## Table of Contents

8.1 Introduction .... 4
8.1.1 Objectives of Highway Drainage .... 4
8.1.2 Basic Policy .... 4
8.1.3 Design Frequency .... 4
8.1.3.1 FHWA Directive .... 5
8.1.3.2 DNR-DOT Cooperative Agreement .... 5
8.1.3.3 DOT Facilities Development Manual .... 5
8.1.4 Hydraulic Site Report .... 5
8.1.5 Hydraulic Design Criteria for Temporary Structures .... 5
8.1.6 Erosion Control Parameters .... 6
8.1.7 Bridge Rehabilitation and Hydraulic Studies .... 6
8.2 Hydrologic Analysis .... 7
8.2.1 Regional Regression Equations .... 7
8.2.2 Project Site at Streamgage .... 7
8.2.3 Project Site at Ungaged Location on a Gaged Stream .... 7
8.2.4 Flood Insurance and Floodplain Studies .... 8
8.2.5 Natural Resources Conservation Service .... 8
8.3 Hydraulic Design of Bridges .... 9
8.3.1 Hydraulic Design Factors .... 9
8.3.1.1 Velocity .... 9
8.3.1.2 Roadway Overflow .... 9
8.3.1.3 Bridge Skew .... 9
8.3.1.4 Backwater and High-water Elevation .... 9
8.3.1.5 Freeboard .... 10
8.3.1.6 Scour .... 11
8.3.2 Design Procedures .... 11
8.3.2.1 Determine Design Discharge .... 11
8.3.2.2 Determine Hydraulic Stream Slope .... 11
8.3.2.3 Select Floodplain Cross-Section(s) .... 11
8.3.2.4 Assign “Manning n” Values to Section(s) .... 12
8.3.2.5 Select Hydraulic Model Methodology .... 12
8.3.2.6 Develop Hydraulic Model .... 13

8.3.2.6.1 Bridge Hydraulics ........ 14
8.3.2.6.2 Roadway Overflow ........ 14
8.3.2.7 Conduct Scour Evaluation ........ 17
8.3.2.7.1 Live Bed and Clear Water Scour ........ 18
8.3.2.7.2 Long-term Aggradation and Degradation ........ 18
8.3.2.7.3 Contraction Scour ........ 18
8.3.2.7.4 Local Scour ........ 20
8.3.2.7.5 Design Considerations for Scour ........ 23
8.3.2.8 Select Bridge Design Alternatives ........ 23
8.4 Hydraulic Design of Box Culverts ........ 24
8.4.1 Hydraulic Design Factors ........ 24
8.4.1.1 Economics ........ 24
8.4.1.2 Minimum Size ........ 24
8.4.1.3 Allowable Velocities and Outlet Scour ........ 24
8.4.1.4 Roadway Overflow ........ 24
8.4.1.5 Culvert Skew ........ 25
8.4.1.6 Backwater and Highwater Elevations ........ 25
8.4.1.7 Debris Protection ........ 25
8.4.1.8 Anti-Seepage Collar ........ 25
8.4.1.9 Weep Holes ........ 26
8.4.2 Design Procedure ........ 27
8.4.2.1 Determine Design Discharge ........ 27
8.4.2.2 Determine Hydraulic Stream Slope ........ 27
8.4.2.3 Determine Tailwater Elevation ........ 27
8.4.2.4 Design Methodology ........ 27
8.4.2.5 Develop Hydraulic Model ........ 27
8.4.2.6 Roadway Overflow ........ 34
8.4.2.7 Outlet Scour and Energy Dissipators ........ 34
8.4.2.7.1 Drop Inlet ........ 34
8.4.2.7.1.1 Drop Inlet Example Calculations ........ 38
8.4.2.7.2 Drop Outlets ........ 39
8.4.2.7.2.1 Drop Outlet Example Calculations ........ 43
8.4.2.7.3 Hydraulic Jump Stilling Basins ........ 46

8.4.2.7.3.1 Hydraulic Jump Stilling Basin Example Calculations .......................... 48
8.4.2.7.4 Riprap Stilling Basins.................................................................... 49
8.4.2.8 Select Culvert Design Alternatives ................................................... 49
8.5 References............................................................................................. 50
8.6 Appendix 8-A, Check List for Hydraulic/Site Report.......................................... 52
8.7 Appendix 8-B, FHWA Hydraulic Engineering Publications............................... 54

# 8.1 Introduction

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

## 8.1.1 Objectives of Highway Drainage

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

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

## 8.1.2 Basic Policy

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

## 8.1.3 Design Frequency

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

### 8.1.3.1 FHWA Directive

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

### 8.1.3.2 DNR-DOT Cooperative Agreement

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

### 8.1.3.3 DOT Facilities Development Manual

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

## 8.1.4 Hydraulic Site Report

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

## 8.1.5 Hydraulic Design Criteria for Temporary Structures

The basic design criteria for temporary structures will to be the ability to pass a 5-year storm (Q5) with only 0.5 feet of backwater over existing conditions. This 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. 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 taken from contour maps are acceptable when the information is supplemented with field survey sections and data. Additional sections may be required to develop a proper hydraulic model for the site.

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

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

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

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

### 8.3.2.5 Select Hydraulic Model Methodology

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

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

1. HEC-RAS

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

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

2. HY8

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

### 8.3.2.6 Develop Hydraulic Model

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

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

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

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

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

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

### 8.3.2.6.1 Bridge Hydraulics

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

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

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

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

### 8.3.2.6.2 Roadway Overflow

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

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

The weir discharge equation is:

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

Where:

| | | |
|---|---|---|
| Q | = | discharge |
| $C_f$ | = | coefficient of discharge for free flow conditions |
| B | = | length of flow section along the road normal to the direction of flow |
| H | = | total head = h + $h_v$ |
| $k_t$ | = | submergence factor |

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

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

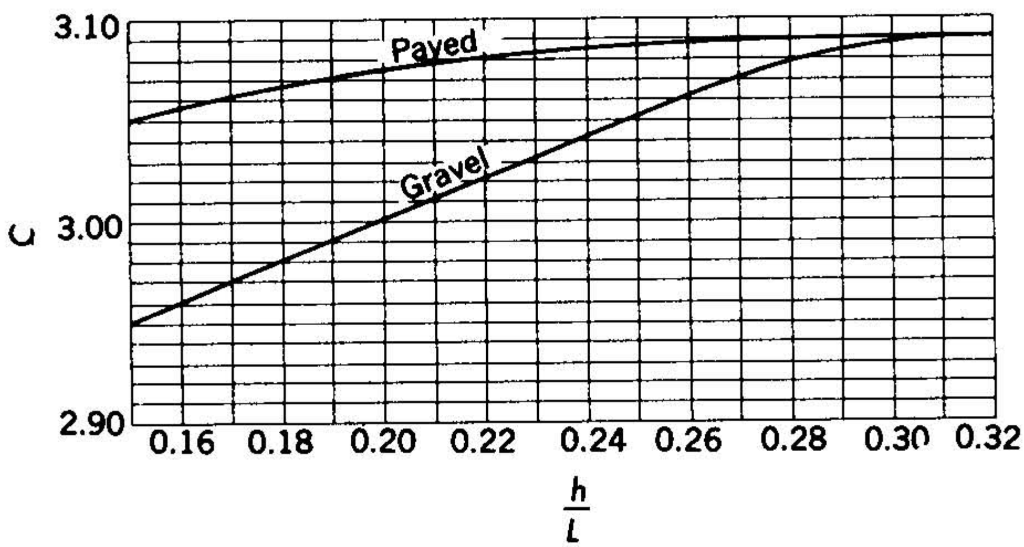



**Figure 8.3-1**
Discharge Coefficients, $C_f$, for Highway Embankments for H/L Ratios > 0.15

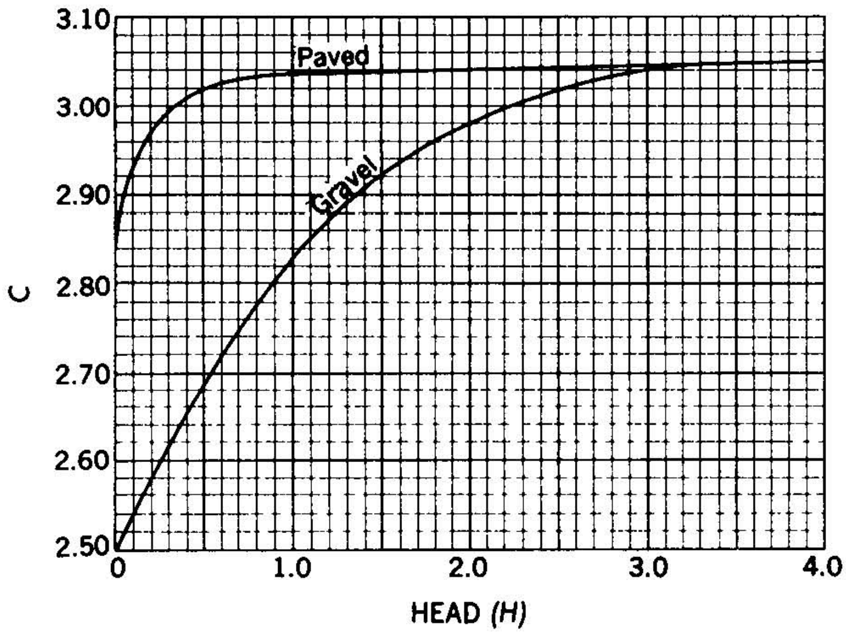



**Figure 8.3-2**
Discharge Coefficients, $C_f$, for Highway Embankments for H/L Ratios < 0.15

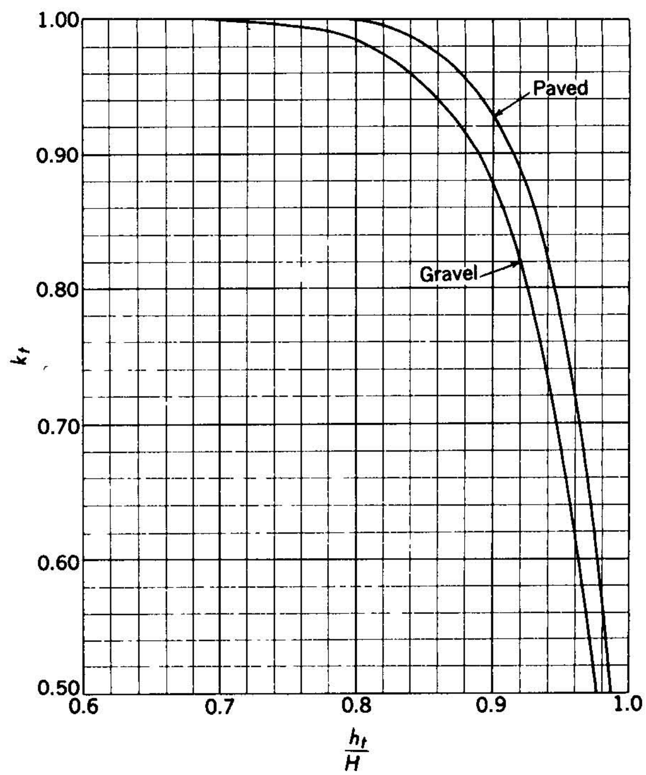



**Figure 8.3-3**
Definition of Adjustment Factor, $k_t$, for Submerged Highway Embankments

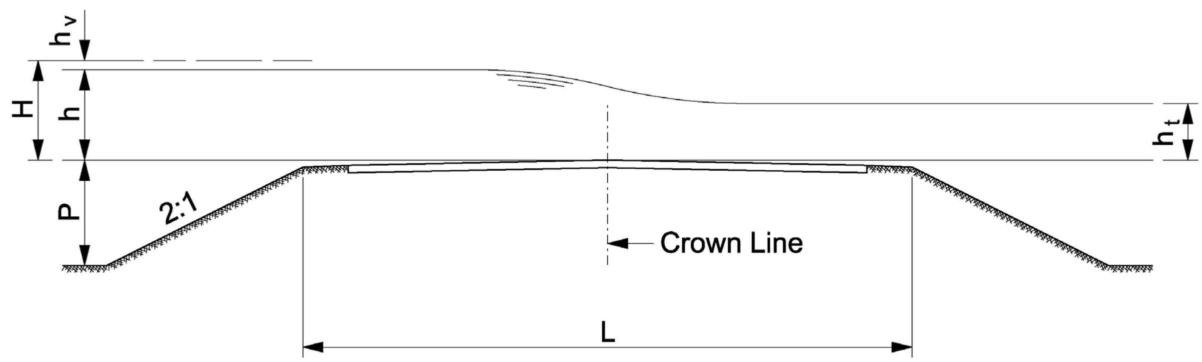



**Figure 8.3-4**
Definition Sketch of Flow Over Highway Embankment

### 8.3.2.7 Conduct Scour Evaluation

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

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

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. See 8.5 reference (14) & (15) for discussion and methods of analysis.

If a pressure flow condition exists at the bridge opening, then vertical contraction scour must be evaluated. 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 2) | |
|---|---|
| Shape of Pier Nose | $K_1$ |
| (a) Square Nose | 1.1 |
| (b) Round Nose | 1.0 |
| (c) Circular Cylinder | 1.0 |
| (d) Group of Cylinders | 1.0 |
| (e) Sharp Nose | 0.9 |

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

| Correction Factor, $K_2$, for Angle of Attack, Θ, of the Flow (HEC-18 Table 3) | | | |
|---|---|---|---|
| Angle | L/a = 4 | L/a = 8 | L/a = 12 |
| 0 | 1.0 | 1.0 | 1.0 |
| 15 | 1.5 | 2.0 | 2.5 |
| 30 | 2.0 | 2.75 | 3.5 |
| 45 | 2.3 | 3.3 | 4.3 |
| 90 | 2.5 | 3.9 | 5.0 |
| Angle = skew angle of flow<br>L = length of pier, ft<br>a = pier width, ft | | | |

**Table 8.3-2**
Correction Factor, $K_2$, for Angle of Attack, $\theta$, of the Flow

| Increase in Equilibrium Pier Scour Depths, $K_3$, for Bed Conditions (HEC-18 Table 4) | | |
|---|---|---|
| Bed Condition | Dune Height, ft | $K_3$ |
| Clear – water Scour | N/A | 1.1 |
| Plane Bed and Antidune Flow | N/A | 1.1 |
| Small Dunes | $3 > H \geq 0.6$ | 1.1 |
| Medium Dunes | $9 > H \geq 3$ | 1.2 to 1.1 |
| Large Dunes | $H \geq 9$ | 1.3 |

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

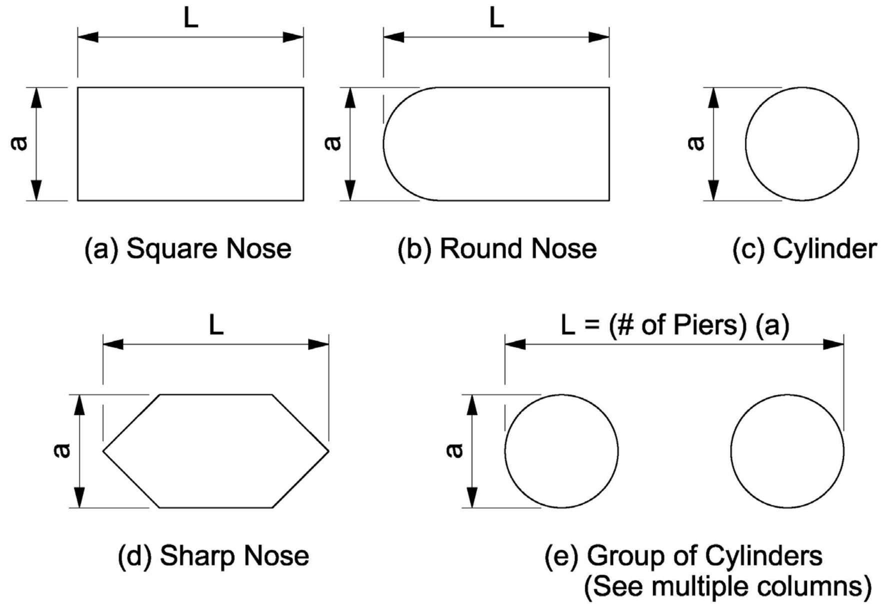



**Figure 8.3-5**
Common Pier Shapes

2. Abutment Scour Equations

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

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

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

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

#### 8.3.2.7.5 Design Considerations for Scour

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

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

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

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

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

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.

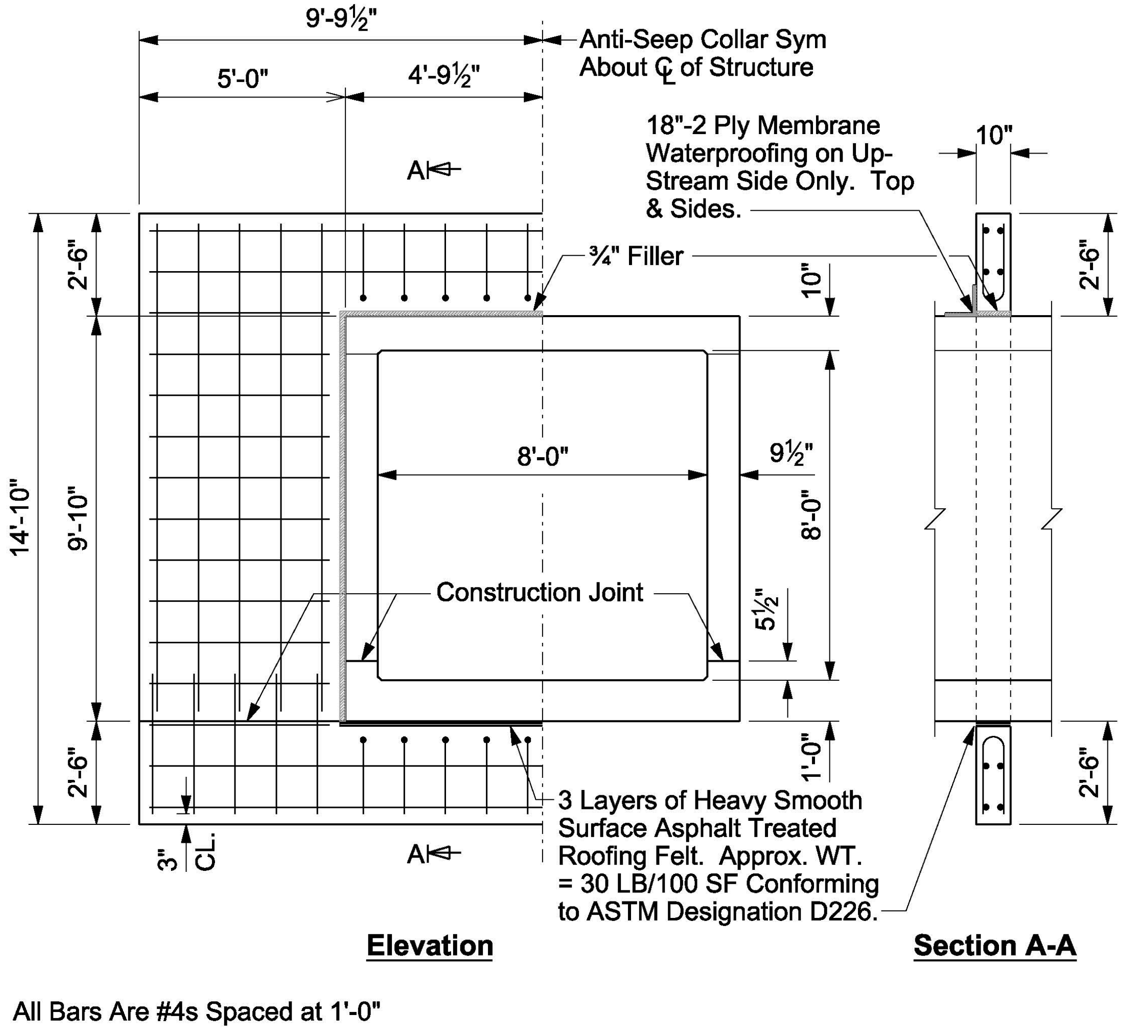



**Figure 8.4-1**
Anti-Seepage Collar

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

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

### 8.4.2.5 Develop Hydraulic Model

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

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

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

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

1. Outlet Control Problem.

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

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

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

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

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

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

2. Inlet Control Problem.

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

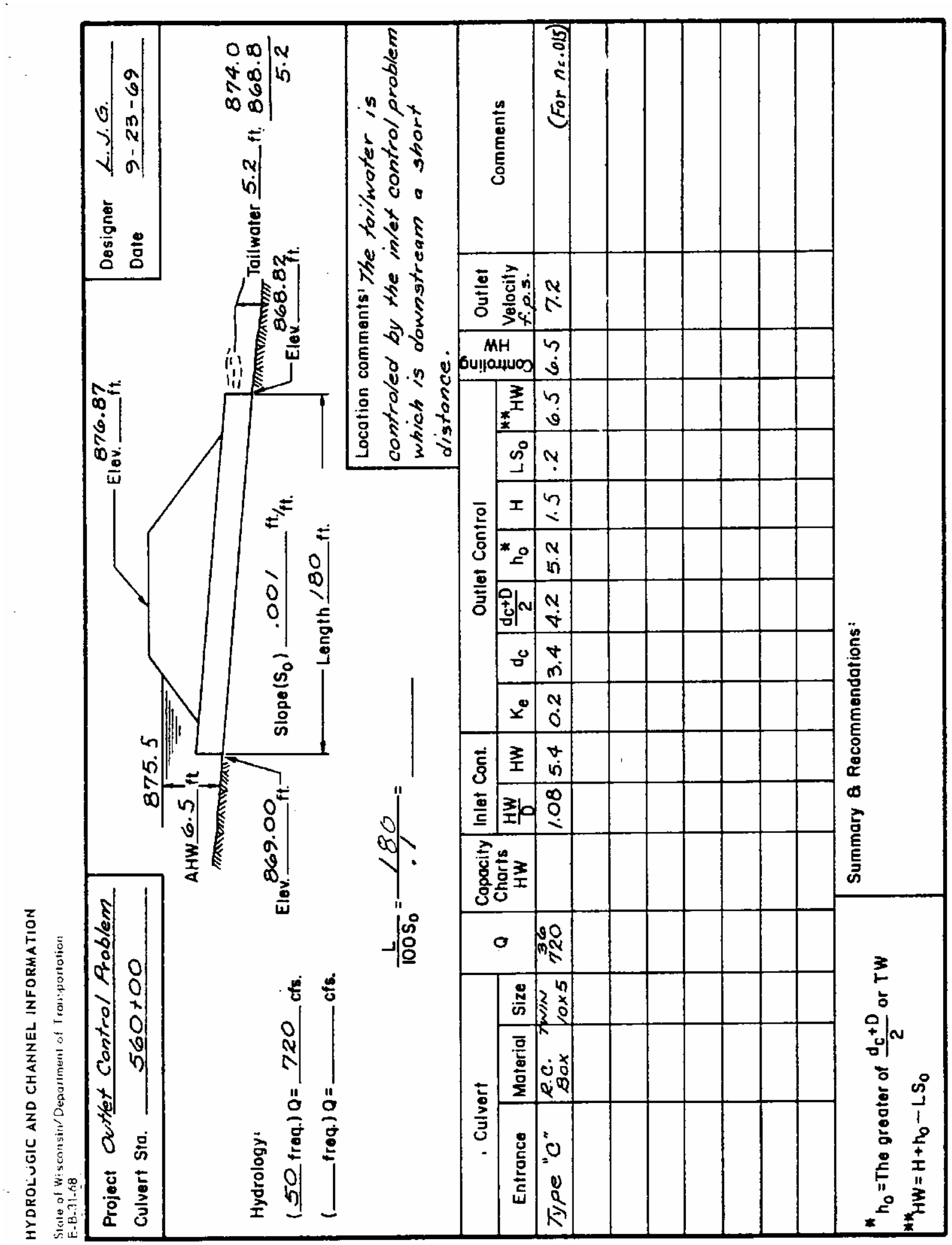
   Check for Inlet Control: For a Q/B value of 36 and a twin 10 x 5 box with type “C” inlet; HW/D = 1.08 from Figure 8.4-3.

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

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

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

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



HYDROLOGIC AND CHANNEL INFORMATION

State of Wisconsin/Department of Transportation
F-B-11-68

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 | Controlling HW | Outlet Velocity f.p.s. | Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Type "C" | R.C. Box | Twin 10x5 | 36 720 | | 1.08 | 5.4 | 0.2 | 3.4 | 4.2 | 5.2 | 1.5 | .2 | 6.5 | 6.5 | 7.2 | (For n=.015) |

* $h_o$ = 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

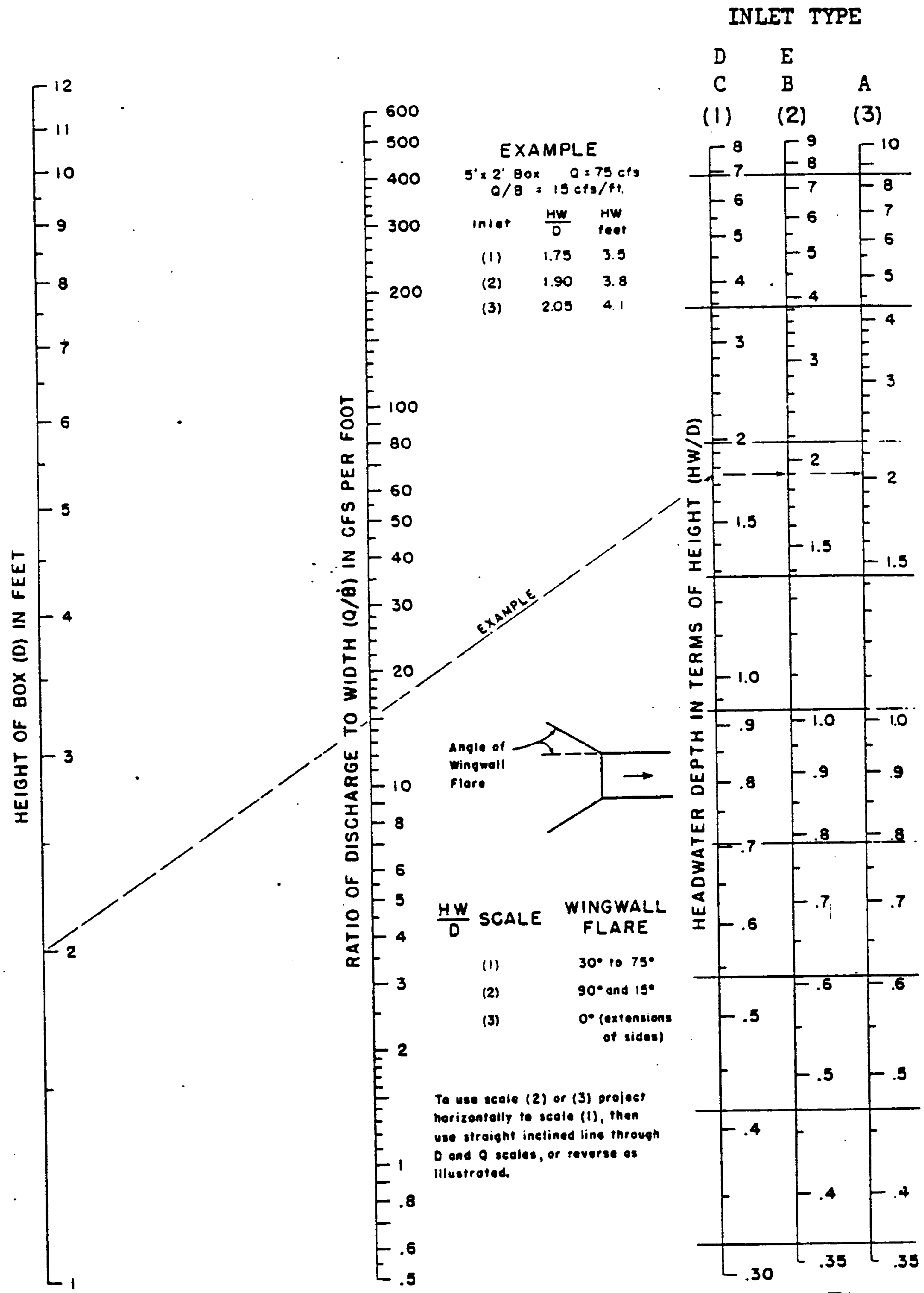



**Figure 8.4-3**
Headwater Depth for Box Culverts with Inlet Control

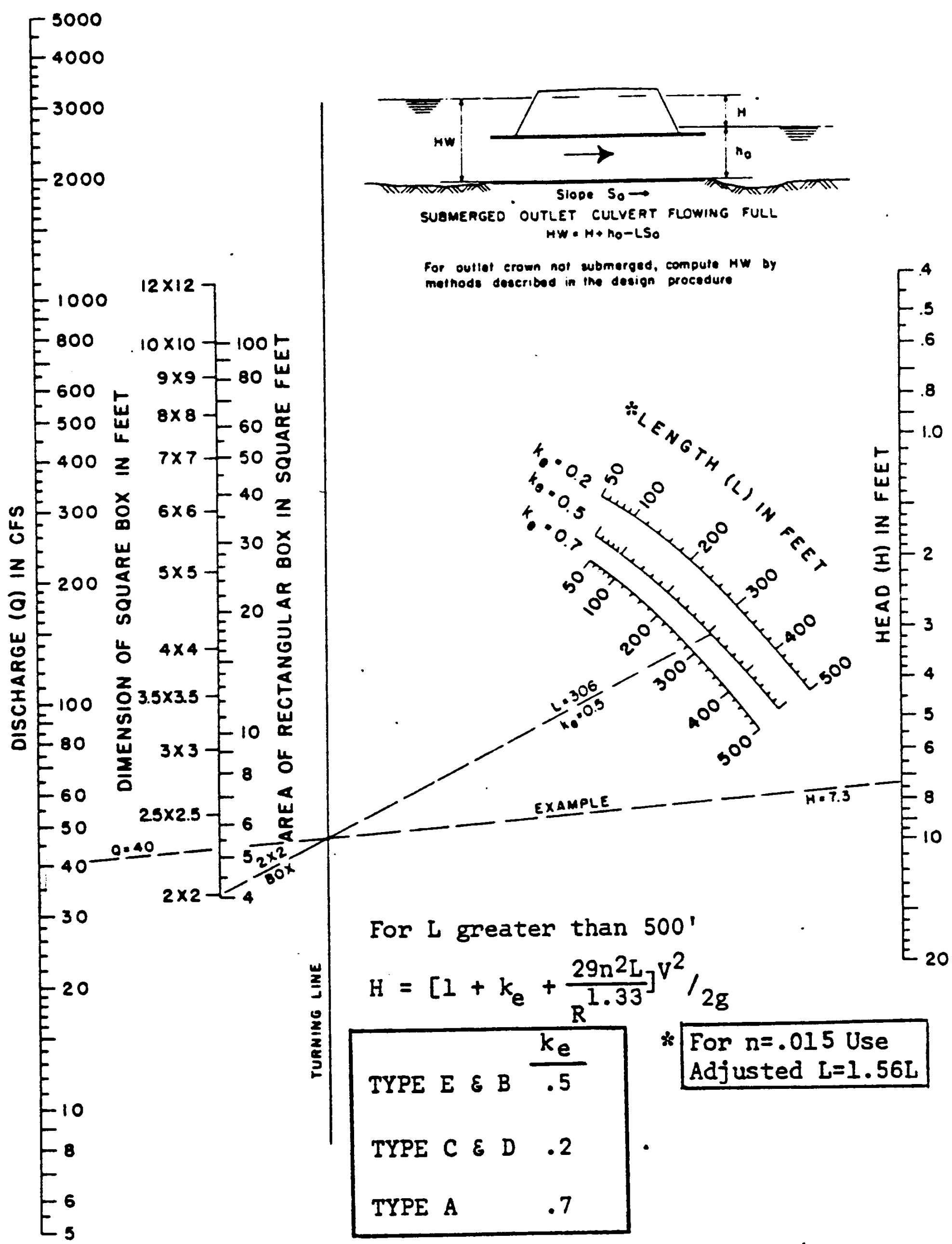
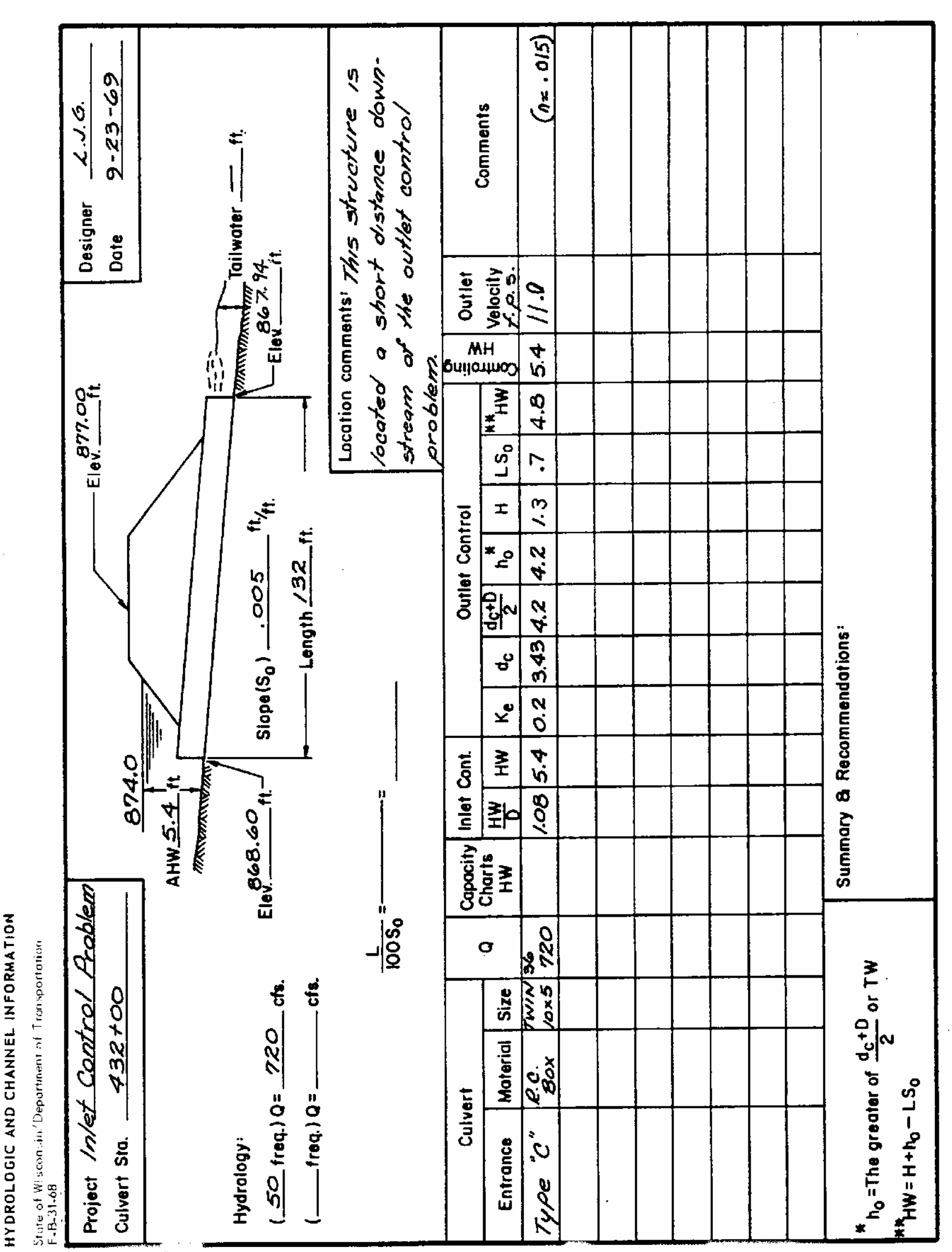



**Figure 8.4-4**

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.

$\frac{L}{100 S_0}$ = ___ = ___

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) |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
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| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |

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

Summary & Recommendations:

**Figure 8.4-5**
Culvert Computation Form

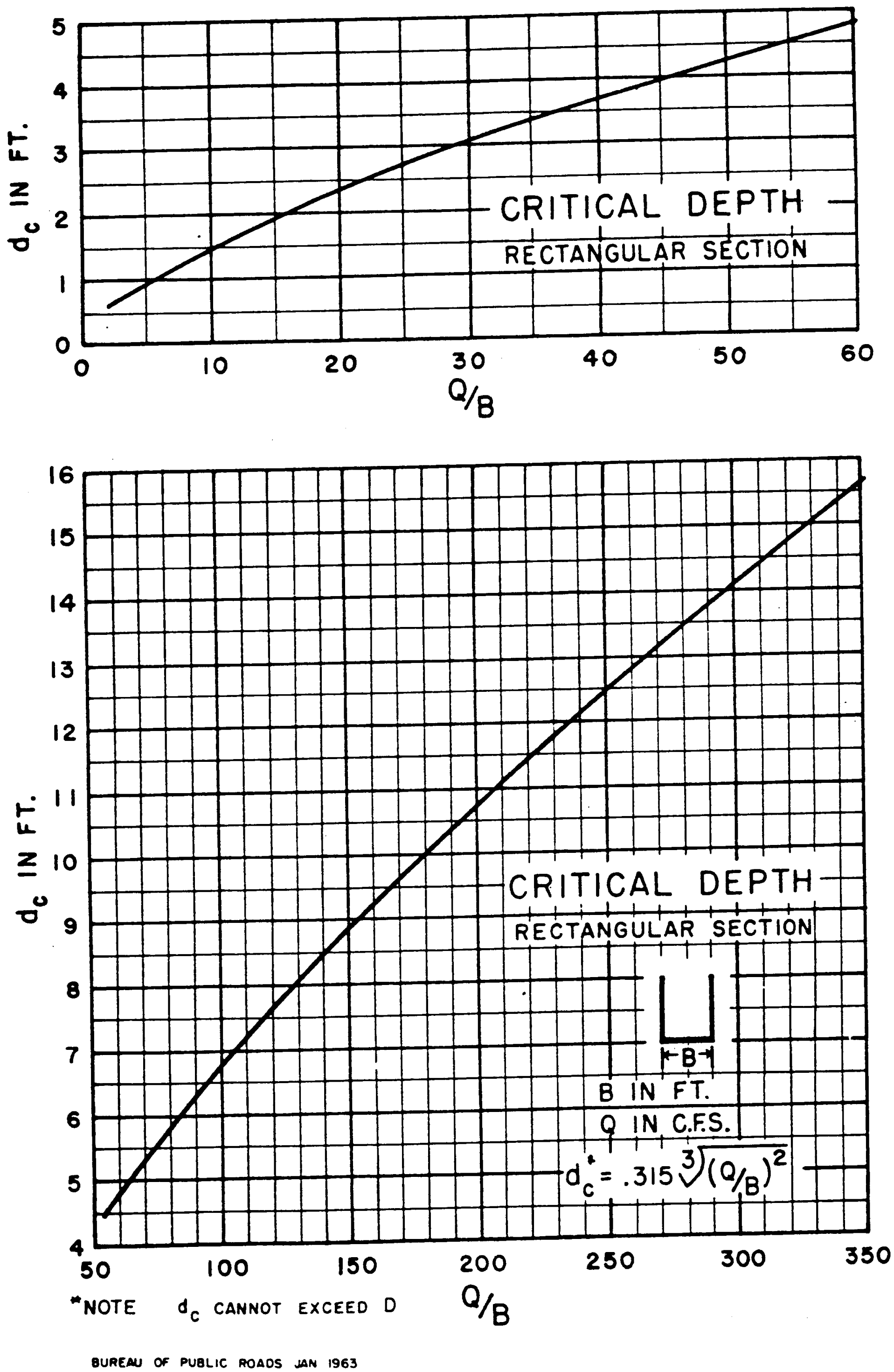


**Figure 8.4-6**
Critical Depth – Rectangular Section

#### 8.4.2.6 Roadway Overflow

See 8.3.2.6.

#### 8.4.2.7 Outlet Scour and Energy Dissipators

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

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

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

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

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

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

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

##### 8.4.2.7.1 Drop Inlet.

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

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

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

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

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

and

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

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

1. Correction for head H/W (Table 8.4-1)

2. Correction for box-inlet shape B/W. (Table 8.4-2)

3. Correction for approach channel width $W_c$/L (Table 8.4-3).

   Where: $W_c$ = approach channel width = Area/Depth

4. Correction for dike effect X/W (Table 8.4-4)

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

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

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

Sample computations are shown in 8.4.2.7.1.1.

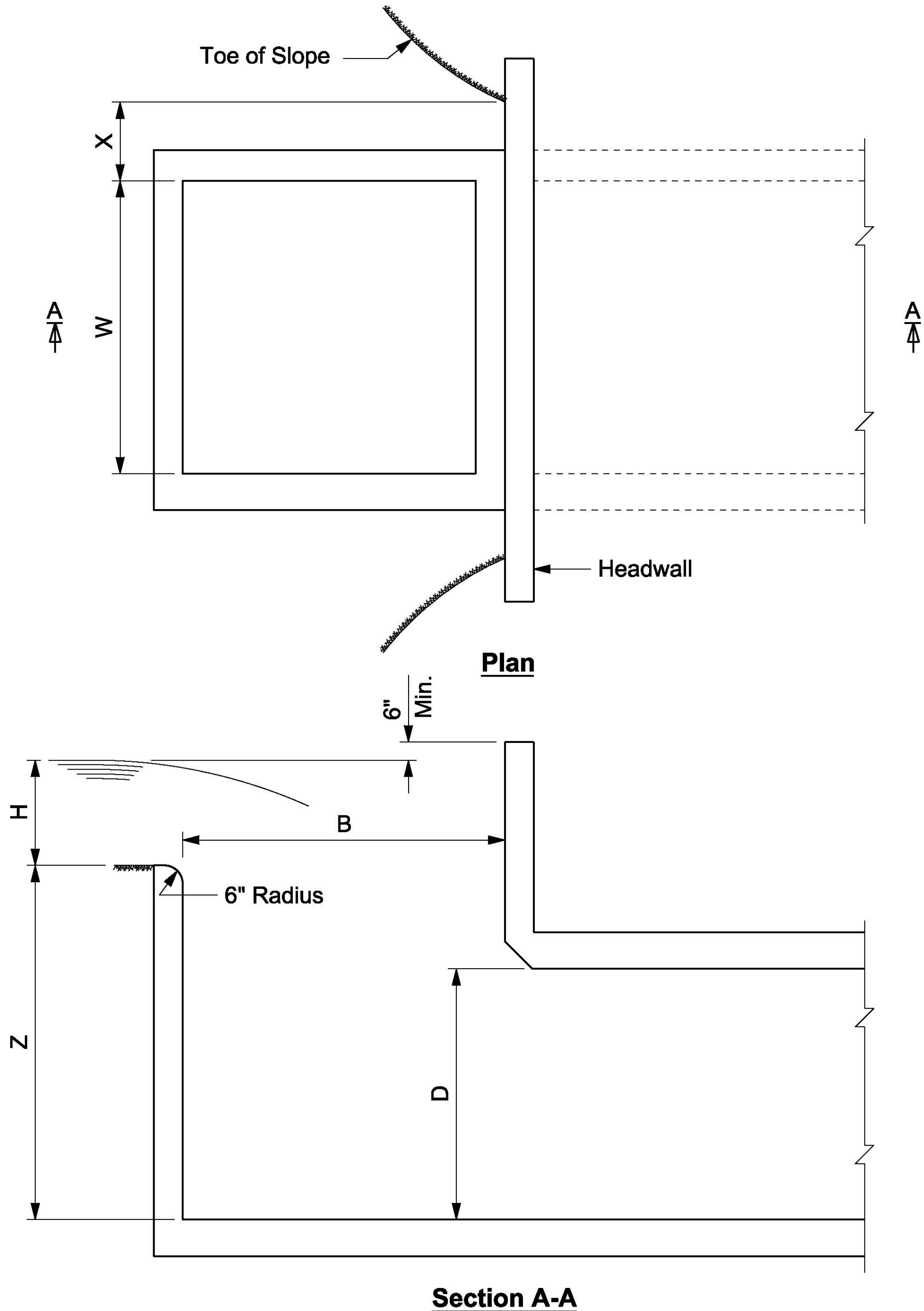


**Figure 8.4-7**
Box Drop Inlet

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

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

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

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

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

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

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

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

8.4.2.7.1.1 Drop Inlet Example Calculations

Given:

Q = 420 cfs through single 9'x6' box

H = 4.4' in a 27 ft. wide channel

Drop = 5 ft

Assume:

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

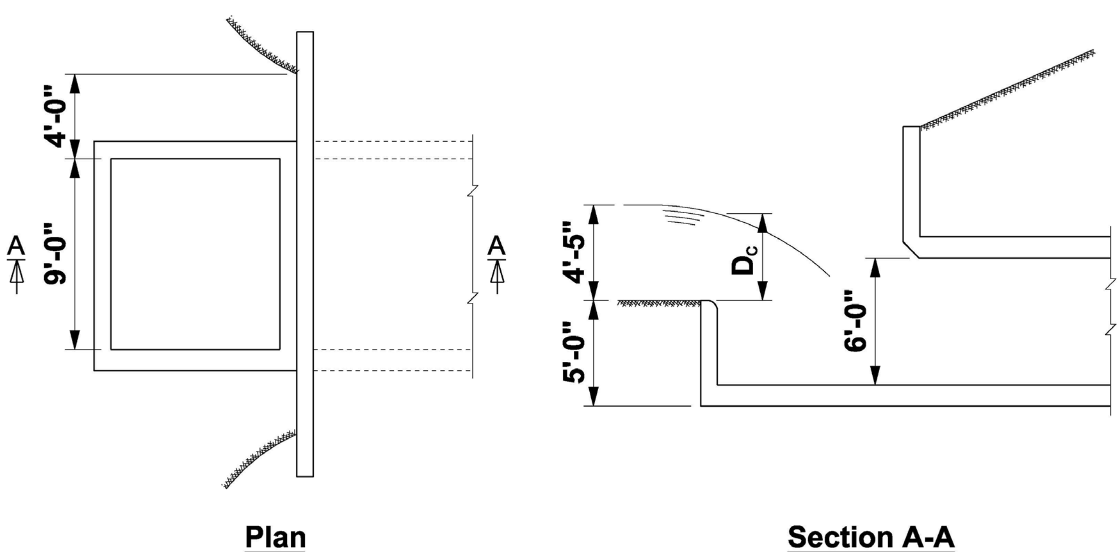


**Figure 8.4-8**
Drop Inlet Example

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

Corrections:

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

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

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

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

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

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

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

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

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

HW = 1.19x6 = 7.14

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

### 8.4.2.7.2 Drop Outlets

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

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

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

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

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

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

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

$X_b = 0.8\ d_c$

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

$X_c \geq 1.75\ d_c$

2. The floor blocks are proportioned as follows:

a. The height of the floor blocks is:

$0.8\ d_c$

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

$0.4\ d_c$

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

c. The floor blocks are square in plan.

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

3. The height of the end sill is:

$0.4\ d_c$

4. The sidewall height above the tailwater level is:

$0.85\ d_c$

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

$d_2 = 2.15\ d_c$

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

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

Where:

| | | |
|---|---|---|
| $y_1$ | = | top of water at crest |
| V | = | velocity of approach |

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

Sample computations are shown in 8.4.2.7.2.1.

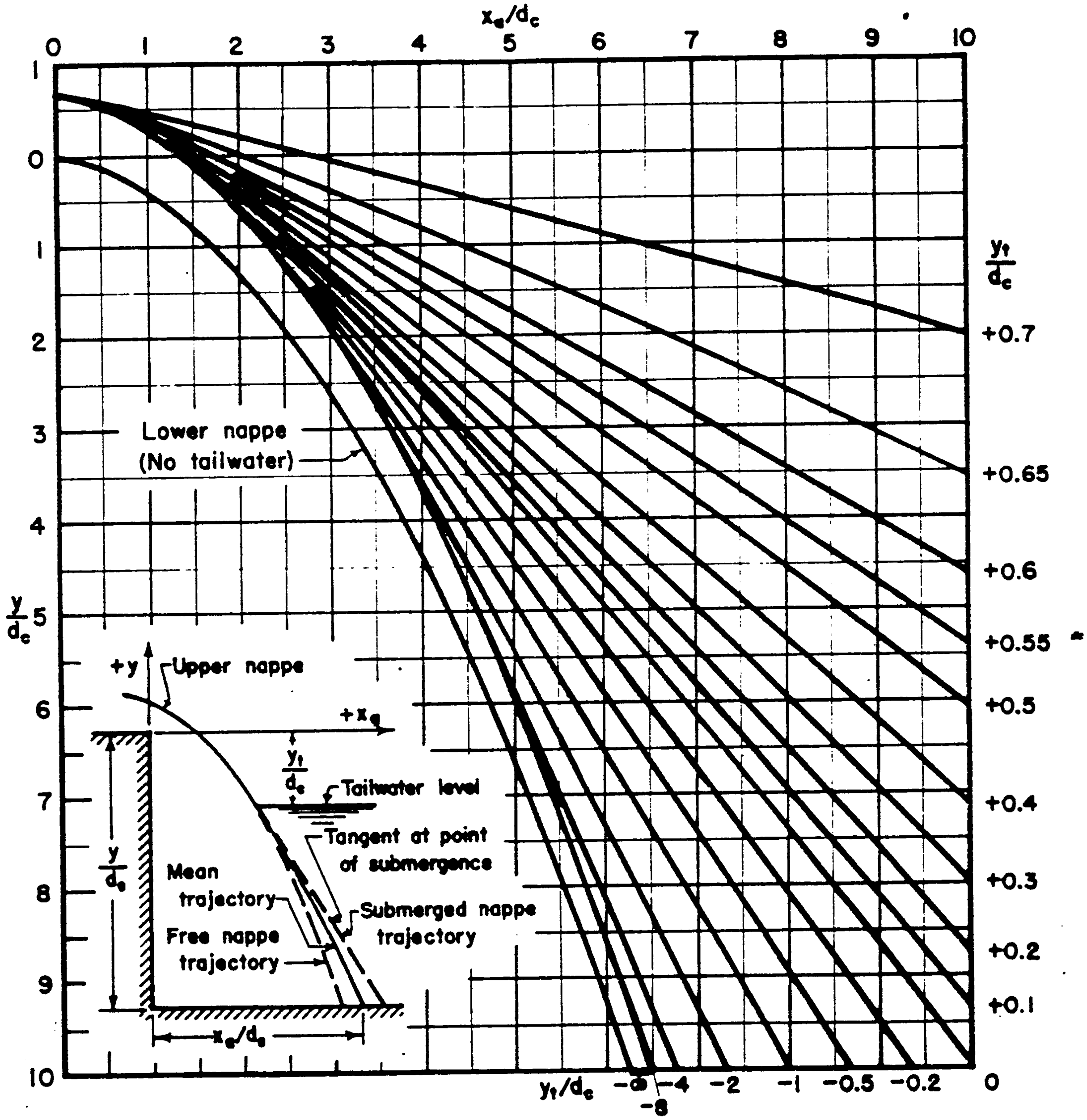


**Figure 8.4-9**
Design Chart for Determination of "$X_a$"

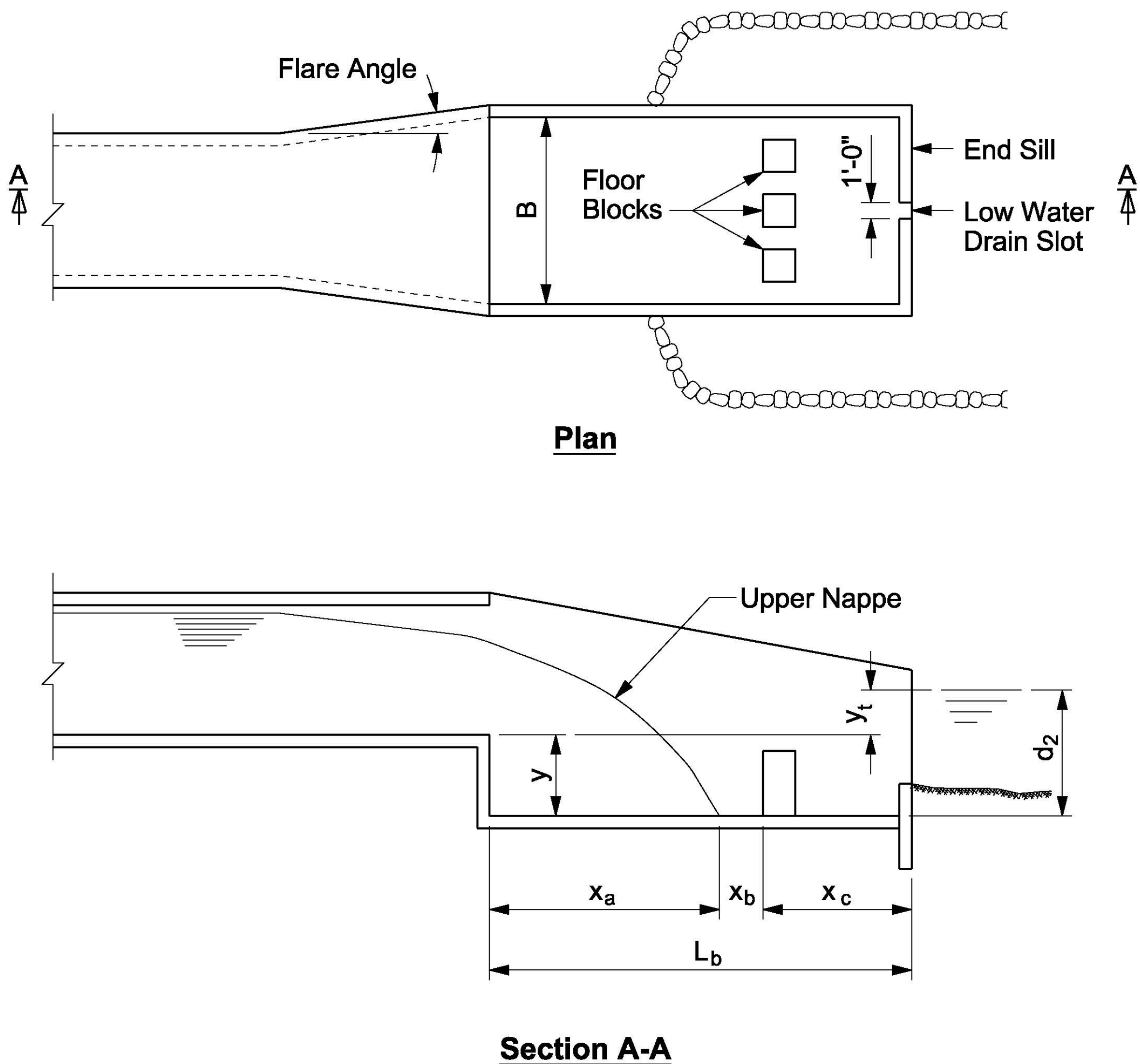


**Figure 8.4-10**
Straight Drop Outlet Stilling Basin

8.4.2.7.2.1 Drop Outlet Example Calculations

Given:

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

Depth = 7.5 ft

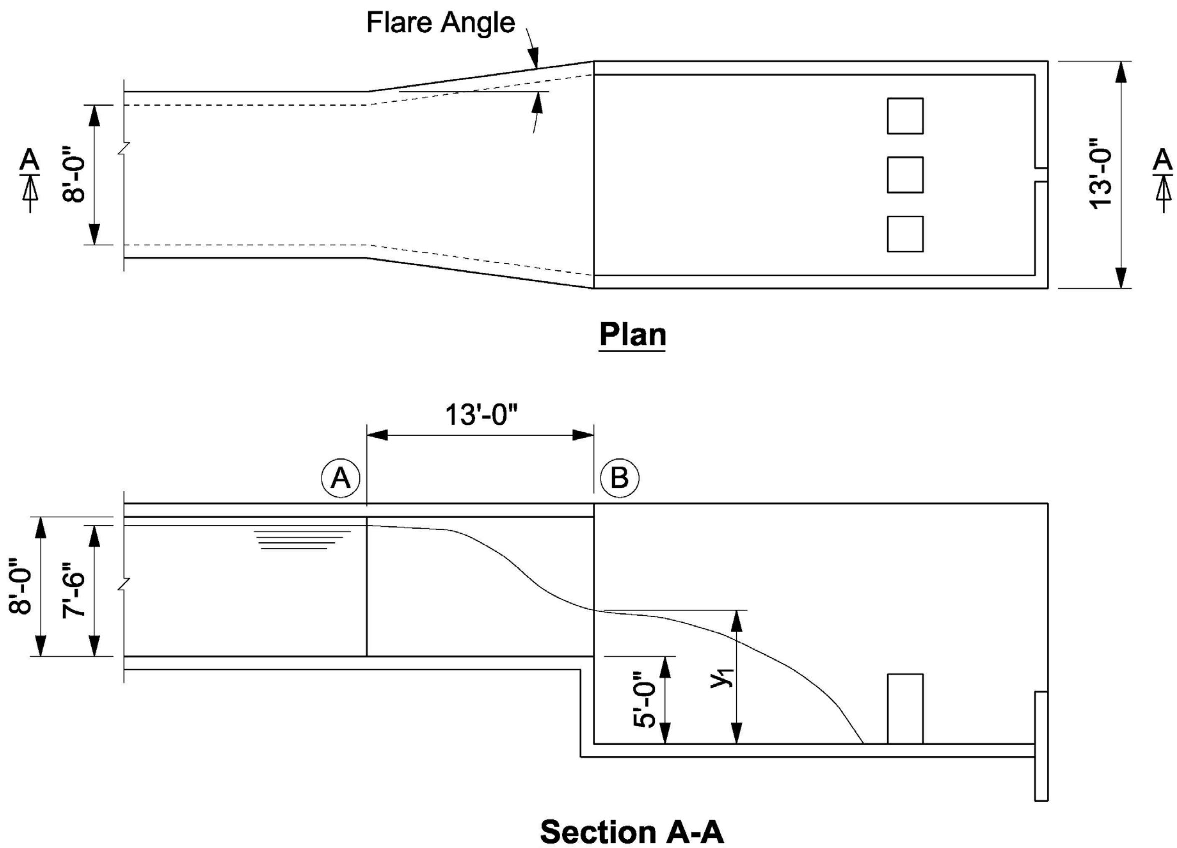


**Figure 8.4-11**
Drop Outlet Example

Assumptions:

- That the specific head of “A” is approximately equal to the specific head at “B”. Therefore, the elevation head + velocity head at “A” = elevation head + velocity head at “B”.

- The end sill height should be less than or equal to 2’-0”.

If the drop were placed at “A”:

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

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

To obtain a 2’-0” sill, set $d_c$ = 2’-0”/0.4 = 5 ft

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

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

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

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

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

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

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

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

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

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

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

Dimensions:

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

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

### 8.4.2.7.3 Hydraulic Jump Stilling Basins

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

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

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

Where:

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

End sill height ($\Delta Z_0$) 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.

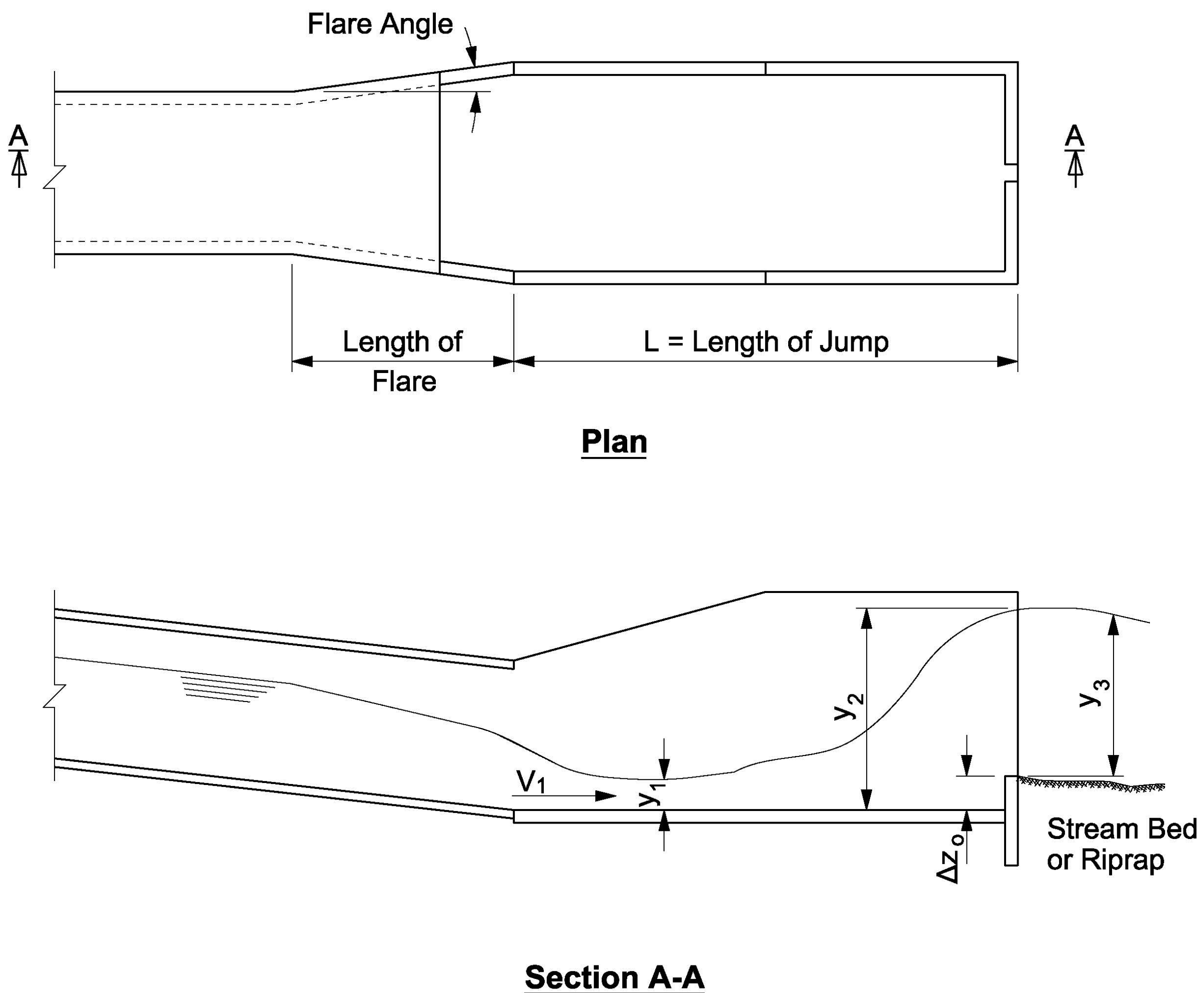


**Figure 8.4-12**
Hydraulic Jump Stilling Basin

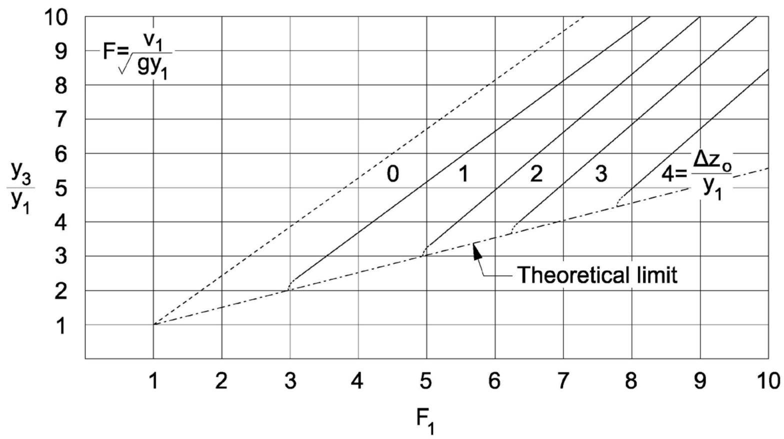


**Figure 8.4-13**
Characteristics of a Hydraulic Jump at an Abrupt Rise

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.

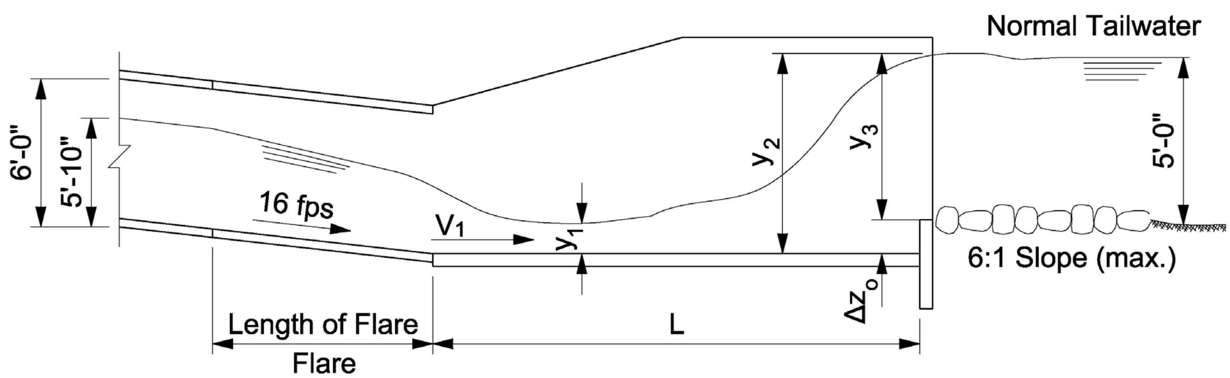


**Figure 8.4-14**
Hydraulic Jump Stilling Basin Example

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 $\dfrac{V_1^2}{2 \cdot g} = 9.775 - 2.2 = 7.575'$

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

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

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

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

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

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

7. U.S. Army Corps of Engineers, *HEC-RAS River Analysis System Users Manual*, (CPD-68), Hydrologic Engineering Center, Davis CA, Version 4.0, March 2008.

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

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

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

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

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

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

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

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

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

17. U.S. Department of Transportation, Federal Highway Administration, *Highways in the River Environment,* Report No. FHWA-HI-90-016, February 1990.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

- Provide a discussion of calculated scour depths 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, but will be updated in the future.

| Code | Title | Year | Publication # | NTIS # |
|---|---|---|---|---|
| HDS 01 | Hydraulics of Bridge Waterways | 1978 | FHWA-EPD-86-101 | PB86-181708 |
| HDS 02 | Highway Hydrology Second Edition | 2002 | FHWA-NHI-02-001 | |
| HDS 03 | Design Charts for Open-Channel Flow | 1961 | FHWA-EPD-86-102 | PB86-179249 |
| HDS 04 | Introduction to Highway Hydraulics | 2001 | FHWA-NHI-01-019 | |
| HDS 05 | Hydraulic Design of Highway Culverts | 2005 | FHWA-NHI-01-020 | |
| HDS 06 | River Engineering for Highway Encroachments | 2001 | FHWA-NHI-01-004 | |
| HEC 09 | Debris Control Structures Evaluation and Countermeasures | 2005 | FHWA-IF-04-016 | |
| HEC 11 | Design of Riprap Revetment | 1989 | FHWA-IP-89-016 | PB89-218424 |
| HEC 14 | Hydraulic Design of Energy Dissipators for Culverts and Channels | 2006 | FHWA-NHI-06-086 | |
| HEC 15 | Design of Roadside Channels with Flexible Linings, Third Edition | 2005 | FHWA-IF-05-114 | |
| HEC 17 | The Design of Encroachments on Flood Plains Using Risk Analysis | 1981 | FHWA-EPD-86-112 | PB86-182110 |
| HEC 18 | Evaluating Scour at Bridges, Fifth Edition | 2012 | FHWA-HIF-12-003 | |
| HEC 20 | Stream Stability at Highway Structures Fourth Edition | 2012 | FHWA-NIF-12-004 | |
| HEC 21 | Bridge Deck Drainage Systems | 1993 | FHWA-SA-92-010 | PB94-109584 |
| HEC 22 | Urban Drainage Design Manual Second Edition | 2001 | FHWA-NHI-01-021 | |
| HEC 23 | Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance Third Edition, Volume 1 | 2009 | FHWA-NHI-09-111 | |
| HEC 23 | Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance Third Edition, Volume 2 | 2009 | FHWA-NHI-09-112 | |
| HEC 24 | Highway Stormwater Pump Station Design (cover) | 2001 | FHWA-NHI-01-007 | |
| HEC 24 | Highway Stormwater Pump Station Design | 2001 | FHWA-NHI-01-007 | |
| HEC 25 | Tidal Hydrology, Hydraulics, and Scour at Bridges | 2004 | FHWA-NHI-05-077 | |
| HEC 25 | Highways in the Coastal Environment - 2nd edition | 2008 | FHWA-NHI-07-096 | |
| HRT | Assessing Stream Channel Stability at Bridges in Physiographic Regions | 2006 | FHWA-HRT-05-072 | |
| HRT | Effects of Inlet Geometry on Hydraulic Performance of Box Culverts | 2006 | FHWA-HRT-06-138 | |
| HRT | Junction Loss Experiments: Laboratory Report | 2007 | FHWA-HRT-07-036 | |
| HRT | Hydraulics Laboratory Fact Sheet | 2007 | FHWA-HRT-07-054 | |

| Code | Title | Year | Publication # | NTIS # |
|---|---|---|---|---|
| Other | Geosynthetic Design and Construction Guidelines | 1995 | FHWA-HI-95-038 | PB95-270500 |
| Other | Underwater Evaluation And Repair of Bridge Components | 1998 | FHWA-DP-98-1 | |
| Other | Best Management Practices for Erosion and Sediment Control | 1995 | FHWA-FLP-94-005 | |
| Other | Underwater Inspection of Bridges | 1980 | FHWA-DP-80-1 | |
| Other | Culvert Management Systems User Manual | 2001 | FHWA-02-001 | |
| Other | FHWA Hydraulics Library on CD-ROM FHWA Hydraulics Library on CD-ROM (Updated Browser) | 2002 | | |
| Other | Hydraulic Performance of Curb and Gutter Inlets | 1999 | FHWA-KU-99-1 | |
| Other | Culvert Management Systems Source Code | 2001 | | |
| Other | NCHRP Report 25-25 (04) Environmental Stewardship Practices, Procedures, and Policies for Highway Construction and Maintenance | 2004 | | |
| Other | New England Transportation Consortium: Performance Specs for Wood Waste Materials as an Erosion Control Mulch and as a Filter Berm | 2001 | FHWA-NETC 25 | |
| Other | Bridge Scour Protection Systems Using Toskanes | 1994 | FHWA-PA-94-012 | PB95-266318 |
| Other | Structural Design Manual for Improved Inlets and Culverts | 1983 | FHWA-IP-83-6 | PB84-153485 |
| Other | Culvert Inspection Manual | 1986 | FHWA-IP-86-2 | PB87-151809 |
| RD | Bottomless Culvert Scour Study: Phase II Laboratory Report | 2007 | FHWA-HRT-07-026 | |
| RD | Effects of Gradation and Cohesion on Scour, Volume 2, "Experimental Study of Sediment Gradation and Flow Hydrograph Effects on Clear Water Scour Around Circular Piers" | 1999 | FHWA-RD-99-184 | PB2000-103271 |
| RD | Effects of Gradation and Cohesion on Scour, Volume 1, "Effect of Sediment Gradation and Coarse Material Fraction on Clear Water Scour Around Bridge Piers" | 1999 | FHWA-RD-99-183 | PB2000-103270 |
| RD | Portable Instrumentation for Real Time Measurement of Scour At Bridges | 1999 | FHWA-RD-99-085 | PB2000-102040 |
| RD | Users Primer for BRI-STARS | 1999 | FHWA-RD-99-191 | PB2000-107371 |
| RD | Effects of Gradation and Cohesion on Scour, Volume 3, "Abutment Scour for Nonuniform Mixtures" | 1999 | FHWA-RD-99-185 | PB2000-103272 |
| RD | Remote Methods of Underwater Inspection of Bridge Structures | 1999 | FHWA-RD-99-100 | PB9915-7968 |
| RD | Hydraulics of Iowa DOT Slope-Tapered Pipe Culverts | 2001 | FHWA-RD-01-077 | |

| Code | Title | Year | Publication # | NTIS # |
| --- | --- | --- | --- | --- |
| RD | Users Manual for BRI-STARS | 1999 | FHWA-RD-99-190 | PB2000-107372 |
| RD | Effects of Gradation and Cohesion on Scour, Volume 4, "Experimental Study of Scour Around Circular Piers in Cohesive Soils" | 1999 | FHWA-RD-99-186 | PB2000-103273 |
| RD | Effects of Gradation and Cohesion on Scour, Volume 5, "Effect of Cohesion on Bridge Abutment Scour" | 1999 | FHWA-RD-99-187 | PB2000-103274 |
| RD | Effects of Gradation and Cohesion on Scour, Volume 6, "Abutment Scour in Uniform and Stratified Sand Mixtures" | 1999 | FHWA-RD-99-188 | PB2000-103275 |
| RD | Durability Analysis of Aluminized Type 2 Corrugated Metal Pipe | 2000 | FHWA-RD-97-140 | |
| RD | Performance Curve for a Prototype of Two Large Culverts in Series Dale Boulevard, Dale City, Virginia | 2001 | FHWA-RD-01-095 | |
| RD | Bottomless Culvert Scour Study: Phase I Laboratory Report | 2002 | FHWA-RD-02-078 | |
| RD | Bridge Scour in Nonuniform Sediment Mixtures and in Cohesive Materials: Synthesis Report | 2003 | FHWA-RD-03-083 | PB-2204-104690 |
| RD | Enhanced Abutment Scour Studies For Compound Channels | 2004 | FHWA-RD-99-156 | |
| RD | Field Observations and Evaluations of Streambed Scour at Bridges | 2005 | FHWA-RD-03-052 | |
| RD | South Dakota Culvert Inlet Design Coefficients | 1999 | FHWA-RD-01-076 | |

**Figure 8.7-1**

FHWA Hydraulic Engineering Publications

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

**Figure 8.7-2**
FHWA Hydraulics Software List

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