FHWA Hydraulic Engineering Publications

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

Figure 8.7-2
FHWA Hydraulics Software List

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Table of Contents

12.12 MSE Walls at Abutments

MSE walls in front of pile supported abutments, as shown in Figure 12.1-1, may be an alternative to traditional sill abutments (Type A1) constructed at the top of the slope. This configuration shortens span lengths, which may be particularly beneficial in urban areas with limited right of way or other site constraints. However, use of MSE walls at abutments should be evaluated on a project-by-project basis. Historically, MSE walls have not been used for the singular purpose of reducing span length. See Chapter 14 - Standard Details for additional information.

MSE walls in front of Type A3 semi-retaining abutments may be considered only when traditional Type A3 abutments (without an MSE wall) or a modified Type A3 abutment (MSE wall in front of two rows of vertical piles) are not feasible. This configuration typically provides a single row of vertical piles for vertical support only and does not rely on these piles for lateral resistance to avoid complicating the MSE wall design. Lateral loads from the superstructure and earth pressures on the abutment backwall are designed to be fully restrained by an abutment anchorage system. An MSE wall system is typically used for this purpose; a dead man anchor system is an alternative. See Chapter 14 - Standard Details for additional information.

Abutments supported entirely by MSE walls, also referred to as a “true MSE bridge abutment" and similar to GRS abutments, are prohibited.

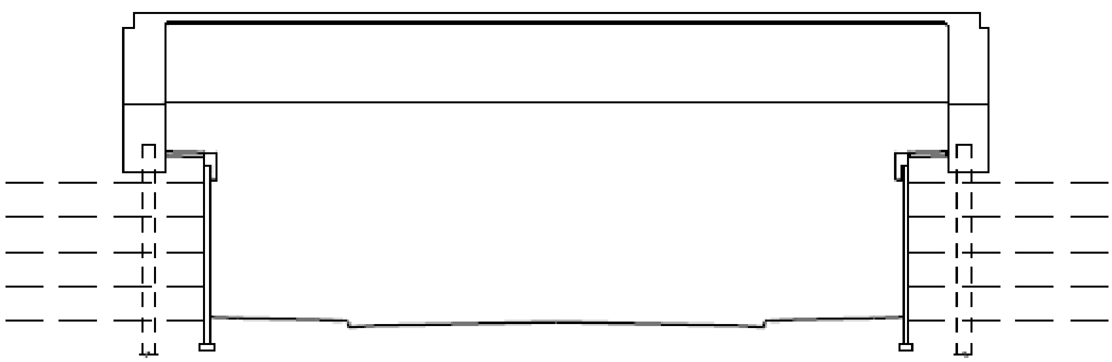

Figure 12.12-1
Sill Abutment with MSE Walls

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Table of Contents

13.2 Pier Types

The pier types most frequently used in Wisconsin are:

- Multi-column piers (Standards for Multi-Columned Pier and for Multi-Columned Pier – Type 2)
- Pile bents (Standard for Pile Bent)
- Pile encased piers (Standard for Pile Encased Pier)
- Solid single shaft / hammerheads (Standards for Hammerhead Pier and for Hammerhead Pier – Type 2)

Design loads shall be calculated and applied to the pier in accordance with 13.4 and 13.5. The following sections discuss requirements specific to each of the four common pier types.

13.2.1 Multi-Column Piers

Multi-column piers, as shown in Standard for Multi-Columned Pier, are the most commonly used pier type for grade separation structures. Refer to 13.6 for analysis guidelines.

A minimum of three columns shall be provided to ensure redundancy should a vehicular collision occur. If the pier cap cantilevers over the outside columns, a square end treatment is preferred over a rounded end treatment for constructability. WisDOT has traditionally used round columns. Column spacing for this pier type is limited to a maximum of 25'.

Multi-column piers are also used for stream crossings. They are especially suitable where a long pier is required to provide support for a wide bridge or for a bridge with a severe skew angle.

Continuous or isolated footings may be specified for multi-column piers. The engineer should determine estimated costs for both footing configurations and choose the more economical configuration. Where the clear distance between isolated footings would be less than 4'-6", a continuous footing shall be specified.

A variation of the multi-column pier in Standard for Multi-Columned Pier is produced by omitting the cap and placing a column under each girder. This detail has been used for steel girders with girder spacing greater than 12'. This configuration is treated as a series of single column piers. The engineer shall consider any additional forces that may be induced in the superstructure cross frames at the pier if the pier cap is eliminated. The pier cap may not be eliminated for piers in the floodplain, or for continuous slab structures which need the cap to facilitate replacement of the slab during future rehabilitation.

See Standard for Highway Over Railroad Design Requirements for further details on piers supporting bridges over railways.

13.2.2 Pile Bents

Pile bents are most commonly used for small to intermediate stream crossings and are shown on the Standard for Pile Bent.

Pile bents shall not be used to support structures over roadways or railroads due to their susceptibility to severe damage should a vehicular collision occur.

For pile bents, pile sections shall be limited to 12¾" or 14" diameter cast-in-place reinforced concrete piles with steel shells spaced at a minimum center-to-center spacing of 3'. A minimum of five piles per pier shall be used on pile bents. When a satisfactory design cannot be developed with one of these pile sections at the minimum spacing, another pier type should be selected. The outside piles shall be battered 2" per foot, and the inside piles shall be driven vertically. WisDOT does not rely on the shell of CIP piles for capacity; therefore these piles are less of a concern for long term reduced capacity due to corrosion than steel H-piles. For that reason the BOS Development Chief must give approval for the use of steel H-piles in open pile bents.

Because of the minimum pile spacing, the superstructure type used with pile bents is generally limited to cast-in-place concrete slabs, prestressed girders and steel girders with spans under approx. 70' and precast, prestressed box girders less than 21" in height.

To ensure that pile bents are capable of resisting the lateral forces resulting from floating ice and debris or expanding ice, the maximum distance from the top of the pier cap to the stable streambed elevation, including scour, is limited to:

- 15' for 12¾" diameter piles (or 12" H-piles if exception is granted).
- 20' for 14" diameter piles (or 14" H-piles if exception is granted).

Use of the pile values in Table 11.3-5 or Standard for Pile Details is valid for open pile bents due to the exposed portion of the pile being inspectable.

The minimum longitudinal reinforcing steel in cast-in-place piles with steel shells is 6-#7 bars in 12" piles and 8-#7 bars in 14" piles. The piles are designed as columns fixed from rotation in the plane of the pier at the top and at some point below streambed.

All bearings supporting a superstructure utilizing pile bents shall be fixed bearings or semi-expansion.

Pile bents shall meet the following criteria:

- If the water velocity, V_{100}, is greater than 7 ft/sec, the quantity of the 100-year flood shall be less than 12,000 ft^3/sec.
- If the streambed consists of unstable material, the velocity of the 100-year flood shall not exceed 9 ft/sec.

Table of Contents

18.1 Introduction

18.1.1 General

This chapter considers the following types of concrete structures:

- Flat Slab
- Haunched Slab

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

18.1.2 Limitations

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

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

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

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

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

WisDOT policy item:

Slab bridges, due to camber required to address future creep deflection, do not ride ideally for the first few years of their service life and present potential issues due to ponding. As such, if practical (e.g. not excessive financial implications), consideration of other structure types should be given for higher volume/higher speed facilities, such as the Interstate.

Table of Contents

36.1 Design Method

36.1.1 Design Requirements

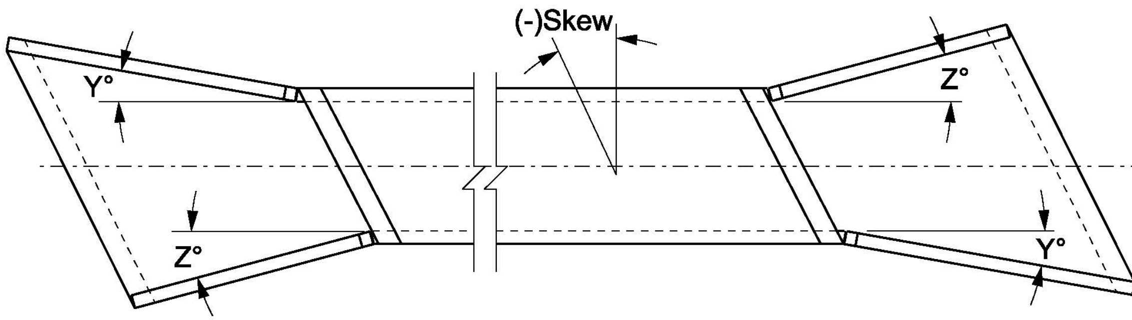
All new box culverts are to be designed using *AASHTO LRFD Bridge Design Specifications*, hereafter referred to as *AASHTO LRFD*.

36.1.2 Rating Requirements

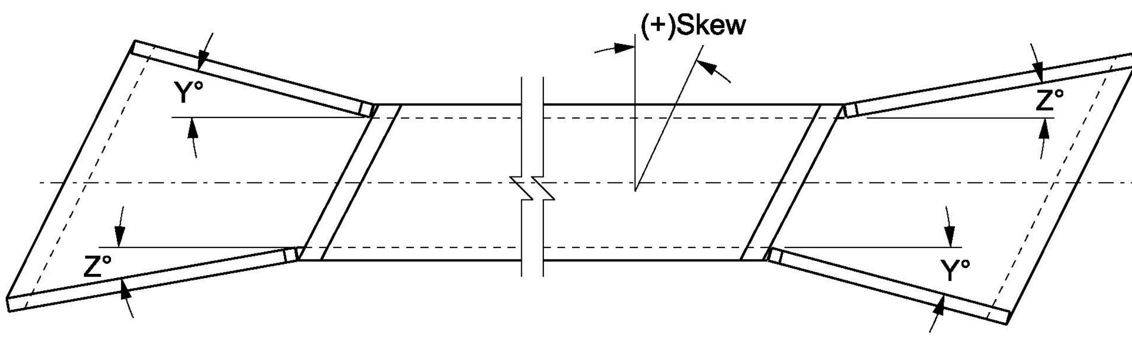
The current version of *AASHTO Manual for Bridge Evaluation (LRFR)* covers rating of concrete box culverts. Refer to 45.8 for additional guidance on load rating various types of culverts.

36.1.3 Standard Permit Design Check

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



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



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

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

36.7.3 Type E

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

36.7.4 Wingwall Design

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

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

Where:

W_H = Wing height measured from top of bottom slab to top of wing

For wing heights over 7 feet (13’-0” Max.) the wall length is divided into two or more segments to determine the area of steel required. Use the same bar size and spacing throughout each segment, as determined by using the maximum wall height in the segment.

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

Table of Contents

45.9 Load Rating Documentation and Submittals

The Bridge Rating and Management Unit is responsible for maintaining information for every structure in the Wisconsin inventory, including load ratings. This information is used throughout the life of the structure to help inform decisions on potential load postings, repairs, rehabilitation, and eventual structure replacement. That being the case, it is critical that WisDOT collect and store complete and accurate documentation regarding load ratings.

45.9.1 Load Rating Calculations

The rating engineer is required to submit load rating calculations. Calculations should be comprehensive and presented in a logical, organized manner. The submitted calculations should include a summary of all assumptions used (if any) to derive the load rating.

45.9.2 Load Rating Summary Forms

After the structure has been load rated, the WisDOT Bridge Load Rating Summary Form shall be completed and utilized as a cover sheet for the load rating calculations (see Figure 45.9-1). This form may be obtained from the Bureau of Structures or is available on the following website:

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

If required, the Refined Analysis Rating Form (see 45.9.5 and Figure 45.9-2) is available at the same location.

Instructions for completing the forms are as follows:

Load Rating Summary Form

1. Fill out applicable Bridge Data, Structure Type, and Span information using HSIS as reference.

2. Select the rating method and rating vehicle used to rate the bridge in the spaces provided.

3. Enter the inventory/operating ratings, live load factor, load governing member, rating force effect, controlling location, live load distribution factor (LLDF), LLDF level (single-lane or multi-lane), and rating limit state for the rating vehicle.

    a. If the load distribution was determined through refined methods (i.e., finite element analysis), it is not necessary to record the live load distribution factor. Instead, enter “REFINED” in the space provided and use the “Comments” section to describe the methods used to determine live load distribution.

4. The rating for the Wisconsin Special Permit vehicle (Wis-SPV) is always required and shall be given on the rating sheet for both a multi-lane distribution and a single-lane distribution. Make sure not to include the future wearing surface in these calculations.

All reported ratings are based on current conditions and do not reflect future wearing surfaces. Enter the Maximum Vehicle Weight (MVW) for the Wis-SPV analysis, live load factor, load governing member, rating force effect, controlling location, live load distribution factor (LLDF), LLDF level (single-lane or multi-lane), and rating limit state.

5. When necessary, AASHTO legal and WisDOT Specialized annual Permit vehicles shall be analyzed and load postings determined per the requirements of 45.10.

   a. Enter the lowest operating rating factor for each appropriate vehicle type, along with live load factor, load governing member, rating force effect, controlling location, live load distribution factor (LLDF), LLDF level (single-lane or multi-lane), and rating limit state.

   b. If a posting vehicle analysis was performed, check the box indicating if a load posting is required or not required. The weight limit in tons is automatically calculated when posting vehicle rating factors are below 1.0, using the rating factor multiplied by the gross vehicle weight. The overall controlling posting value is automatically calculated by rounding down to the nearest five tons among vehicles requiring a posting. This value may be manually overridden if appropriate, with an explanation included in the comments. If analysis shows that a load posting is required, specify the level of posting and contact the Bureau of Structures Rating Unit immediately.

6. When necessary, emergency vehicles shall be analyzed and weight limit restrictions determined per the requirements of 45.10.

   a. Enter the lowest operating rating factor for each emergency vehicle, along with live load factor, load governing member, rating force effect, controlling location, live load distribution factor (LLDF), LLDF level (single-lane or multi-lane), and rating limit state.

   b. Check the box indicating if an emergency vehicle weight limit is required or not required. The single axle, tandem axle, and gross vehicle weight limits are automatically calculated when emergency vehicle rating factors are below 1.0. If analysis shows that an emergency vehicle weight limit is required, specify the level of the limit and contact the Bureau of Structures Rating Unit immediately.

7. Enter all additional remarks as required to clarify the load capacity calculations.

8. It is necessary for the responsible engineer to sign and seal the form in the space provided. This is true even for rehabilitation projects with no change to the ratings.

45.9.3 Load Rating on Plans

The plans shall contain the following rating information:

- Inventory Load Rating – The plans shall have either the HS value of the inventory rating if using LFR or the rating factor for the HL-93 if using LRFR. For LFR ratings, the rating should be rounded down to the nearest whole number. This rating shall be based on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface. See 6.2.2.3.4 for more information on reporting ratings on plans.

- Operating Load Rating – The plans shall have either the HS value of the operating rating if using LFR or the rating factor for the HL-93 if using LRFR. For LFR ratings, the rating should be rounded down to the nearest whole number. This rating shall be based on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface. See 6.2.2.3.4 for more information.

- Wisconsin Special Permit Vehicle – The plans shall also contain the results of the Wis-SPV analysis utilizing single-lane distribution and assuming that the vehicle is mixing with normal traffic and that the full dynamic load allowance is utilized. This rating shall be based on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface. The recorded rating for this is the total allowable vehicle weight rounded down to the nearest 10 kips. If the value exceeds 250 kips, limit the plan value to 250 kips. See 6.2.2.3.4 for more information.

45.9.4 Computer Software File Submittals

If analysis software is used to determine the load rating, the software input file shall be provided as a part of the submittal. The name of the analysis software and version should be noted on the Load Rating Summary form in the location provided.

45.9.5 Submittals for Bridges Rated Using Refined Analysis

Additional pages of documentation are required when performing a refined analysis. In addition to the Load Rating Summary Form, also submit the Refined Analysis Rating Form as shown in Figure 45.9-2.

45.9.6 Other Documentation Topics

Structures with Two Different Rating Methods

There may be situations where a given superstructure contains elements that were constructed at different times. In these situations, two different rating methods are used during the design/rating process. For example, a girder replacement or widening. In this case, the new girder(s) would be designed/rated using LRFR, while the existing girders would be rated using LFR. A Load Rating Summary Form shall be submitted for both new & existing structure analysis methods; controlling LRFR rating of the new superstructure components, and controlling LFR rating of the existing superstructure. Both sets of controlling rating values (new & existing) shall be noted on the plan set, as noted in 6.2.2.3.4.

Wisconsin Department of Transportation

Bridge Load Rating Summary

v. 07-2026

Bridge Data

Bridge Number:		Traffic Count:		Clear Roadway Width (ft):	
Owner:		Truck Traffic %:		Overburden Depth (in):	
Municipality:		Inspection Date:			
Feature On:		Component Condition Ratings:			
Feature Under:		Deck	Superstructure	Substructure	Culvert
Design Load:					

Structure Type & Span Information

Span #	Material	Configuration	Length (ft)	Span Continuity	Deck Interaction
1					

Load Rating Summary

Rating Method:	
Rating Vehicle:	

Computer Software & Version Used:	

Design Load Rating Analysis

	Rating	LL Factor	Load Governing Member	Rating Force Effect	Control Location	LLDF	LLDF Level	Rating Limit State
Inventory								
Operating								
Wis-SPV (Single)								
Wis-SPV (Multi)								

Load Posting Analysis (when required per Wisconsin Bridge Manual, Chapter 45)

Posting Vehicle	Vehicle GVW (k)	Rating Factor	Weight Limit	LL Factor	Load Governing Member	Rating Force Effect	Control Location	LLDF	LLDF Level	Rating Limit State
Type 3	50		N/A							
Type 3S2	72		N/A							
Type 3-3	80		N/A							
SU4	54		N/A							
SU5	62		N/A							
SU6	69.5		N/A							
SU7	77.5		N/A							
PUP	98		N/A							
SEMI	98		N/A							
EV2	57.5		N/A							
EV3	86		N/A							

Posting for Legal/Specialized Permit Vehicles:

X Not Required

Required ____ T

Weight Limits for Emergency Vehicles:

X Not Required

Required ____ T Single Axle// ____ T Tandem // ____ T Gross

Comments:

Load Rating Engineer	
Name:	
Date:	
PE #:	

Figure 45.9-1

Bridge Load Rating Summary Form

In Addition to this form, submit electronic analysis files (eg. .MDX, .bdb)

ANALYSIS FILE SUMMARY (FILL OUT FOR EACH ANALYSIS FILE SUBMITTED)

Analysis Type:	☐ Grid/Grillage ☐ Plate & Ecc. Beam ☐ 3D FEM ☐ Other *(describe below)*
Analysis Program:	☐ MDX ☐ AASHTOWare ☐ CSI Bridge ☐ LARSA ☐ Other
Program Version:	
File Name:	
File Description:	Describe the purpose of the file. Example: This file is used for the Wis-SPV rating using single lane distribution.
Analysis Assumptions:	Highlight key assumptions in modeling. (This section may be omitted if submitting MDX or AASHTOWare analysis files. This section may also be omitted if submitting separate document containing analysis assumptions and results). Example of things to include: a description of the finite element model, simplifications made to model, exceptions to original design plans, loads applied, how loads are applied (e.g. equally distributed to all girders), support conditions, composite/non-composite sections.
Summary of Results:	Summarize results. (This section may be omitted if submitting MDX or AASHTOWare analysis files. This section may also be omitted if submitting separate document containing analysis assumptions and results). Provide table of results for service load reactions, moment, shear, and/or stress output for members at 10th points (minimum) for the appropriate load cases. Provide a table of capacities at each 10th point, such that load ratings can be directly computed with appropriate load and/or resistance and impact factors. Provide example or typical calculations.

Figure 45.9-2

Refined Analysis Rating Form

45.10 Load Postings

45.10.1 Overview

Legal-weight for vehicles travelling over bridges is determined by state-specific statutes, which are based in part on the Federal Bridge Formula. The Federal Bridge Formula is discussed in 45.2.5. When a bridge does not have the capacity to carry legal-weight traffic, more stringent load limits are placed on the bridge – a load posting. Currently in Wisconsin, load postings are based on gross vehicle weight; there is no additional consideration for number of axles or axle spacing. Load posting signage is discussed further in 0.

A separate analysis is conducted for emergency vehicles (EVs). As a result of the 2015 Fixing America's Surface Transportation Act (FAST Act), FHWA requires bridges to be load rated for emergency vehicles where they are exempt from regular weight limits, and restricted if necessary. When a bridge does not have the capacity to carry the FAST Act EVs, emergency vehicle-specific load postings are required for bridges on the Interstate and within reasonable access to the Interstate. Because Wisconsin statutes also exempt emergency vehicles from state laws governing weight provisions, bridges located beyond reasonable access with insufficient capacity will be placed on the Emergency Vehicles Restricted Bridge List (under development). Weight limit restrictions for emergency vehicles are based on a combination of the single axle, tandem axle, and gross vehicle weight limits, discussed further in 45.10.3. Additional information on FAST Act EV load rating requirements may be found in FHWA's memorandum, "Action: Load Rating for the FAST Act's Emergency Vehicles" (November 2016) and the technical guidance, "Questions and Answers: Load Rating for the FAST Act's Emergency Vehicles, Revision R01" (March 2018).

In order to remain open to traffic, a bridge should be capable of carrying a minimum gross live load weight of three tons at the Operating level. Bridges not capable of carrying a minimum gross live load weight of three tons at the Operating level must be closed. As stated in the **MBE [6A.8.1]** and **[6B.7.1]**, when deciding whether to close or post a bridge, the Owner should consider the character of traffic, the volume of traffic, the likelihood of overweight vehicles, and the enforceability of weight posting.

The owner of a bridge has the responsibility and authority to load post a bridge as required. The State Bridge Maintenance Engineer has the authority to post a bridge and must issue the approval to post any State bridge.

WisDOT policy items:
Consult the Bureau of Structures Rating Unit as soon as possible with any analysis that results in a load posting or emergency vehicle weight limit for any structure on the State or Local system.

45.10.2 Load Posting Live Loads

The live loads to be used in the rating formula for posting considerations are any of the three typical AASHTO Commercial Vehicles (Type 3, Type 3S2, Type 3-3) shown in Figure 45.10-1, any of the four AASHTO Specialized Hauling Vehicles (SHVs - SU4, SU5, SU6, SU7) shown in Figure 45.10-2, the WisDOT Specialized Annual Permit Vehicles shown in Figure 45.10-3, and the Wisconsin Standard Permit Vehicle shown in Figure 45.12-1.

The AASHTO Commercial Vehicles and Specialized Hauling Vehicles are modeled on actual in-service vehicle configurations. These vehicles comply with the provisions of the Federal Bridge Formula and can thus operate freely without permit; they are legal weight/configuration.

The WisDOT Specialized Annual Permit Vehicles are Wisconsin-specific vehicles. They represent vehicle configurations made legal in Wisconsin through the legislative process and current Wisconsin state statutes.

The Wisconsin Standard Permit Vehicle (Wis-SPV) is a configuration used internally by WisDOT to assist in the regulation of multi-trip (annual) permits. Multi-trip permits and the Wis-SPV are discussed in more detail in 45.11.2 and 45.12.

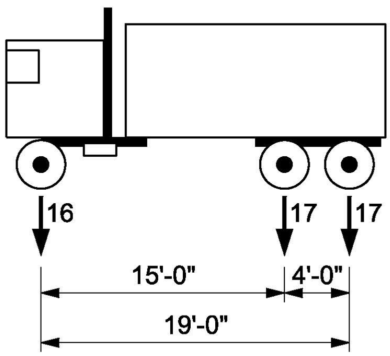
As stated in **MBE [6A.4.4.2.1a]**, for spans up to 200', only the vehicle shall be considered present in the lane for positive moments. It is unnecessary to place more than one vehicle in a lane for spans up to 200' because the load factors provided have been modeled for this possibility. For spans 200' in length or greater, the AASHTO Type 3-3 truck multiplied by 0.75 shall be analyzed combined with a lane load as shown in Figure 45.10-4. The lane load shall be taken as 0.2 klf in each lane and shall only be applied to those portions of the span(s) where the loading effects add to the vehicle load effects.

Also, for negative moments and reactions at interior supports, a lane load of 0.2 klf combined with two AASHTO Type 3-3 trucks multiplied by 0.75 shall be used. The trucks should be heading in the same direction and should be separated by 30 feet as shown in Figure 45.10-4. There are no span length limitations for this negative moment requirement.

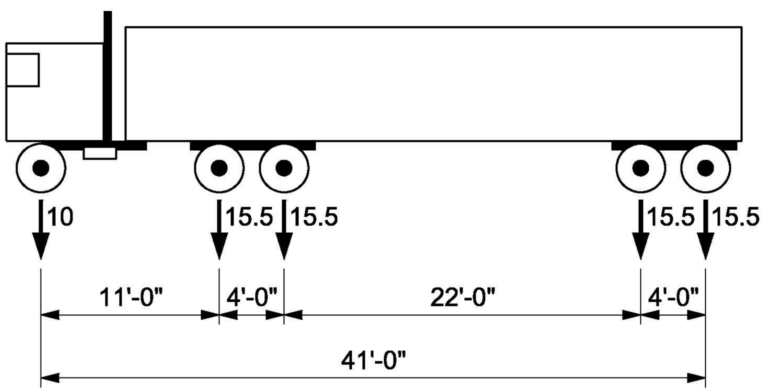
When the lane-type load model (see Figure 45.10-4) governs the load rating, the equivalent truck weight for use in calculating a safe load capacity for the bridge shall be taken as 80 kips as is specified in **MBE [6A.4.4.4]**.

For emergency vehicle weight limits, FHWA has determined that, for the purpose of load rating, two emergency vehicle configurations (EV2 and EV3) produce effects in typical bridges that envelop the effects resulting from the family of typical emergency vehicles covered by the FAST Act. The EV2 and EV3 are shown in Figure 45.10-5.

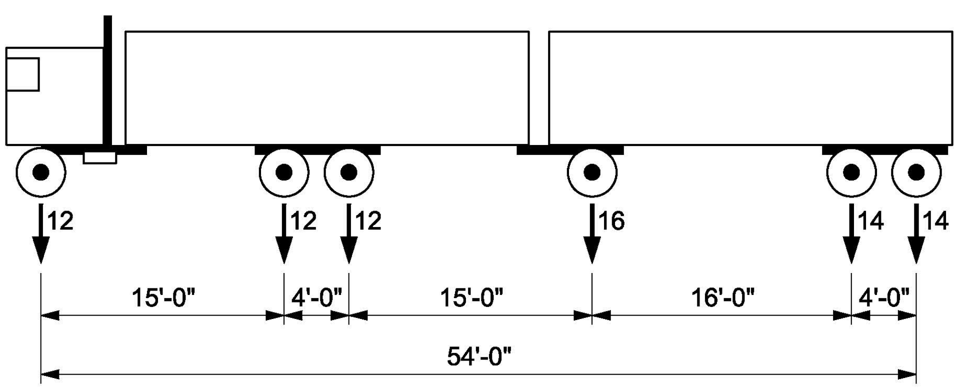
Indicated concentrations
are axle loads in kips.



Type 3 Unit Weight = 50 Kips (25 tons)



Type 3S2 Unit Weight = 72 Kips (36 tons)



Type 3-3 Unit Weight = 80 Kips (40 tons)

Figure 45.10-1
AASHTO Commercial Vehicles

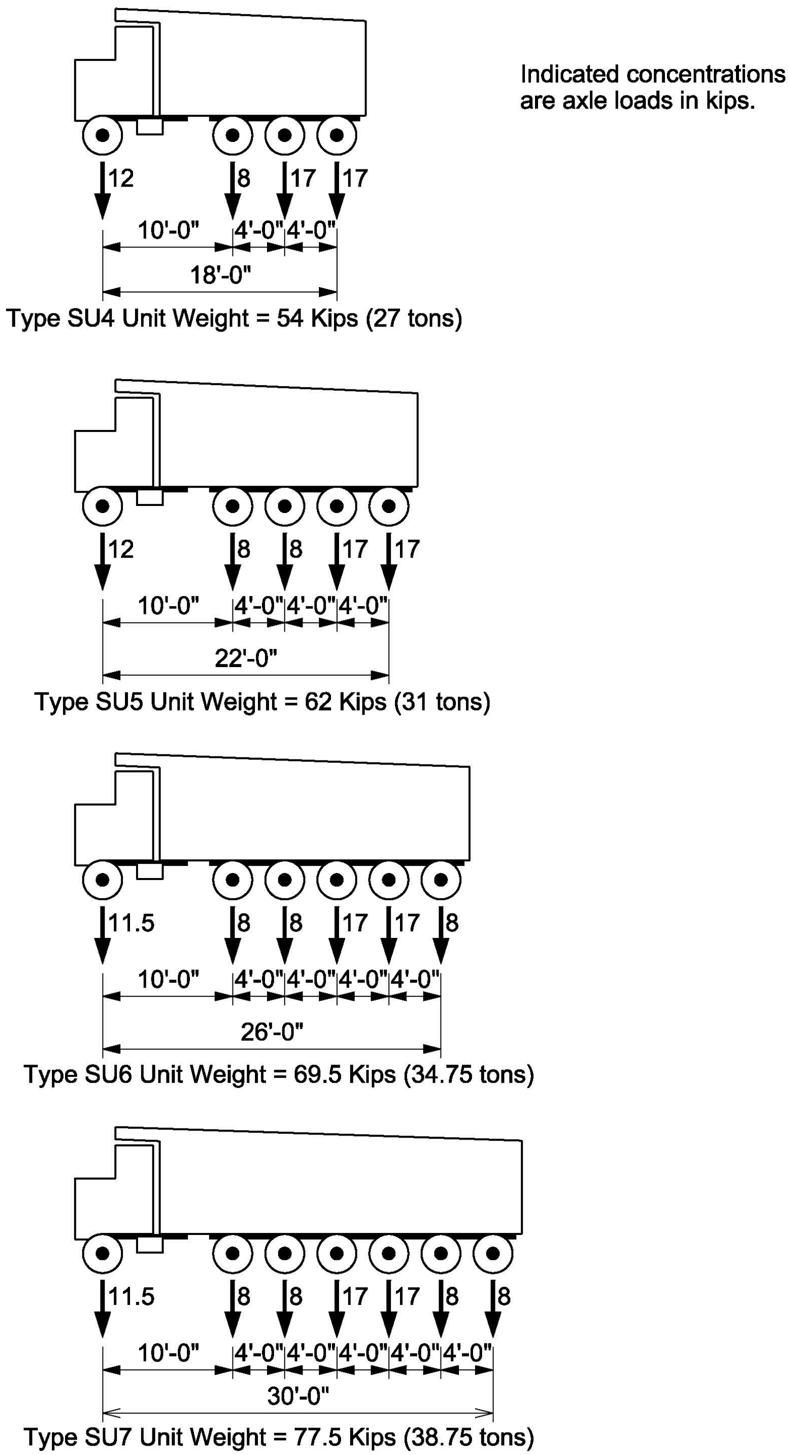


Figure 45.10-2
AASHTO Specialized Hauling Vehicles (SHVs)

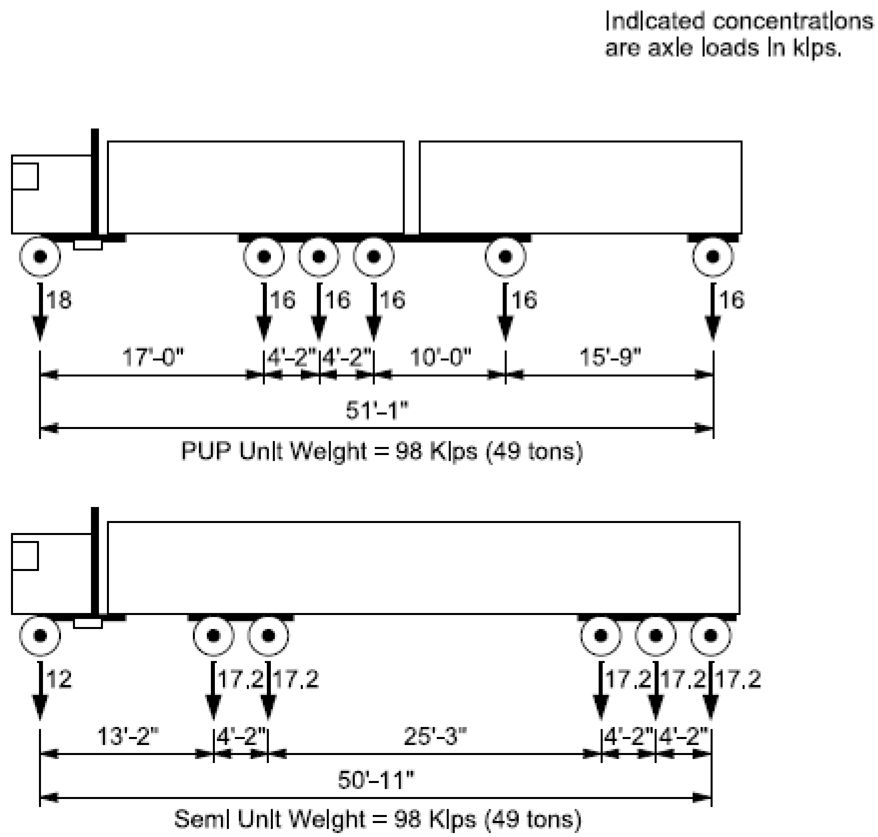
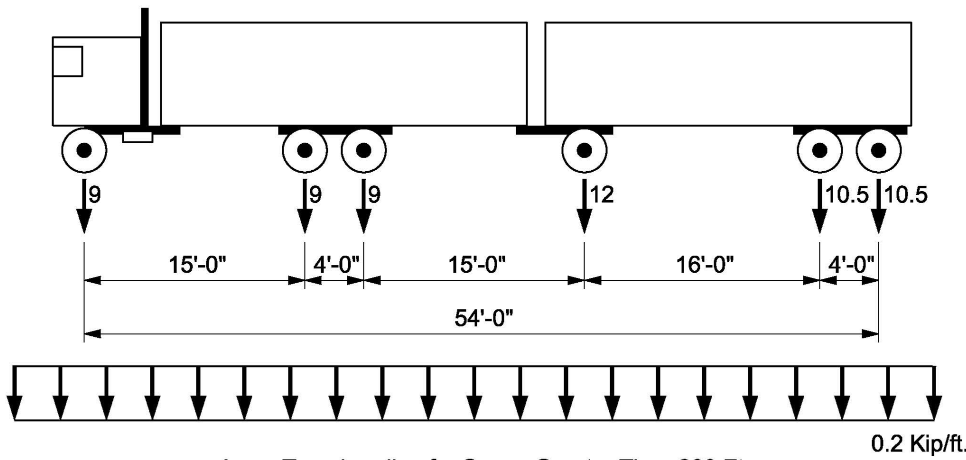


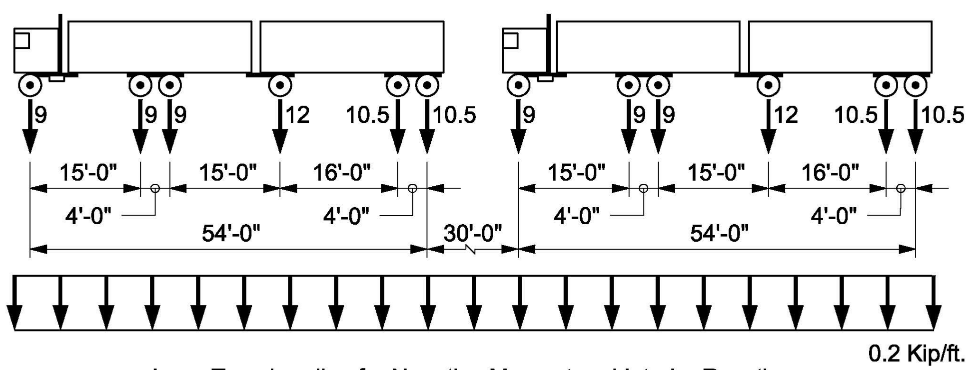
Figure 45.10-3

WisDOT Specialized Annual Permit Vehicles

Indicated concentrations are axle loads in kips (75% of type 3-3).



Lane-Type Loading for Spans Greater Than 200 Ft.



Lane-Type Loading for Negative Moment and Interior Reaction.

Figure 45.10-4
Lane Type Legal Load Models

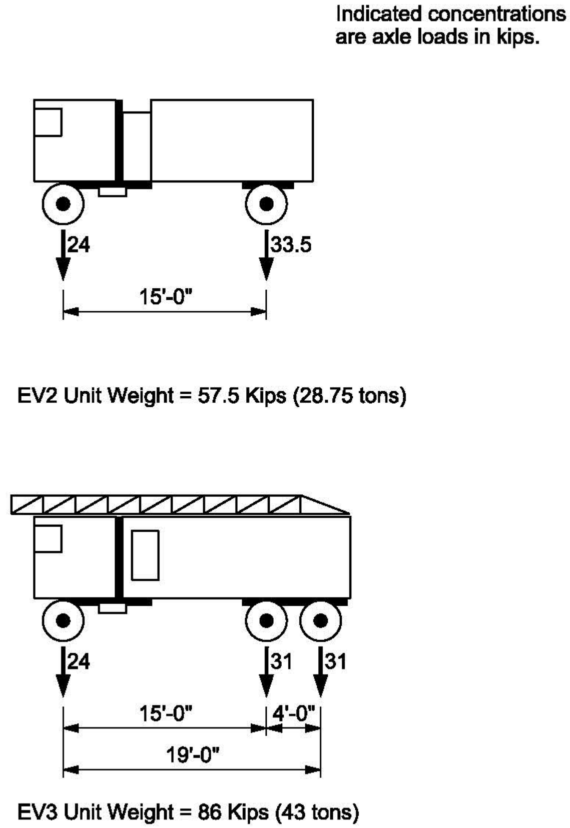


Figure 45.10-5
Emergency Vehicle Load Models

45.10.3 Load Posting Analysis

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

- Less than 1.0 for HL-93 loading using LRFR methodology.
- Less than 1.0 for HS-20 loading using LFR/ASR methodology; or
- Less than or equal to 1.3 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 or ASR, multiply the rating factor by the gross vehicle weight in tons. To calculate the posting load for a bridge analyzed with LRFR, refer to MBE 6A.8.3. Contact the Bureau of Structures Load Rating Unit before recommending a posting based on LRFR methodology.

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.

45.10.3.2 Distribution Factors for Load Posting Analysis

WisDOT policy items:

The AASHTO Commercial Vehicles, Specialized Hauling Vehicles, and Emergency Vehicles shall be analyzed using a multi-lane distribution factor for bridge clear roadway widths 18'-0" or larger. Single lane distribution factors are used for bridge clear roadway widths less than 18'-0".

The WisDOT Specialized Annual Permit Vehicles shown in Figure 45.10-3 shall be analyzed using a single-lane distribution factor, regardless of bridge width.

The Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed for load postings using a multi-lane distribution factor for bridge clear roadway widths 18'-0" or larger. Single lane distribution factors are used for bridge clear roadway widths less than 18'-0".

For Specialized Hauling Vehicles, single-lane distribution factor may be considered on two-lane roadways with travel in opposite directions to avoid a new or reduced load posting, if the bridge has demonstrated an ability to carry routine legal loads in its vicinity. Contact the Bureau of Structures Rating Unit for approval to use single-lane distribution factors on bridges with multiple lanes.

For Emergency Vehicles, refined analysis may be used to determine alternative distribution factors based on only one EV in one lane loaded simultaneously with other unrestricted legal vehicles in other lanes. This exception will reduce the computed load effects and yield higher load ratings. Refer to FHWA's "Questions and Answers: Load Rating for the FAST Act's Emergency Vehicles, Revision R01" (March 2018).

45.10.4 Load Posting Signage

Current WisDOT policy is to post State bridges for a single gross weight, in tons. Bridges that cannot carry the maximum weight for the vehicles described in 45.10.2 at the operating level are posted with the standard sign shown in Figure 45.10-6. This sign shows the bridge capacity for the governing load posting vehicle, in tons. The sign should conform to the requirements of the *Wisconsin Manual for Uniform Traffic Control Devices (WMUTCD)*.

In the past, local bridges were occasionally posted with the signs shown in Figure 45.10-7 using the H20, Type 3 and Type 3S2 vehicles. The H20 represented the two-axle vehicle, the Type 3 represented the three-axle vehicle and the Type 3S2 represented the combination vehicle. This practice is not encouraged by WisDOT and is generally not allowed for State-owned structures, except with permission from the State Bridge Maintenance Engineer.

Emergency vehicle posting signs, however, are based on a combination of the single axle, tandem axle, and gross vehicle weight limits, as shown in Figure 45.10-8. Emergency vehicle posting signs are only required for bridges on the Interstate and within reasonable access (one road mile) to or from an Interstate interchange.

WEIGHT
LIMIT
10
TONS

BRIDGE
CLOSED

Figure 45.10-6
Standard Signs Used for Posting Bridges

WEIGHT LIMIT
2 AXLE VEHICLES
15 TONS
3 AXLE VEHICLES
20 TONS
COMBINATION
VEHICLES
30 TONS

WEIGHT LIMIT
2 AXLE VEHICLES
14 TONS
3 AXLE VEHICLES
18 TONS
COMBINATION VEHICLES
28 TONS

WEIGHT LIMIT
2 AXLE VEHICLES
14 TONS
3 AXLE VEHICLES
18 TONS
COMBINATION
VEHICLES
28 TONS

Figure 45.10-7
Historic Load Posting Signs

EMERGENCY
VEHICLE
WEIGHT LIMIT

SINGLE AXLE	15 TONS
TANDEM	25 TONS
GROSS	35 TONS

Figure 45.10-8
Emergency Vehicle Load Posting Signs

45.11 Over-Weight Truck Permitting

45.11.1 Overview

Size and weight provisions for vehicles using the Wisconsin network of roads and bridges are specified in the Wisconsin Statutes, Chapter 348: Vehicles – Size, Weight and Load. Weight limits for legal-weight traffic and over-weight permit requirements are defined in detail in this chapter. The webpage for Chapter 348 is shown below.

https://docs.legis.wisconsin.gov/statutes/statutes/348

Over-weight permit requests are processed by the WisDOT Oversize Overweight (OSOW) Permit Unit in the Bureau of Highway Maintenance. The permit unit collaborates with the WisDOT Bureau of Structures Rating Unit to ensure that permit vehicles are safely routed on the Wisconsin inventory of bridges.

While the Wisconsin Statutes contain several industry-specific size and weight annual permits, in general, there are two permit types in Wisconsin: multi-trip (annual) permits and single-trip permits.

45.11.2 Multi-Trip (Annual) Permits

Multi-trip permits are granted for non-divisible loads such as machines, self-propelled vehicles, mobile homes, etc. They typically allow unlimited trips and are available for a range of three months to one year. The permit vehicle may mix with typical traffic and move at normal speeds. Multi-trip permits are required to adhere to road and bridge load postings and are subject to additional restrictions based on restricted bridge lists supplied by the WisDOT Bureau of Structures Rating Unit and published by the WisDOT OSOW Permit Unit. The restricted bridge lists are developed based on the analysis of the Wisconsin Standard Permit Vehicle (Wis-SPV). For more information on the Wis-SPV and required analysis, see 45.12. The carrier is responsible for their own routing, and are required to avoid these restrictions and load postings.

Vehicles applying for a multi-trip permit are limited to 170,000 pounds gross vehicle weight, plus additional restrictions on maximum length, width, height, and axle weights. Please refer to the WisDOT Oversize Overweight (OSOW) Permits website or the Wisconsin Statues (link above) for more information.

https://www.dot.wisconsinwisconsindot.gov/business/carriers/osowgeneral.htm

45.11.3 Single Trip Permits

Non-divisible loads which exceed the annual permit restrictions may be moved by the issuance of a single trip permit. When a single trip permit is issued, the applicant is required to indicate on the permit the origin and destination of the trip and the specific route that is to be used. A separate permit is required for access to local roads. Each single trip permit vehicle is individually analyzed by WisDOT for all state-owned structures that it encounters on the designated permit route.

Live load distribution for single trip permit vehicles is based on single lane distribution. This is used because these permit loads are infrequent and are likely the only heavy loads on the structure during the crossing. The analysis is performed at the operating level.

At the discretion of the engineer evaluating the single trip permit, the dynamic load allowance (or impact for LFR) may be neglected provided that the maximum vehicle speed can be reduced to 5 MPH prior to crossing the bridge and for the duration of the crossing.

In some cases, the truck may be escorted across the bridge with no other vehicles allowed on the bridge during the crossing. If this is the case, then the live load factor (LRFR analysis) can be reduced from 1.20 to 1.10 as shown in Table 45.3-3. It is recommended that the truck be centered on the bridge if it is being escorted with no other vehicles allowed on the bridge during the crossing.

Vehicles with non-standard axle gauges may also receive special consideration. This may be achieved by performing a more-rigorous analysis of a given bridge that takes into account the specific load configuration of the permit vehicle in question instead of using standard distribution factors that are based on standard-gauge axles. Alternatively, modifications may be made to the standard distribution factor in order to more accurately reflect how the load of the permit vehicle is transferred to the bridge superstructure. How non-standard gauge axles are evaluated is at the discretion of the engineer evaluating the permit.

45.12 Wisconsin Standard Permit Vehicle (Wis-SPV)

45.12.1 Background

The Wis-SPV configuration is shown in Figure 45.12-1. It is an 8-axle, 190,000lbs vehicle. It was developed through a Wisconsin research project that investigated the history of multi-trip permit configurations operating in Wisconsin. The Wis-SPV was designed to completely envelope the force effects of all multi-trip permit vehicles operating in Wisconsin and is used internally to help regulate multi-trip permits.

45.12.2 Analysis

- New Bridge Construction

For any new bridge design, the Wis-SPV shall be analyzed. The Wis-SPV shall be evaluated at the operating level. When performing this design check for the Wis-SPV, the vehicle shall be evaluated for single-lane distribution assuming that the vehicle is mixing with normal traffic and that the full dynamic load allowance is utilized. For this design rating, a future wearing surface shall be considered. Load distribution for this check is based on the interior strip or interior girder and the distribution factors given in Section 17.2.7, 17.2.8, or 18.4.5.1 where applicable. See also the WisDOT policy item in 45.3.7.8.1.

For LRFR, the Wis-SPV design check shall be a permit load rating and shall be evaluated for the limit states noted in Table 45.3-1 and Table 45.3-3.

The design engineer shall check to ensure the design has a RF > 1.0 (gross vehicle load of 190 kips) for the Wis-SPV. If the design is unable to meet this minimum capacity, the engineer is required to adjust the design until the bridge can safely handle a minimum gross vehicle load of 190 kips.

Results of the Wis-SPV analysis shall be reported per 45.9.

- Bridge Rehabilitation Projects

For rehabilitation design, analysis of the Wis-SPV shall be performed as described above for new bridge construction. All efforts should be made to obtain a RF > 1.0 (gross vehicle load of 190 kips) within the confines of the scope of the project. However, it is recognized that it may not be possible to increase the Wis-SPV rating without a significant change in scope of the project. In these cases, consult the Bureau of Structures Rating Unit for further direction.

Results of the Wis-SPV analysis shall be reported per 45.9.

- Existing (In-Service) Bridges

When performing a rating for an existing (in-service) bridge, analysis of the Wis-SPV shall be performed as described above for new bridge construction. In this case – where the bridge in question is being load rated but not altered in any way – the results of the Wis-SPV analysis need simply be reported as calculated per 45.9. If the results of this analysis produce a rating factor less than 1.0 (gross vehicle load less than 190 kips), notify the Bureau of Structures Rating Unit.

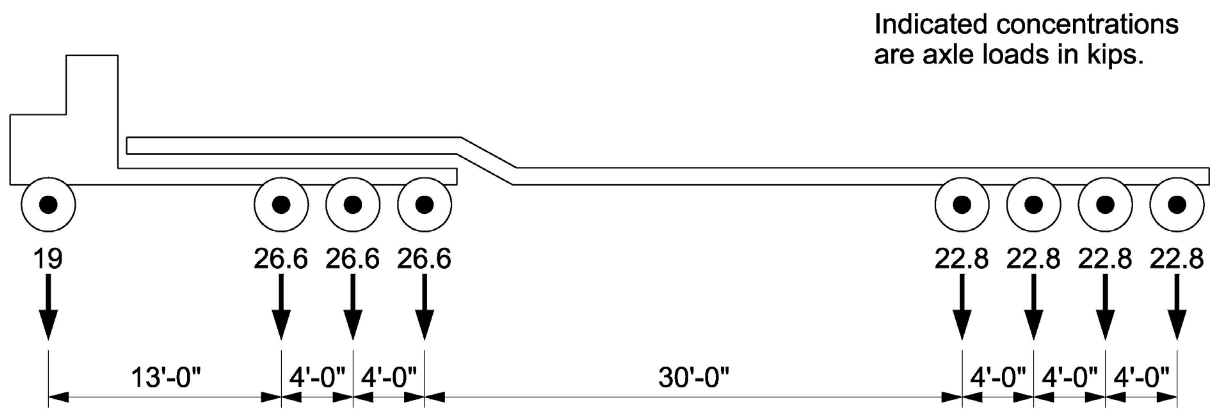


Figure 45.12-1
Wisconsin Standard Permit Vehicle (Wis-SPV)

45.13 References

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45.14 Rating Examples