Chapter Three
Hydrology and Hydraulics

3.1 INTRODUCTION

The objectives of a hydraulic design are to identify the stream forces that may cause harm to the bridge or roadway system and to provide a safe level of service acceptable to the needs of the traveling public without unreasonable effect on adjacent property.

3.1.1 POLICY AND COORDINATION

Consideration of the effects of constructing a bridge across a waterway is key to assuring the long-term stability of the structure. Confining the floodwater may cause excessive backwater or overtopping of the roadway, or it may induce excessive scour. These effects may result in damage to upstream land and improvements or endanger the bridge. Conversely, an excessively long bridge may cause flooding downstream or cost far more than can be justified by the benefits obtained. Somewhere between these extremes is the design that will be the most economical to the public over a long period of time.

The designer must evaluate existing upstream conditions as well as future upstream development—25 years, if possible—in sizing a structure. The amount of information available to assist the designer in predicting future development is limited. Available information includes the following:

- DelDOT's Division of Planning projects land use data for 20 years. The projections include population, dwelling units, number of vehicles, and employment for each traffic zone in the counties' system zones as well as the area type—city, urban, inner suburban, outer suburban, or rural.
- Each county maintains zoning maps, which show the current zoning status of land within the county.
- The Department of Planning in each county prepares a comprehensive development plan which presents their concept of the optimum use of the land within the county. The plans include detailed maps that show the current and proposed use of all lands in the county. These plans must be used with judgment since they do not necessarily conform with current zoning and the land may not be developed within the projected time frame.

Article 10 of the *New Castle County Unified Development Code* establishes criteria for structures in or near floodplains and floodways. All projects in New Castle County are subject to this ordinance. Any structure to be located, relocated, constructed, reconstructed, extended, enlarged, or structurally altered within a designated floodplain is subject to the County Code. The major items that must be included in the application procedures that affect structure designers are as follows:

- the site location and tax parcel number;
- a brief description of the proposed work;
- a plan of the site showing the exact size and location of the proposed construction as well as any existing structures;
- an engineering analysis of the impact on the floodplain utilizing HEC-RAS, WSPRO or other acceptable backwater analysis model;
- an accurate delineation of the floodplain area, including the location of any adjacent floodplain development or structures and the location of any existing or proposed subdivision and land development;
- delineation of existing and proposed contours;
- information concerning the 100-year flood elevations and other applicable information such as the size of structures, location and elevation of streets, water supply and sanitary sewer facilities, soil types, and flood-proofing measures; and
- a document, certified by a registered professional engineer, which states that any proposed construction has been adequately designed to withstand the 100-year flood pressures, velocities, impact and uplift forces, and other hydrostatic, hydrodynamic and buoyancy factors associated with the 100-year flood.

Refer to Appendix 1 of the *New Castle County Unified Development Code* for the specific requirements.

Projects in New Castle, Kent, and Sussex Counties are subject to the regulations administered by the Federal Emergency Management Agency (FEMA). However, New Castle County's Unified Development Code contains more stringent requirements concerning increases in water surface profiles. When water surface profiles are increased greater than permitted by the FEMA regulations, a Conditional Letter of Map Revision (CLOMR) is required. Refer to FEMA publication *Procedures for Coordinating Highway Encroachments on Floodplains with Federal Emergency Management Agency*.

3.1.2 DESIGN RESPONSIBILITIES

Responsibilities for drainage design are divided between the Bridge Design Section and the Project Development sections based primarily on the size of the drainage area. Bridge Design is responsible for all watersheds over 300 acres [120 hectares] and locations where the existing structure opening exceeds 20 ft^2 [1.86 m^2]. Project Development is responsible for watersheds smaller than 300 acres [120 hectares].

The Bridge Design Section is responsible for design of all locations requiring structures exceeding 20 ft^2 [1.86 m^2] of waterway opening for a single location. Bridge Design will also be responsible for design of pipe culverts, closed drainage systems, roadside ditches, and stormwater management on "bridge only" projects.

The guidelines noted in this chapter are applicable to all structures and culverts.

Refer to Chapter Six of the *Road Design Manual* for the design and construction of adjacent drainage ditches, pipe culverts, closed drainage systems, and erosion control near stream crossings.

If the designer is using a design flood frequency less than that shown in Figure 3-2, a risk analysis must be performed. This needs approval by the Bridge Design Engineer. Where a risk analysis is needed, a complete hydraulic report should be prepared giving consideration to each alternative under study. The lower level of study, risk assessment, should always be considered as the first course of action. Only if a detailed economic accounting of the risks and potential harm is needed should an extensive risk analysis be performed. A detailed analysis must be performed in situations involving substantial losses resulting in high encroachment costs. It will always be necessary to apply good engineering judgment in determining the level of evaluation to be performed. Information found in *HEC-17, Design of Encroachments of Flood Plains Using Risk Analysis,* may provide the needed guidance.

3.1.3 HYDRAULIC ASSESSMENT CHECKLIST

Completion of the Hydraulic Assessment Checklist (Figure 3-1) documents the design of the structure as well as providing guidance to the designer to ensure that all items are completed. The designer is responsible for completing the checklist. It is completed in three phases:

- Part I, General Site Data, and Part II, Existing Structure, are to be completed in the office in preparation for the field inspection;
- Part III, Field Inspection, provides guidelines to ensure that all items are completed during the field inspection; and
- Part IV, Proposed Structure, documents the design of the proposed structure and the level of evaluation.

3.1.4 TOPOGRAPHIC SURVEY

The designer must request a survey. Any specific information needed for the Hydraulic Checklist or information in addition to that normally required must be included in the designer's request. See Chapter Two for a sample survey request form.

Figure 3-1a
Hydraulic Assessment Checklist for Drainage Design

Part I. General Site Data
Bridge No. ________ Project No. _______________________________________
County: New Castle __ Kent __ Sussex __ Region: North __ South __
Road Number _________ Road Name: ____
Project Location: ____
Site Name: ____________________________________ Station: ________ M.P. ______
Site Description: Cross Road Drain __, Tax Ditch __, Storm Drain __,
Channel Change __, Tidal __, Long. Encroach. __, Other ___________________
Project Type: Repair ___, Replacement ___, Relocation ___
Stream Name: ____
Functional Class: __ Interstate __ Collector
__ Major Arterial __ Local
__ Minor Arterial __ Entrance/Side Road
Traffic Data: Current ADT ______ Year______ % Trucks_____
Design ADT ______ Year ______ % Trucks ______
Design Speed ______ Design H. Vol. (DHV) _______
Route Importance: Defense route ____
Emergency, School ____________________________________
Can route be closed? Yes __ No __
Adequate detour? Yes __ Length ____________________
No __Restrictions __________________
Temporary pedestrian crossing required? Yes __ No __

Part II. Existing Structure (From existing plans and/or survey)
Survey source: Field __ Aerial __ Other ______________________________________
Survey datum: ____
Date survey received: ________ From: ____
Description: ____
Existing Plans Available: Yes __ No __
Roadway Approach Width: _____ Bridge Width: ___ (Curb to Curb) ____ (Out to Out)
Streambed Elevation: ____ Avg. Low Steel Elevation or Top of Opening Elevation ____
Underclearance: _____ Overtopping Elevation _____
(Bottom of beam to normal water surface elevation)
Clearance from Flood of Record _________ Existing waterway opening ___________
Drainage area (Total) ________ Acres
Drainage area controlled by upstream storage _____ Acres
Navigable waterway? Yes __ No __

Figure 3-1b
Hydraulic Assessment Checklist for Drainage Design

Part III. Field Inspection
1. Site inspected by: ______________________________ Date: __________________
2. Survey appears correct? Yes __ No __ Apparent errors: ________________________
____
3. Flooding apparent? No __ Yes __; HW marks obtained Yes __ No __ because ______
____
4. Damage from previous floods: Yes __ (If yes, attach report) No __
Road closings for floods (Dates/hours): _________________________________
____
H. W. of record: Elevation ____
Circumstances: ____
____
5. Channel cross sections obtained? Yes __ No __, Because ________________________
____
6. Channel unstable? No __ Yes __ Because of __ headcutting observed and __ amount and location obtained, __ bank caving, __ braiding, __ increased meander activity, __ degrading, __ aggrading, __ other ____
On meander ____, bend ____, or straightaway ____
Channel alignment: ____
Slope _________________________
Ordinary high water (Active channel) elevation _______________________________
100-yr. Floodplain elevation (FEMA) ______________________________________
Floodway elevation (FEMA) ____
Regulated floodway ___________________________________
Permits required: COE ___ Coast Guard ___ Subaqueous ___ NCC ___
Evidence of drift ____
Evidence of scour ____
Evidence of ice damage ____
Evidence of overtopping damage to road _________________________________
Bank protection needed ____
Energy dissipater needed ____
Channel change needed ____
Raise/lower profile grade ____
Upstream erosion control needed _______________________________________
Downstream apron needed ____
7. Structure scour in evidence No __ Minor __ Yes __ and __ obtained bed/bank samples and __ noted any flow alignment problems, Yes _ and bed/bank samples not obtained and __ flow alignment not noted because _____________________________________
8. Manning's "n" obtained? Yes __ No __ Because ______________________________
Description of banks and channels: _____________________________________
____

Figure 3-1c
Hydraulic Assessment Checklist for Drainage Design

9. Property damage due to flooding? No __ Yes __
Elevation/property type checked, Yes __, No __ because: ___________________________________
____
Controls affecting water surface elevation:

Structures:
RR bridge ___ Location ___ feet (upstream/downstream)
Highway bridge ___ Location ___ feet (upstream/downstream)
Rock outcrop ___ Location ___ feet (upstream/downstream)
Other____

10. Environmental hazards present? No __ Yes __ and details obtained, Yes __, No __ because
____ Not sure ____

11. Ground photos taken? __ Upstream floodplain and all property, __ Downstream floodplain and all property, __ Site looking downstream, __ Site looking upstream, __ Channel material w/scale, __ Evidence of channel instability, __ Evidence of scour, __ Existing structure inlet/outlet, __ Other __________________________________

12. Land use:

Rural
Pasture ____%
Cultivated ____%
Forest Cover ____ Hectares

Suburban ____%
Density ____Houses/hectare

Urban
Residential____ %
Industrial/commercial____ %

High runoff area (Type D Soils) _____ Acres
Low runoff area (Type A Soils) _____ Acres
Potential for future development: ____
Upstream development:

Residence ____ Story
Basement? ____
Occupied ____
Condition ____
Location ____
1st fl. elevation ____

Industry type __________________
Locations __________________
Condition __________________

13. Should new structure be considered? Yes __ No __
If yes, reasons:

Inadequate roadway: ____
Inadequate clearance: ____
Inadequate loading: ____
Inadequate waterway: ____
Structural deterioration: ____
Route relocation: ____

Sufficiency rating: _____
Date of suff. rating: _____ Year
Posted: _____
Year built: _____
Year modified: _____

Figure 3-1d
Hydraulic Assessment Checklist for Drainage Design

Part IV. Proposed Structure

Bridge description: ____

Approach roadway width: _____________ Bridge roadway width: _____________

Horizontal alignment: Tangent _______ Curve _______ Skew _______ °

Superelevation: _______

Vertical alignment: Tangent grade: _______ %

Vertical curve: G1 _______ % G2 _______ % L _______

PI Station: _______ PI Elevation: _______

Underclearance: _______ feet

Low superstructure element elev. _______ at _______ year design flood

Overtopping elevation: _______ Freeboard: ______________

Storm	Discharge	Existing HW Elevation	Existing Backwater Elevation	Proposed HW Elevation	Proposed Backwater Elevation
Q_{25}	_______	_______	_______	_______	_______
Q_{50}	_______	_______	_______	_______	_______
Q_{100}	_______	_______	_______	_______	_______
Q_{500}	_______	_______	_______	_______	_______
Overtopping Q	_______	_______	_______	_______	_______

Location of overtopping: Bridge __ Roadway __ (Station)

Design frequency: _______

Total waterway provided: _______ Design waterway provided: _______

Average velocity @ Q_{des} _______

Reference and Special Reports Available:

________ Field Check Report
________ Designer's Hydraulic Report
________ Risk Assessment Report
________ Risk Analyses Report
________ Corps of Engineers Flood Plain Information Report
________ Corps of Engineers Study
________ USGS Study
________ SCS Watershed Report
________ FEMA—FIA Flood Insurance/Zoning Report
________ Other

List Attachments

1. ____
2. ____
3. ____
4. ____

3.1.5 HYDRAULIC REPORT

The minimum items required in a hydraulic report include:

- a location map;
- a hydrologic and hydraulic narrative that provides site data, a hydraulic analysis, and flood profiles for the design year, the 100-year and 500-year floods; and
- recommendations.

The report should be supplemented, as needed, with:

- photographs as applicable;
- USGS topographic maps;
- flood frequency curves;
- stage discharge curves for existing and proposed conditions;
- a scour analysis;
- water surface profiles for existing and proposed conditions based on HEC-RAS, WSPRO or other approved method;
- a 100-year flood boundary plan;
- stream cross sections;
- stream and flood surface profiles; and
- hydraulic assessment list.

3.2 HYDROLOGY

3.2.1 INTRODUCTION

Hydrologic analysis is used to determine the rate of flow, runoff, or discharge that the drainage facility will be required to accommodate.

3.2.2 DOCUMENTATION

The design of highway facilities should be adequately documented. Frequently, it is necessary to refer to plans, specifications, and hydrologic analyses long after the actual construction has been completed. One of the primary reasons for documentation is to evaluate the hydraulic performance of structures after large floods to determine whether the structures performed as anticipated or to establish the cause of unexpected behavior. In the event of failure, it is essential that contributing factors be identified to avoid recurring damage.

The documentation of a hydrologic analysis is the compilation and preservation of all pertinent information on which the hydrologic decision was based. This might include drainage areas and other maps, field-survey information, source references, photographs, hydrologic calculations, flood-frequency analyses, stage-discharge data and flood history, including narratives from highway maintenance personnel and local residents who witnessed or had knowledge of an unusual event.

Hydrologic data shown on project plans ensures a permanency of record, serves as a reference in making plan reviews, and aids field engineers during construction. Sample plan presentation data are provided in Section 3.6.

3.2.3 ESTIMATING FLOOD RUNOFF AND MAGNITUDES

In Delaware, there are six methods of estimating flood magnitudes in sizing a waterway opening for a given structure:

1. USGS Method,
2. TR-55,
3. Recorded Data,
4. Published Reports,
5. Rational Method, and
6. TR-20.

3.2.3.1 USGS Method

DelDOT uses the equations in the current version of the US Geological Survey (USGS) publication *Technique for Estimating Magnitude and Frequency of Floods in Delaware* to estimate flood runoff for drainage areas of 300 acres [120 hectares] or greater. This is the method typically used by DelDOT. These equations are based on specific studies of the watersheds in Delaware and adjacent states. This method relies on data from streamflow gaging station records combined statistically within a hydrologically homogenous region to produce flood-frequency relationships applicable throughout the region. If the designer is using gaging station records and wishes to evaluate these values for upstream or downstream sites, the procedures in the USGS publication should be followed.

From the study, it was concluded that reasonable estimates of flood runoff can be made by dividing the State into two regions. In the northern region, only the size of the drainage area, basin development factor, and storage are considered in the equations; the other factors that affect runoff are considered in the constants and exponents. In the southern region, basin relief, forest cover, and two soil categories must be considered in addition to the drainage area and storage. Each of these is discussed in the following sections.

Land use is not considered in the runoff equations for the Northern Region. Forest cover is a factor in the equations for the Southern Region.

In areas where land use may change, the TR-55 method should be used. If USGS is used, the designer is cautioned to consider the effects of possible changes in land use.

3.2.3.2 TR-55

The TR-55 program can generate and plot hydrographs, compute peak discharges, and perform detention pond storage estimates. It can account for hydrograph shift and attenuation due to reach routing. The program uses the Graphical Peak Discharge Method to compute peak flows for different return events in a watershed. TR-55 is a Soil Conservation Service (SCS) program which is applicable to small urban watersheds. Refer to the TR-55 manual for additional limitations to ensure that the degree of error is tolerable.

3.2.3.3 Recorded Data

The method of analyzing flood-frequency relationships from actual streamflow data for a single gaging station enables the use of records of past events to predict future occurrences. Consideration of future development must be included in these predictions. This method assumes that there are no changes in the nature of the factors causing the peak magnitudes. The ramifications of this assumption can be minimized by making every effort to determine the past conditions of the drainage area and, if possible, making allowances for changes. The most common changes are man-made and consist of such modifications as storage and land development. The user of hydrologic data must be acquainted with the procedures for evaluating streamflow data, the techniques for preparing a flood-frequency curve, and the proper interpretation of the curve.

There will be times when estimates made from a regional analysis will not agree with a flood-frequency analysis of a gaging station on the stream being studied. Various factors such as length of runoff records, storm distribution and parameters used in the regional analysis could account for some of the discrepancies. Since all the stream records in Delaware are sufficiently long to give good flood-frequency relationships, considerable weight should be given to the stream record in estimating design floods. When gaging station records are used, the designer should consult the USGS (**http://www.usgs.gov/**) publication *Techniques for Estimating Magnitude and Frequency of Floods in Delaware* and current USGS data.

3.2.3.4 Published Reports

Published reports may be used for comparison with the calculated runoff. The FEMA Flood Insurance Study contains runoff information for many streams in Delaware. The report documents the methods used to determine runoff for each stream. Several reports were prepared by the U.S. Army Corps of Engineers for New Castle County. These reports contain floodplain information for many streams in New Castle County. The reports include historical runoff data as well as calculated runoff.

3.2.3.5 The Rational Method

The rational method is an empirical formula relating rainfall to runoff. It is the method used almost universally for computing urban runoff. It is also used to estimate bridge deck drainage for the design of scuppers.

Discharge, as computed by this method, is related to frequency by assuming the discharge has the same frequency as the rainfall used. Because of the assumption that the rainfall is of equal intensity over the entire watershed, it is recommended that this formula be used only for estimating runoff from small areas, up to 300 acres [120 hectares].

3.2.3.6 TR-20

The TR-20 computer program, developed by the SCS, develops flood hydrographs from runoff and routes the flow through stream channels and reservoirs. Routed hydrographs are combined with those of tributaries. The program provides procedures for hydrograph separation by branching or diversion of flow, and for adding baseflow. Peak discharges, their times of occurrence, water surface elevations and duration of flows can be computed at any desired cross section or structure. Complete discharge hydrographs, as well as discharge hydrograph elevations, can be obtained if requested. The program provides for the analysis of up to nine different rainstorm distributions over a watershed under various combinations of land treatment, floodwater retarding structures, diversions, and channel modifications. Such analyses can be performed on as many as 200 subwatersheds or reaches and 99 structures in any one continuous run.

3.2.4 DESIGN FLOOD FREQUENCY

The design frequencies for bridges and pipe culverts for each highway functional classification are shown in Figure 3-2. If a design frequency less than that shown in Figure 3-2 is used, the design must be based on a risk analysis and must be approved by the Bridge Design Engineer.

3.2.5 FREQUENCY MIXING

Often, the designer is faced with the situation wherein the hydraulic characteristics of the subject facility are influenced by a flood condition of a separate and independent drainage course. For example, a small stream may outfall into a major river that itself is an outfall for a large and independently active watershed. It can reasonably be expected that these two waterways would seldom peak at the same time. Consequently, there are two independent events: one, a storm event occurring on the small stream; the other, a storm event applicable to the larger watershed.

Figure 3-2
Design Frequency Criteria

Functional Classification	Design Frequency (Years)	
	Bridges (Over 20 feet [6.1 m]) [2]	Pipes and Culverts[1]
Interstates, Freeways and Expressways	50	50
Principal Arterials and Minor Arterials	50	50
Major Collectors and Minor Collectors	50	50
Local Roads and Streets and Subdivision Streets	25	25

[1]Greater than 20 s.f. [1.8 m^2].
[2]Includes total out-to-out length of multiple pipe installations.
Note: Use a 25-year frequency for rural collectors.

Figure 3-3
Frequencies for Coincidental Occurrence

Area Ratio	25 Year Design		50 Year Design	
	Main Stream	Tributary	Main Stream	Tributary
10,000 to 1	2	25	2	50
	25	2	50	2
1,000 to 1	5	25	5	50
	25	5	50	5
100 to 1	10	25	10	50
	25	10	50	10
10 to1	10	25	25	50
	25	10	50	25
1 to 1	25	25	50	50
	25	25	50	50

Reference: Hydraulic Manual, Texas Department of Transportation.

In ordinary hydrologic circumstances, flood events on different watersheds are not usually entirely independent. Therefore, guidelines are needed to provide acceptable mixing criteria for independent waterways affected by separate storm events. To address this gray area, the table in Figure 3-3, based upon relative watershed sizes, is suggested as a guide to an appropriate assignment of event frequencies. The table, which is empirical and somewhat arbitrary, was devised by the Corps of Engineers, and is limited in scope. However, it does illustrate the concept described above. Note that, as the watershed sizes are more similar, the variance in the two frequency probability percentages diminishes.

For a given area ratio and design year, the designer should estimate the total flood discharge by adding the estimated discharges from the main stream and tributary based on the design year frequency. Two estimates must be made: one with the design storm applied to the main stream watershed and one with the tributary drainage area experiencing the design storm. The design should be based on the highest discharge.

The effects of tidal flows must be considered when the designer is evaluating the frequency mixing relationships. Refer to Section 3.3.5.

3.2.6 SPILLWAY DESIGN

Spillway design must take into consideration field survey data, drainage areas, reservoir capacity (from elevation and storage data), tidal influences, magnitude of peak in-flows for the design storm (considering frequency mixing), the Probable Maximum Flood, required freeboard below the top of the retention structure, water surface profiles, anticipated future development, and breach damage potentials. The significant range and nature of the influences that apply to normal hydrologic and hydraulic analyses also apply to spillway design. The designer is referred to various publications of the Army Corps of Engineers concerning spillway design requirements. See Section 3.7.

The design storm for spillway design shall be based on risk evaluation as described in Section 3.1.4. The design storm shall be approved by the Bridge Design Engineer. The minimum design storm for spillway design is the 100-year storm. Provisions should be made for drainage of the pond.

Normally, HEC-RAS (River Analysis System), HEC-1 (Flood Hydrograph Package) and HEC-HMS (Dam Breach Routine) are used by the designer. Critical to any spillway design are the breach analysis and Flood Damage estimates including economic losses and loss of life. Care must be taken not to affect existing pond water levels in the new design. Changes could have detrimental effects on adjacent properties.

3.3 HYDRAULICS

3.3.1 CULVERT HYDRAULICS

Culverts exhibit a wide range of flow patterns under varying discharges and tailwater elevations. To simplify the design process, two broad flow types are defined—inlet control and outlet control. A culvert operates with inlet control when the flow capacity is controlled at the entrance by the depth of headwater and the entrance geometry, including the barrel shape, the cross-sectional area, and the inlet edge. With inlet control, the roughness and length of the culvert barrel and outlet conditions are not factors in determining culvert hydraulic performance. Special entrance designs can improve hydraulic performance and result in a more efficient and economical structure. In outlet control, the culvert hydraulic performance is determined by the factors governing inlet control plus the controlling water surface elevation at the outlet and the slope, length and roughness of the culvert barrel. With outlet control, factors that may appreciably affect performance for a given culvert size and headwater are barrel length and roughness, and tailwater depth. Although entrance geometry is a factor, only minor improvement in performance can be achieved by modifications to the culvert inlet. For each type of control, the headwater elevation is computed using applicable hydraulic principles and coefficients, and the greater headwater elevation is adopted for the design.

The minimum freeboard for culverts is 1 foot [0.3 m] below the top of the roadway slope. Consideration should be given to the impact on the upstream properties.

Sometimes a natural bottom in culverts is required to facilitate the passage of fish. The designer should evaluate this need. The culvert may be lowered to allow siltation to provide a natural bottom. Only one barrel of a multi-barrel installation needs to be lowered. A three-sided rigid frame provides a natural bottom without the necessity of lowering the invert.

Single-barrel culvert designs are preferred. No more than three barrels should be constructed at a single location. Allow at least 3 feet [0.9 m] between pipe culverts on multi-pipe installations to allow room for compaction equipment.

Culvert hydraulic computations should follow the standard FHWA procedures for conventional culverts described in *HDS-5, Hydraulic Design of Highway Culverts*. A complete computer software program entitled HY-8, Culvert Analysis, is available, which applies the theories and principles of HDS-5. Refer to Section 3.3.2 for the use of HEC-RAS or WSPRO to establish water surface profiles. Information from this program is used to make freeboard calculations.

3.3.2 BRIDGE HYDRAULICS

Freeboard for a bridge is defined as the clear vertical distance between the water surface and the low point of the superstructure. The minimum freeboard is 1 foot [0.3 m]. In no case will the bearings be submerged during the design storm.

Two commonly used computer programs are generally accepted and used by DelDOT. These programs are used for modeling water surface profiles and for sizing the waterway opening of a bridge. Generally, DelDOT prefers the use of HEC-RAS over WSPRO because HEC-RAS is a more user-friendly program and provides graphical output. WSPRO is more exact for scour analysis; see Section 3.4.

3.3.2.1 HEC-RAS

HEC-RAS, River Analysis System, performs open channel analysis for steady or unsteady, one-dimensional, gradually varied flow in both natural and manmade river channels. Some HEC-RAS capabilities include the following:

- Simulates surface runoff into a river basin and the interconnected basins of hydrologic and hydraulic components.
- Model water surface profiles in both subcritical and supercritical flows around

various obstructions such as bridges, culverts, weirs, and structures in the floodplain.

- Incorporates rainfall-runoff analysis, river hydraulics, reservoir system simulation, and sedimentation analysis with a graphic user interface.

The designer should request that the appropriate cross sections be taken in the survey. Refer to Section 2.9.3.1.

3.3.2.2 WSPRO

WSPRO (for Water Surface Profile Computational Model HY-7) is a computer model capable of determining water-surface profiles through unconstricted stream reaches and through bridges. The program uses a standard step method to compute backwater and requires the description of a series of cross sections that divide the stream into short lengths allowing the assumption of gradually varying, steady flow within the section. Definition of the geometry and roughness of each section is needed. Roughness is defined by Manning's n-values. Expansion and contraction coefficients, friction loss, equations, and variable flow lengths for over bank and main channel areas for each section can be specified. The designer is referred to *FHWA Report No. RD-86/108*, *Bridge Waterways Analysis Model: Research Report*, and the *WSPRO Users Manual*.

The designer should request that the appropriate cross sections be taken in the survey. Refer to Section 2.9.3.1.

3.3.3 STREAM CHANNEL

The natural or altered condition of stream channels affects the flow characteristics. Any work being performed, proposed, or completed that modifies a stream channel changes the hydraulic efficiency of the stream and must be studied to determine its effect on the stream flow. The effect on peak flows at the structure site due to modification of a stream's hydraulic characteristics must be determined. The engineer should be aware of plans for channel modifications which might affect the stream hydraulics. Similarly, the effects of storm drainage systems and other water-related projects should be investigated. Any modifications that affect stream alignment should be kept to a minimum, particularly for straightening meandering streams.

3.3.4 ICE AND DEBRIS

The quantity and size of ice and debris carried by a stream should be investigated and recorded for use in the design of drainage structures. The times of occurrence of ice or debris in relation to the occurrence of flood peaks should be determined, and the effect of backwater from ice or debris jams or recorded flood heights should be considered in using stream-flow records. The location of the constriction or other obstacle-causing jams, whether at the site or structure under study or downstream, should be investigated, and the feasibility of correcting the problem should be considered.

Under normal circumstances, one foot [0.3 m] freeboard is sufficient to permit passage of ice flow and debris. At locations where large pieces or quantities of debris are anticipated, the designer should consider increasing the freeboard.

3.3.5 TIDAL HYDRAULICS-BRIDGES AND CULVERTS

At this time, there is no single authoritative reference for guiding the engineer in modeling the effect of tidal flows on the hydraulics of a structure. There are several circumstances in which the potential for tidal impacts is significant.

The size of the bridge opening may be controlled in a case of incoming (flood) tidal flows and peak storm discharge. Another consideration that may control the size of the opening is the storm surge at peak flood tidal flows. In the same manner, scour of the stream bottom is a concern on outgoing (ebb) tidal flows and peak storm discharges.

These and other combinations of tidal and storm flows must be considered in the sizing and design of a structure.

Refer to the following models, studies and reports as appropriate:

- any of the various tidal models for Chesapeake and Delaware Bays in combination with the non-tidal flow calculated above to produce the maximum flood which does not overtop the roadway or structure;
- existing FEMA studies;
- existing Coastal Engineering Research Center reports;
- tidal prism method given in HEC-18; or
- FESWMS (Finite-Element Surface-Water Modeling System).

3.3.6 BRIDGE DECK DRAINAGE

Bridge deck drainage systems direct runoff off the bridge and away from the traveled lanes. Scuppers or complex piping systems to carry roadway runoff are not always required. When not properly designed or routed, ancillary problems such as poor visibility, icing, and corrosion of structural members can result. Use the rational method to determine the runoff.

Bridge deck designs should conform with roadway standards for crown cross slope and superelevation. These standards are as follows:

- crown cross slope: 2 percent, and
- superelevation: 6 percent maximum, but 8 percent maximum for rural roadways.

Refer to the FHWA publication *HEC-21, Bridge Deck Drainage Systems*. This utilizes design nomographs based on grade, cross slope, design speed, design rainfall intensity, and bridge deck width, to determine an estimate of the bridge length without scuppers. Because bridge deck drainage systems can cause maintenance problems, their use should be avoided or at least minimized. Once it is determined that scuppers are required, additional criteria for scupper type, spacing, and location are provided.

Because bridges freeze before roadways, the allowable spread of water adjacent to the curb or parapet should be restricted to the width of the shoulder. On narrow bridges, with little or no shoulder, a minimum of 8 feet [2.4 m] of travel lane must be maintained in each direction.

The ten-year storm should be used as a minimum for design of deck drainage. Refer to *FHWA-SA-92-010, Design of Bridge Deck Drainage*.

Locating a bridge in a sag curve or in a flat grade less than 0.5 percent should be avoided to the extent possible. If such location cannot be avoided, the scuppers should be placed at the low point and, if necessary, one on each side of the low point where the grade elevation is 2.5 inches [60 mm] higher than the low point. The capacity of the scupper located at the low point should assume 50 percent efficiency, to allow for the possibility of clogging. All scuppers in the sag should be of the same capacity. Additional scuppers must be added to provide adequate drainage based on the design year storm in accordance with the above-stated spread restrictions. Provisions should be made to collect water to prevent it from running on the bridge.

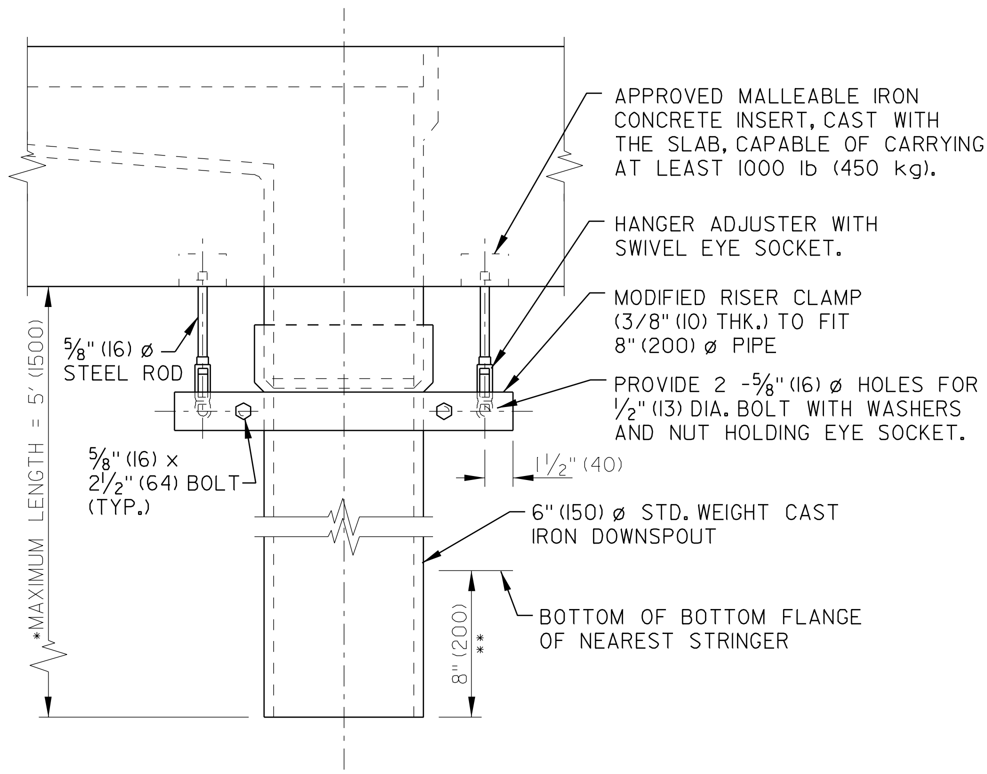
Several significant considerations should be made to provide the best possible design:

- Adequate longitudinal slope to move water off the bridge should be provided.
- All free-falling scupper outlet pipes should extend a minimum of 8 inches [200 mm] below the nearest superstructure element within 10 feet [3 m] of the scupper. Aesthetics should be considered in this.
- The drainage from a scupper outfall pipe should not be allowed to freefall to the ground if in so doing other deleterious effects (i.e. erosion, icy conditions, etc.) can result.
- Scupper downspout pipes, where needed, should always be attached to the exterior surface of the substructure and never placed within the concrete substructure elements. Water may freeze in the pipes and break the concrete. Refer to Figures 3-4 and 3-5.

3.3.7 TAX DITCHES

The Soil Conservation Service (SCS) builds and maintains ditches in poorly drained areas to drain fields and cultivated areas to improve the ability to farm these areas. These "tax ditches," are located in Kent and Sussex Counties. When designing structures over waterways that may be tax ditches, the designer should contact SCS to determine whether a tax ditch is involved and, if so, the requirements to be met. The design should be checked to ensure that the footings for the proposed structure are located below the dredge line and beyond the limits of the tax ditch.

Figure 3-4
Scupper Downspout Support Bracket Elevation



* WHEN MAXIMUM LENGTH IS EXCEEDED AN ADDITIONAL SUPPORT BRACKET IS NECESSARY AT LOWER PORTION OF PIPE.

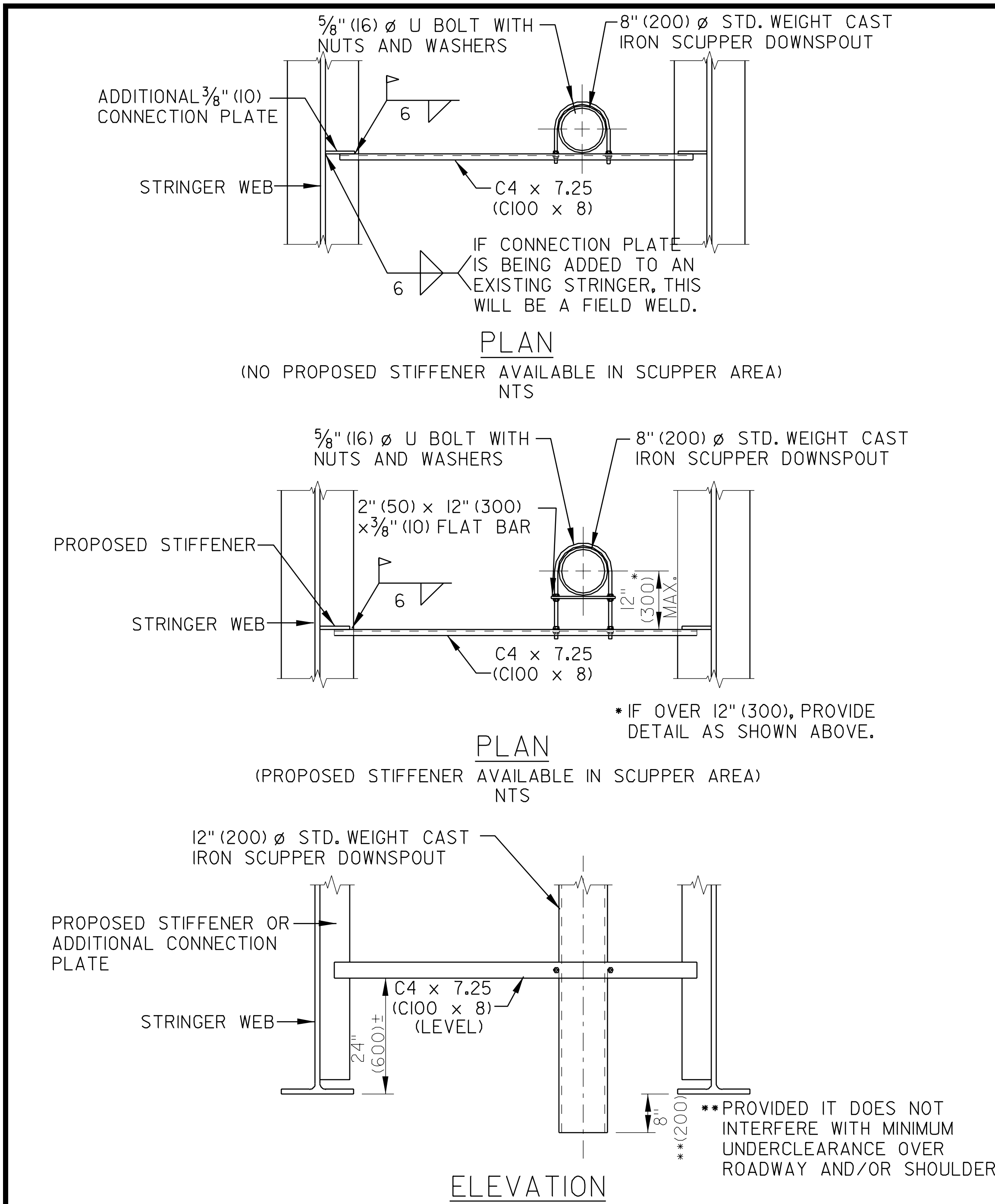
** PROVIDED IT DOES NOT INTERFERE WITH MINIMUM UNDERCLEARANCE OVER ROADWAY AND/OR SHOULDER.

NOTES:
1. ALL EXPOSED MATERIAL, EXCEPT CAST IRON DOWNSPOUT, TO BE EPOXY COATED AS INDICATED FOR EPOXY COATED REBAR.
2. EXPOSED PORTIONS OF CAST IRON DOWNSPOUT TO BE COATED WITH COAL TAR EXPOXY CONFORMING TO SPECIFICATION SSPC-16.

ELEVATION

NTS

Figure 3-5
Scupper Downspout Support Bracket Details



NOTES:

1. U-BOLTS, NUTS, WASHERS, BARS, CHANNELS, ETC. SHALL RECEIVE THE SAME NUMBER OF COATS AND SAME COLOR FIELD PAINT AS REQUIRED FOR STRUCTURAL STEEL. IF WEATHERING STEEL IS UTILIZED FOR ANY ELEMENTS, IT NEED NOT BE PAINTED.
2. EXPOSED PORTIONS OF CAST IRON DOWNSPOUT TO BE COATED WITH COAL TAR EPOXY CONFORMING TO SPECIFICATION SSPC-16.

3.4 SCOUR EVALUATION AND PROTECTION

Changes in the bed level of a stream affect highway structures and may be described by three types of actions: (1) general scour (contraction scour), (2) local scour, and (3) degradation or aggradation of the stream channel. Scour and degradation are discussed in this section. Other types of erosion and aggradation are discussed in Section 3.5.

Every bridge over a waterway should be evaluated as to its vulnerability to scour in order to determine the appropriate protective measures. Most waterways can be expected to experience scour over a bridge's service life (which could approach 100 years). The need to ensure public safety and to minimize the adverse effects stemming from bridge closures requires the best effort to improve the state-of-practice of designing and maintaining bridge foundations to resist the effects of scour. Current information on this subject has been assembled in *HEC-18*, *Evaluating Scour at Bridges*.

Scour evaluations of new and existing bridges should be conducted by an interdisciplinary team composed of hydraulic, geotechnical, and structural engineers.

Bridges over waterways—both tidal and non-tidal—with scourable beds should withstand the effects of scour from a superflood (500-year flood) without failing.

Hydraulic studies shall include estimates of scour at bridge piers and evaluation of abutment stability. Bridge foundations shall be designed to withstand the effects of scour for the worst conditions resulting from floods. The geotechnical analysis of bridge foundations shall be performed on the basis that all streambed material in the scour prism above the total scour line for the designated flood (for scour) has been removed.

For the design flood, the stability of the bridge foundation shall be investigated using the service and strength limit states. The design flood for scour shall be the more severe of the 100-year event or from an overtopping event of smaller recurrence interval.

For the Q_{500} super flood conditions, the foundation shall be designed to be stable for the extreme event limit state. The super flood for scour shall be the more severe of the 500-year event or from an overtopping event of smaller recurrence interval.

In general, foundations shall be designed to be stable without relying on scour countermeasures. The only exception to this is when designing for local scour at abutments. Because the local scour equations tend to overestimate the magnitude of scour at abutments, they are generally used only to gain insight into the scour potential at an abutment. In most cases, a scour countermeasure, properly designed and installed in accordance with the procedures outlined in HEC-23, is provided to resist the local scour at abutments. Both the abutment foundation and the scour countermeasure must be designed to be stable after the effects of the estimated long-term degradation and contraction scour. Ensure that the top of the footing is below the sum of the long-term degradation, contraction scour, and lateral migration; stub abutments are an exception to this requirement, but the slopes in front of them should be adequately protected and/or sheeting should be provided to prevent undermining of the abutment and loss of fill. Riprap (minimum size R-5) must always be used to protect abutments from erosion for maintenance purposes, even if it is not required to resist the effects of local scour.

The *AASHTO Specifications* contain requirements for designing bridges to resist scour. Particular attention is directed to Sections 2.6.4.4.2 and 3.7.5.

3.4.1 EVALUATION CRITERIA

3.4.1.1 Analysis Procedure

Scour analysis should be performed according to the FHWA publication *HEC-18, Evaluating Scour at Bridges*. Computer software HY-9, Scour at Bridges, should be used to check the manual calculations. Any countermeasures required should be designed using the methods in:

- *HEC-18, Evaluating Scour at Bridges*,
- *HEC-11, Design of Riprap Revetment*,
- *FHWA-HI-90-016, Highways in the River Environment*,
- *HEC-20, Stream Stability at Highway Structures*, or
- *HEC-23, Bridge Scour and Stream Instability Countermeasures*.

Minimum riprap size must conform with the requirements of R-5 in Section 712 of the Standard Specifications. Larger riprap may be specified if it is needed. The riprap in the channel shall be covered with a minimum of one foot of natural stream bed channel. A low-flow channel shall be formed at that point, if applicable.

Also refer to *NCHRP Report 587, Countermeasures to Protect Bridge Abutments from Scour*.

3.4.1.2 Scourability of Rock

Evaluate the scour potential of rock by following the procedure for rock quality designation (RQD) in FHWA Mid-Atlantic Region Memorandum, Scourability of Rock Formations, to determine scourability. The following criteria represent the values to define rock quality and scourability of rock:

- The RQD value is a modified computation of the percent of rock core recovery that reflects the relative frequency of discontinuities and the compressibility of the rock mass and may indirectly be used as a measure of scourability. The RQD is determined by measuring and summing all the pieces of sound rock 6 inches [150 mm] and longer in a core run and dividing this by the total core run length. The RQD should be computed using NX diameter cores or larger and on samples from double tube core barrels. Scourability potential will increase as the quality of rock becomes poorer. Rock with an RQD value of less than 50 percent should be assumed to be soil-like with regard to scour potential.
- The primary intact rock property for foundation design is unconfined compressive strength (ASTM Test D2938). Although the strength of jointed rocks is generally less than individual units of the rock mass, the unconfined compressive strength provides an upper limit of the rock mass bearing capacity and an index value for rock classification. In general, samples with unconfined compressive strength below 250 psi [1724 kPa] are not considered to behave as rock. There is only a generalized correlation between unconfined compressive strength and scourability.
- The slake durability index (SDI as defined by the International Society of Rock Mechanics) is a test used on metamorphic and sedimentary rocks such as slate and shale. An SDI value of less than 90 indicates poor rock quality. The lower the value, the more scourable and less durable the rock.

- AASHTO Test T104 is a laboratory test for soundness of rock. A soaking procedure in magnesium and sodium sulfate solution is used. Generally the less sound the rock, the more scourable it will be. Threshold loss rates of 12 (sodium) and 18 (magnesium) can be used as an indirect measure of scour potential.
- The Los Angeles abrasion test (AASHTO T96) is an empirical test to assess abrasion of aggregates. In general, the less a material abrades during this test, the less it will scour. Loss percentages greater than 40 percent indicate scourable rock.

The other methods described in that memorandum should be used if required. For other soil types, existing surface borings and tests of soil samples should be interpreted.

3.4.1.3 Scour Evaluation Report

The scour evaluation report must contain the following items:

- table of contents;
- bridge description—bridge number, type, size, location, and National Bridge Inventory Record Item 113, Scour coding;
- executive summary of hydrologic and hydraulic methods, scour results, conclusions, and any countermeasure recommendations required, with plan and profile views showing scour depths and limits;
- scour computations (including computer input and output);
- bridge drawings, cross sections, soils information, test results, and other miscellaneous data; and
- references.

3.4.1.4 Plan Presentation

The following information will be provided in the Project Notes on the plans:

- a note stating that the structure has been analyzed for the effects of scour in accordance with the procedures described in *HEC-18, Evaluating Scour at Bridges*;
- scour analysis design flow volume, frequency, velocity, and water surface elevation;
- scour analysis check flow volume, frequency, velocity, and water surface evaluation;
- the calculated design scour depth; and
- the calculated check scour depth.

See Figure 3-6 for a sample scour project note.

Figure 3-6
Sample Scour Project Note

THE PROPOSED STRUCTURE HAS BEEN ANALYZED FOR THE EFFECTS OF SCOUR IN ACCORDANCE WITH HEC-18 - 'EVALUATING SCOUR AT BRIDGES' AND HEC-23 - 'BRIDGE SCOUR AND STREAM INSTABILITY COUNTERMEASURES.' SCOUR COUNTERMEASURES HAVE BEEN DESIGNED FOR THE WORST CASE OF THE OVERTOPPING FLOOD OR THE 500-YR FLOOD EVENT.

DESIGN EVENT	OVERTOPPING	DESIGN VELOCITY	6.22 FT/S
DESIGN DISCHARGE	535 CFS	DESIGN DEPTH OF FLOW	6.14 FT

3.5 STREAM STABILITY

3.5.1 STREAM STABILITY ANALYSIS

Erosion is considered to be the loss of material on side slopes and stream banks. Types of stream erosion include:

- scour (see Section 3.4);
- the natural tendency of streams to meander within the flood plain;
- bank erosion; and
- degradation.

These are all interrelated to some degree.

The computed velocity is a measure of the potential erosion and scour. Exit velocity from culverts will be computed on the assumptions shown in *HDS-5, Hydraulic Design of Highway Culverts*. (Use HY-8, Culvert Analysis, software based on HDS-5 for the computations.) Average velocity computed on the gross waterway will be the representative velocity for open span structures, furnished by computer analysis for water surface elevations.

Examples of highly erodible soil can be found in all areas of the state. Areas of loamy deposits, which are highly sensitive to erosion, are prevalent in Delaware. County SCS soil maps may aid in judging the in-situ material.

The designer must consider the downstream erosion potential in evaluating and sizing the structure. Under some conditions, any additional erosion would be intolerable. Thus, risk considerations should be included in the site study. It should be recognized that stream banks erode regardless of the presence of a highway crossing. Any alteration of erosion potential by a structure must be closely evaluated in judging the adequacy of a design.

Streams naturally tend to seek their own gradient through either degradation or aggradation. Degradation is the erosion of streambed material, which lowers the streambed. Aggradation is the transport and deposition of the eroded material to change the streambed at another location. The effect of the structure on degradation or aggradation of a stream must be evaluated in bridge crossing design.

The designer should evaluate the stability of the bed and banks of the waterway channel, including lateral movement, aggradation, and degradation, using *HEC-20, Stream Stability at Highway Structures*.

3.5.2 BANK PROTECTION

The most common method of bank protection is the use of rock riprap. Factors to consider in the design of rock riprap protection include:

- the stream velocity,
- the angle of the side slopes, and
- the size of the rock.

Filter blankets of smaller gradation bedding stone or geotextiles are used under riprap to stabilize the subsoil and prevent piping damage. Riprap bank protection should terminate with a flexible cut-off wall.

The designer should specify a minimum blanket thickness of 18 inches [460 mm] for embankment protection and 24 inches [610 mm] for slope protection along stream banks and for streambeds. Refer to *FHWA-HI-90-016, Highways in River Environment*, and *HEC-11, Design of Riprap Revetment*. See Figure 3-7 for typical riprap details and an example of a riprap installation.

3.5.3 CHANNEL MODIFICATIONS

A channel change is the physical relocation of the streambed channel. A channel improvement is the clearing and dressing of stream bank and overbank slopes, or excavation of overbank and stream bank areas but with no change of the streambed profile.

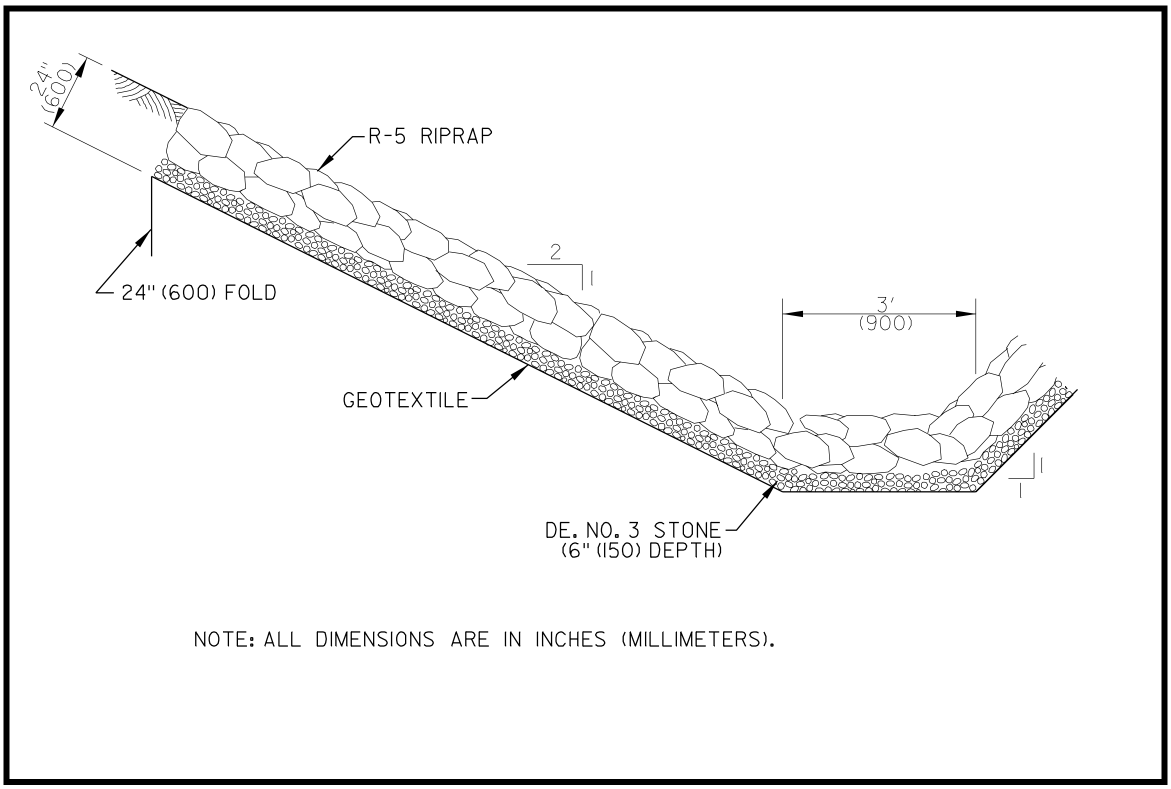
The primary objective in the design of a highway stream crossing is to avoid interruption in the behavior of the stream. Channel modifications should be made only where necessary to accommodate streamflow, and where regular maintenance can maintain the improvements or no maintenance is needed.

The preferred procedure for dealing with channel changes is:

- establish the nature of the existing stream (slope, section, meander pattern, stage-discharge relationship),
- determine limits for changes in the various stream parameters,
- duplicate existing conditions where possible, within established change tolerances, and
- evaluate constructibility, considering water table elevations, streambed materials, and site conditions.

For more guidance, refer to AASHTO's *Highway Drainage* and *FHWA-HI-90-016, Highways in the River Environment*.

Figure 3-7
Typical Riprap Detail



3.6 PLAN PRESENTATION

The following hydrological and hydraulic information is required on the plans of structures over streams and should be included in the Project Notes on the General Notes sheet.

HYDRAULIC DATA

Drainage Area sq. miles [km^2]
Design Frequency years
25 yr Flood Elevation ft [m]
Design Discharge cfs [m^3/s]
Proposed Opening sf [m^2]

Other information that is desirable and is required in the Hydraulic Report, as directed by the Bridge Design Engineer, includes the following:

Overtopping Elevation (Locat.) ft [m]
Overtopping Discharge ft^3/s [m^3/s]
Overtopping Frequency yr.
Overtopping Velocity (Based on Overtopping Discharge) ft/s [m/s] [Note: Overtopping is permitted only if qualified by a risk analysis.]

Design Backwater ft [m]
Design Backwater Elevation ft [m]
Discharge at Q_{100} ft^3/s [m^3/s]
Backwater at Q_{100} ft [m]
Backwater Elevation at Q_{100} ft [m]

Velocity at Q_{100} ft/s [m/s]
Historic Highwater Elevation ft [m]
Ordinary Highwater Elevation ft [m]
Total Waterway Provided ft^2 [m^2]
Design Waterway Provided ft^2 [m^2]

FEMA 100-Yr. Flood plain Elevation (Regulatory) ft [m]
FEMA 100-Yr. Floodway Elevation (Regulatory) ft [m]
Average Velocity at Q_{des} ft/s [m/s]
Average Velocity at Q_{xx} ft/s [m/s]

For tidal areas include the following:

Mean High Water Elevation ft [m]
Mean Low Water Elevation ft [m]
Vertical Under Clearance ft [m]

Refer to Section 3.4.1.4 for plan presentation of scour analysis data. Additional site-specific information may be required and noted on the plans as determined by the Bridge Design Engineer. Descriptions of terms for hydraulic data follow. See Figure 3-8 for a graphical depiction of the definitions.

- Documentation of **Historic High Water** includes year(s) of occurrence and source of information.
- **Ordinary High Water** is required information for the "404" permit. From instructions and definitions furnished by the Corps of Engineers for "404" permit applications, the Ordinary High Water mark means the line on the shore established by the fluctuation of water and indicated by physical characteristics such as a clear, natural line impressed on the bank, shelving, changes in the character of soil, destruction of terrestrial vegetation, the presence of litter and debris, or other appropriate means that consider the characteristics of the surrounding areas. Ordinary High Water will usually be established by the field survey of the site. Where Ordinary High Water is not determined by survey of the physical characteristics, it may be estimated by computation of the normal water surface elevations at the 2-year frequency (Q_2).
- The **Design Discharge** as computed by the methods noted in this manual. When

other methods are applicable and are used to compute the Design Discharge, it should be noted in the hydraulic report.

- **Design Headwater**: As a conservative estimate of the headwater for design, the elevation of the water surface under unrestricted conditions at the upstream face of the bridge is used to compute clearance. It is the assumed condition where the water surface profile is computed at the design discharge (Q_{des}) with gradually varied flow. This computed highwater elevation should always be compared to the highwater elevation of record as furnished by the field survey to determine whether additional grade adjustment should be made for the extreme condition.
- **Clearance** for debris or other purposes is measured from the Design High Water Surface at the bridge and the low point of the superstructure and should be noted on the plans in the Elevation view of the Plan and Profile sheet as: "Design HW Clear.= xx.x ft [m]."
- The **Average Velocity** is computed from the gross area at the bridge opening below Design flow depth, i.e., Q/A_n, where A_n is the gross waterway area in the constriction at Design High Water depth. Design Waterway Provided is the net flow area below the Design High Water elevation. Total Waterway Provided is the net flow area below the bridge. Total Waterway and Design Waterway will be the net flow area (i.e., deduct pier area). Where the stream approach is skewed, all waterway areas should be measured normal to the stream flow, i.e., corrected by the bridge length times the cosine of the skew angle. The projected area of the piers should likewise be corrected. The plans should indicate that the waterway areas are normal to stream flow when corrected for a skewed approach.
- **Design Backwater Elevation**: For convenience, the amount of design backwater is measured as shown on the profile section in Figure 3-8, for the computed design discharge (Q_{des}). Although this may not be the exact location of the maximum high water, it is accurate enough to provide a reasonable estimate. For critical locations where the exact backwater computation might affect the design, i.e., where a FEMA floodway exists, the designer should refer to the methods given in Chapter IV of *HY-1, Hydraulics of Bridge Waterways*.
- The location of the **Overtopping Elevation** for the bridge and approaches may be referred by stationing (e.g. Station 6+95.7) or by distance from the bridge (e.g. 375 feet [115 m] south of bridge abutment No. 1). The location of the overtopping may occur on the bridge. The overtopping roadway elevation may be either the centerline elevation or the high shoulder elevation in a superelevated section.
- **Freeboard**, as applied to bridge hydraulics, is the vertical distance from the design headwater elevation to the low point of the superstructure. This distance is recorded on the Hydraulic Assessment Checklist (Figure 3-1). Where the design headwater elevation is higher than the low point of the superstructure, there is no freeboard.

For culverts, the design headwater elevation is 1 foot [0.3 m] below the top of the slope to prevent overtopping.

Figure 3-8
Typical Section Along Stream