SCDOT

South Carolina Department of Transportation

REQUIREMENTS FOR HYDRAULIC DESIGN STUDIES

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LIST OF TABLES, FIGURES, AND FORMS

SOUTH CAROLINA DEPARTMENT OF TRANSPORTATION REQUIREMENTS FOR HYDRAULIC DESIGN STUDIES

Introduction

This document describes the requirements for hydraulic design studies performed for South Carolina Department of Transportation (SCDOT). Its purpose is to guide Departmental staff and consultants performing design work for the Department in the preparation of construction plans. Before hydraulic design engineers perform a design study using these requirements, they should be thoroughly familiar with the more detailed coverage of the different design procedures given in the manuals in the reference list. Also, this manual may continue to be revised; therefore, the user should insure that the latest revision is being used. Compliance with these procedures will help insure the requirements in the following laws and regulations will be adhered to:

- Federal Highway Administration's Federal-Aid Policy Guide, 23 CFR 650A, December 1, 1991, Transmittal 1, "Bridges, Structures, and Hydraulics"
- The Federal Emergency Management Agency Regulations, 44 CFR Part 65
- The Environmental Protection Agency's (EPA) Regulations 40 CFR Parts 9, 122, 123, and 124 National Pollution Discharge Elimination System (NPDES) as administered under general permit by South Carolina Department of Health and Environmental Control (DHEC)
- The State Stormwater and Sediment and Erosion Control regulations administered by DHEC, 26 S. C. Code Ann. Regs. 72-405 (Supp.1995) et seq.
- Departmental policy
- South Carolina State Water Law

The requirements are presented in two parts. Part 1 is for bridges and bridge-sized culverts (single or multiple culverts with an opening greater than or equal to 20 feet along the highway centerline). In Part 1, references to bridges include bridge-sized culverts unless it is obvious that a bridge structure is being addressed or the information is directed specifically to culverts. Part 2 is for roadway drainage and culverts less than 20 feet wide. Part 2 also includes the guidelines established to insure compliance with the NPDES permit requirements and the State Stormwater Management and Sediment and Erosion Control Act.

A formal study report with a title sheet and an index page will be prepared and both signed and sealed by a registered professional engineer of the State of South Carolina.

The title and date of the edition of the *Requirements for Hydraulic Design Studies,* used for design purposes, shall be shown on the title sheet of plans and included on the cover page of the "Hydraulic Design Study Report."

All forms contained in this document are also located on the SCDOT Intranet/Internet pages. The forms on the Intranet/Internet are updated regularly and take precedence over those in this document.

PART 1: REQUIREMENTS FOR HYDRAULIC DESIGN OF BRIDGES AND BRIDGE-SIZED CULVERTS

1. Analysis Procedures

Bridges and bridge-sized culverts (20 feet or greater as measured along the road centerline) shall be designed by the three-level analysis procedure described in HEC-20 "Stream Stability at Highway Structures." Bridges in the Fast Track Bridge Program require a Level 1 analysis. Level 1 and Level 2 analyses should apply to all designed bridges. Level 3 will apply to those cases where one-dimensional flow analysis is not adequate. Examples where a Level 3 analysis is needed are:

- Wide floodplains with multiple bridges, where curvilinear flow is a major factor
- Any floodplain where curvilinear flow has a major impact on the bridge
- Major tidal estuaries
- In situations where sediment routing is necessary

The Risk Assessment (Section 1.6.4) must be completed when a Level 2 or 3 analysis is required.

The HEC-20 study approach is modified in this document to incorporate SCDOT bridge design requirements and procedures.

Bridges in South Carolina are designed for both riverine and tidal stream crossings. Riverine bridges are designed for steady flow at the peak discharge for the design storm. Hydraulic design for riverine bridges establishes:

- Minimum finished grades
- Bridge location
- Bridge length
- Span configurations
- Substructure type and orientation
- Foundation requirements through scour analysis

Tidal bridges are designed for unsteady flow conditions during the complete rise and fall cycle of a hurricane tidal surge. Hydraulic design for tidal bridges establishes the minimum finished grade and minimum depth requirements for the foundation through scour analysis. For sites further upstream, riverine flow becomes dominant. In some cases both, riverine and tidal flow, must be analyzed to determine the controlling flow.

Each bridge or culvert design study or scour study will be documented following the outline of HEC-20 procedures as delineated below. The documentation will consist of a report, which will follow the outline of the Level 1, 2, or 3 analysis discussed later in this document. It will also include a title sheet (Section 1.6.1), an index, a comparative data sheet (Section 1.6.2), location maps, and site inspection forms (Section 1.6.3). When a Level 2 or 3 analysis is required, then a risk assessment (Section 1.6.4) is also required. In accordance with South Carolina Code of Laws Title 40, Chapter 22 and Code of Regulations Chapter 49, the title sheet for the “Hydraulic

Design Study Report" will be signed and sealed by a registered professional engineer of the State of South Carolina.

1.1 Design Criteria

1.1.1 Design Frequencies.

The design discharge for establishing bridge location and bridge geometry for secondary roads is the 25-year discharge. For primary and interstate routes, the design discharge is the 50-year discharge. All stream crossings are to be analyzed for the 100-year flood to insure that one (1) foot or less of backwater is caused by the proposed bridge when compared to unrestricted or natural conditions.

1.1.2 Floodway - Floodplain Requirements.

All floodplain crossings must meet the Federal Emergency Management Agency (FEMA) regulation requirements. FEMA should be contacted to determine if a Conditional Letter of Map Revision (CLOMR) or Letter of Map Revision (LOMR) has been issued for the subject stream. FEMA currently maintains a list of CLOMRs and LOMRs on their website.

If the stream has a designated floodway, the structure should be designed, if practical, so there will be no change in the 100-year flood elevations, floodway elevations, and floodway widths at any cross section. SCDOT considers a project to be a "No Impact" if there is no change in the 100-year profile or the floodway profile, rounded to the nearest 0.1 foot, or floodway width, rounded to the nearest 1.0 foot, for any cross section outside the Department's right of way. Changes inside the SCDOT right of way are considered internal to the bridge structure and do not affect any property. If these criteria are met, two original copies of the "No Impact" certificate should be prepared with two sets of supporting documentation. The Department's hydraulic design engineer will send one copy to the local community and one copy will be retained in the hydraulic engineering design files.

If there is a change, greater than rounded to the nearest 0.1 foot, in the 100-year or floodway profile or a change in floodway width, greater than rounded to the nearest 1 foot outside the Department's right of way, a Conditional Letter of Map Revision (CLOMR) must be prepared using all the appropriate forms. The elevation and distance discussed above should be rounded to the nearest 0.1 foot and 1 foot respectively. One copy of the CLOMR will be sent to the local community with two additional copies of the community sign-off sheet. The local community will be asked to sign the two copies of the community sign-off sheet and return them to the Department. All property owners affected by the changes in flood profile or floodway width must be identified and their addresses determined. A letter to each of the property owners will be prepared detailing the changes in flood profile and/or width affecting the owner's property. The letters will be sent by certified mail. Flood easements will have to be purchased for properties affected by increases in the flood profile and/or in floodway width. When the receipts for the certified letters are received, a copy of the CLOMR including the signed community sign-off sheet, a check for FEMA's review fee, and a copy of the certified mail receipts are to be submitted to FEMA. The other copy of the CLOMR, along with copies of the supporting documentation listed above, will be filed in the hydraulic engineering design files. When the "as built" final construction plans are complete, three copies of a Letter of Map Revision (LOMR) will be prepared. The LOMR will include a certified, signed, and sealed copy of the "as built" plans; a letter of certification by the resident construction engineer that the project was built according to the plans; and the appropriate forms including a community sign-off sheet. If the project was built to meet the hydraulic conditions proposed in the CLOMR, no hydraulic analysis is necessary. However, if those conditions have been changed, a new hydraulic analysis must be submitted and notification sent via certified mail to owners of affected property.

It is the Department's policy to limit the increase to 1.0 foot or less above the unrestricted or natural 100-year flood profile. If this policy can not be met, a request for a design exception will be required.

1.1.3 Flow Velocities.
Flow velocities within the bridge opening should be limited so there will be minimum scour in the overbank portion of the opening.

1.1.4 Bridge Scour.
Scour analysis will be performed for all bridge type structures, utilizing USGS envelope curves and methods found in HEC-18. Bridge foundations should be designed to withstand the effects of scour without failing for the worst conditions resulting from floods equal to the 100-year flood, or a smaller flood if it will cause scour depths deeper than the 100-year flood. Bridge foundations should be checked to ensure that they will not fail due to scour resulting from the occurrence of a superflood in order of magnitude of a 500-year flood. This requires careful evaluation of the hydraulic, structural, and geotechnical aspects of bridge foundation design.

The 500-year discharge can be determined using the USGS regression equations for South Carolina. If other means are unavailable, then the 500-year discharge can be estimated as 1.7 times the magnitude of the 100-year flood.

1.1.5 Design Freeboard.
All bridges will be designed with a clearance, called the freeboard, above the design high water. The freeboard has two purposes: to protect the structure from damage from debris and to protect the bearings and beam seats from the corrosive effects of water.

1.1.5.1 Freeboard for Riverine Bridges.
It is the Department's policy to provide a minimum freeboard of 2.0 feet above the design high water for all riverine bridges. The freeboard may be increased to a maximum of 7.0 feet for larger rivers such as the Congaree, Great Pee Dee, Santee, and Wateree. The freeboard is based on the potential size of drift and debris on the stream during the design flood. The hydraulic design engineer should evaluate the debris load potential and history of debris accumulation. Using judgment, the hydraulic design engineer should select the appropriate freeboard to allow debris to safely pass under the bridge superstructure without collecting on the bridge.

1.1.5.2 Freeboard for Tidal Bridges.
Bridges on tidal streams will be designed to protect the bridge structure itself. Most of the surrounding land and the approach roadways will be inundated by relatively frequent (10- to 25-year) tidal storm surges. The recommended design freeboard for bridges in these areas is 2.0 feet above the 10-year high-water elevation including wave height. It is also recommended to have the bottom of all interior bent cap elevations above the extreme high tide. The finished grade of the bridge will be set by considering this recommendation, navigation clearances, the approach roadways, topography, and practical engineering judgment.

1.1.5.3 Freeboard for Bridges over Lakes and Reservoirs.
If the bridge is over one of the major lakes or reservoirs where there is boat traffic, the grade should be set so that there is a minimum of 8.0 feet of freeboard above the maximum operating pool.

1.1.5.4 High-water Data.
High-water data shall be computed and supplied to the road designer for use in determining an appropriate finished grade elevation.

1.1.6 Bridge Abutment Protection.
Riprap, or equivalent, will be placed on all bridge end fills per SCDOT standard drawing. The riprap shall be entrenched 2.0 feet below the ground line and extend to 2.0 feet above the design high-water level. Minimum riprap thickness will be calculated as two (2) times the design D_{50}.

1.1.7 Guide Banks.
In wide floodplains where the approach flow outside the bridge is significant, guide banks (spur dikes) will be considered. The two-dimensional computer program FESWMS-Flo2DH should be used to evaluate the need for guide banks per HEC-23.

1.1.8 Bridge-Sized Culverts.
For culverts, determine if the outlet velocity will cause scour of the channel bed or banks. If scour is predicted, outlet protection should be used. The scour protection should be designed using FHWA's HEC-14. Bridge-sized culverts should be designed to meet the bridge hydraulic design criteria.

1.2 Level 1: Qualitative and Geomorphic Analysis
The qualitative and geomorphic analysis is the first step in a logical progression of analysis in bridge design that moves from the general descriptive to the detailed quantitative design. This approach first defines the design problem and then evaluates the stream and its geomorphic responses over time. It evaluates qualitatively the possible stream responses to the proposed or existing highway structures. The design procedure for the Fast Track Bridge Replacement Program uses a modified Level 1 approach (see Section 1.2.2).

1.2.1 Level 1 Procedure.

The standard Level 1 procedure is done in a series of steps. A flow diagram of the procedure is presented in Figure No. 1, Section 1.6.4. (See Chapter 3 of HEC-20 for full details on this procedure.) Determine if the stream contains a designated floodway. If it does, obtain a copy of the original computer program data files used to establish the floodway boundaries. If a floodway is present, go to Level 2 analysis.

Step 1 Stream Characteristics

Describe the stream geomorphic characteristics using the form in Figure No. 3, Section 1.6.4, taken from HEC-20. For a complete discussion of stream geomorphic characteristics, see Chapter 2 of HEC-20. The data for accomplishing this step can be obtained from aerial photography (available in the SCDNR Mapping Section), USGS quad maps, SCS County Soil Books, and a site visit. If meander bends are located near the proposed bridge, historic aerial photographs and maps should be studied to determine the rate of change associated with the bend. The Map Room in the Cooper Library at the University of South Carolina contains aerial photographs of years past. It should be noted that the term "swamp" has been added to the stream size section of the form. Many swamps in South Carolina have no recognizable channel but carry flows requiring a bridge. The lack of a channel is due to flat slopes and thick vegetation on the floodplains. A scour hole resembling a channel will usually form at a bridge site in a swamp; however, a more detailed investigation will reveal that the scour hole will only extend roughly 100 feet upstream and 200 to 300 feet downstream from the bridge.

Step 2 Land Use Changes

Evaluate land use and land use changes. Land use affects both the runoff and the sediment supply to a stream channel. Understanding the history of land use in a drainage area and having an estimate of future changes in land use will give an understanding of the stability of the channel and of changes in runoff. Local planning directors are the best source of information concerning future land use development plans. The historic land usage can be determined from historic aerial photographs (see Step 1).

A decrease in the vegetative cover will increase both the storm runoff and sediment yield. Increasing the impervious cover will increase the rainfall runoff, but may decrease the sediment yield. An increase in sediment supply will cause aggradation, while a decrease in sediment supply will cause degradation. An increase in a stream's flow will increase the sediment carrying capacity of the stream, causing stream instability and degradation. Reference 14 and HEC-20 include in-depth discussions on stream responses to changes in sediment supply.

Step 3 Overall Stability

Assess overall stream stability. Use Table 3.2 and Figure 3.2 in HEC-20 (2001) to assess overall stream stability based on the results of steps 1 and 2.

Step 4 Lateral Stability

The shifting of a meander bend or a channel bank failure can undermine either a bent or a bridge abutment and cause failure of approach fills. The resulting damage may be sufficient to require major repair work or even bridge replacement. A bend near a multiple barrel culvert will cause the flow to favor one of the barrels and deposition to occur in the other(s). This will result in reduced capacity for the culvert with possible failure during a flood event. Reference 14 provides excellent information on stream stability and stream geomorphic responses.

A field inspection is required to assess the potential for a failure due to lateral instability. Nearby bridges should be inspected to determine how the stream has reacted to their presence. Any indication of bank failure and/or the presence of meander bends in close proximity to the proposed bridge site should be noted.

A comparison of aerial photographs taken over a relatively long time interval should give some indication of the rate of movement of meander bends.

When an existing bridge is to be replaced, comparison of the original and current plan and profiles can be used to accurately determine how much the channel has shifted since the bridge was built.

Clearing the right of way in the vicinity of a channel bend can cause acceleration of channel migration. If horizontal channel stability poses a problem for the bridge, some form of channel stabilization will be required. HEC-20 provides a number of alternatives to solve this problem. In addition to the HEC-20 procedures, the Natural Resources Conservation Service (NRCS) has developed the *Streambank Stabilization Manual*, which contains appropriate Best Management Practices. In some specific cases bioengineering techniques may be used.

Step 5 Vertical stability

Bridges and culverts can be affected by streambed elevation changes. Aggradation, a rise in streambed elevation, reduces the openings of both bridges and culverts. This is generally more of a problem for culverts. Aggradation will also cause the channel to widen. This can cause scour problems for bridge piers located on or just behind the original channel bank.

Aggradation is caused by an increase in the sediment supply or by a reduction in the energy grade of the stream. Changes in land use in the drainage basin from pasture or forestlands to row crops will increase the sediment supply. Other land-disturbing activities can also increase the sediment supply. The reduction in energy grade can be caused by the construction of a dam or other backwater-causing structure downstream.

Degradation, a general lowering of the grade of a channel, can lead to piping underneath the bottom slab of a culvert with possible structural failure and breaching of the roadway. At bridges, degradation can cause potential scour depths to increase, thus endangering the bridge substructures. This can result in the catastrophic failure of the bridge. A headcut moving upstream can cause abrupt degradation at a bridge or culvert site.

A decrease in sediment supply or an increase in energy grade also causes degradation. Sediment supply can be reduced by urbanization of the drainage area, conversion of row crops to pasture or forest, construction of a dam upstream, or mining of sand or gravel in the streambed upstream. The energy grade can be increased by straightening the channel or mining in the streambed downstream. Channelization of a swamp can also cause degradation.

Location of the bridge or culvert in or near a meander bend can have a combination of both aggradation and degradation. The buildup or shifting of a point bar on the inside of a bend will cause aggradation. Scour on the outside of the bend will cause degradation.

<u>Step 6</u> Debris Potential

Evaluate potential for debris accumulation. (Note: This step is not included in the HEC-18 procedure, but because of the severe debris problems experienced by the Department, it is included at this point in the analysis.)

Debris accumulation on a structure is one of the main causes for bridge and culvert failure in South Carolina. At bridge sites, the channel can be blocked by debris that has accumulated on the bridge substructure. This will cause flow to be diverted toward the channel banks, damaging piers or scouring out the abutments. Flow will also be directed downward, scouring the channel bottom and undermining the bridge substructure. The reduction in opening due to the debris accumulation on the structure will cause backwater upstream with an increased flooding potential.

Debris can block culverts, causing overtopping to occur. In most cases, this will quickly lead to breaching of the roadway fill and total failure of the culvert.

Debris comes from both natural and man-made sources. Most South Carolina floodplains are forested. Both lateral and vertical channel movement undermine channel banks, causing trees growing along the banks to fall. These trees can be carried downstream and become entangled on the bridge substructure or culvert. Trees that die from natural causes can decay and fall into the channel. Wind or ice storms can also break off trees. All types of waste materials from human activities can end up as debris in stream channels (e.g., tree limbs and tops left over from timbering operations, discarded household appliances, garbage, furniture, mattresses, shopping carts, building materials, and even automobiles). Beavers will sometimes use either the bridge or culvert as supports for their dams.

A field inspection is necessary to determine the potential for debris. Upstream and downstream bridges should be checked to determine if there is any debris accumulation. Any sign of bank instability along the stream is a good indicator of debris potential. Local maintenance personnel and bridge inspectors can provide information on the history of debris problems along the stream.

<u>Step 7</u> Stream Response

An analysis of the information developed in the first 6 steps of this process will give a qualitative estimate of how the stream will respond to the proposed bridge or culvert construction. Chapter 3 of HEC-20 and Chapter 5 of Reference 14 provide good discussions on stream responses to change.

At this point in the process, a determination is made as to whether a more detailed analysis is required. If the bridge or culvert is not susceptible to scour and if hydraulic design is not required, the analysis can be concluded. Otherwise, the next step is a Level 2 Analysis.

1.2.2 Fast Track Bridge Investigation.

The Fast Track Bridge Replacement Program is designed to advance projects to construction quickly, reduce preliminary engineering time and effort, and maximize the number of bridges replaced with the available funds for bridges. A bridge may be replaced under the Fast Track Bridge Replacement Program if all of the following criteria are met:

- No right of way is required
- It can be replaced without a hydrologic and hydraulic study
- No survey is required
- The bridge can be closed to traffic during construction

In order to determine if a bridge that is being fast tracked due to hydrologic/hydraulic issues meets these criteria, a Design Field Review (DFR) team, that includes an experienced hydraulic design engineer, will visit the site. Prior to the site visit, the hydraulic design engineer should determine if the bridge is located in a floodway. If it is in a floodway, a FEMA study shall be performed to handle the FEMA requirements. If a "No Impact Certification" cannot be obtained, the project cannot be handled as a fast track bridge. The hydraulic design engineer should check for high-water measurements from existing plans, USGS measurements, known gauges, and other readily available sources. During the site visit, the hydraulic design engineer shall, based on experience and engineering judgment, do the following:

- Estimate the high-water elevation
- Determine if there are any hydraulic problems at the site
- Determine if any bridges upstream or downstream have had any hydraulic problems
- Estimate bridge scour depths utilizing USGS envelope curves

In conjunction with the bridge engineer, establish the bridge superstructure type, span length, bridge length, and minimum bridge finished grade elevation. A Level 2 study must be performed if one or both of the following occurs:

- If a more detailed study is required to determine any of the above information
- If, in the opinion of the hydraulic design engineer, the high-water elevation is high enough that the grades must be raised to the point that additional right of way is required

1.3 Level 2: Basic Engineering Analysis

A Level 2 Analysis involves the basic engineering analysis to hydraulically design a bridge or culvert, to determine the scour depths, and to design necessary countermeasures. A diagram of the steps in the procedure is presented in Figure No. 2, Section 1.6.4. Two different procedures are given below: one for riverine bridges and one for tidal bridges.

1.3.1 Level 2 Procedures for Riverine Bridges.

There are ten (10) basic steps to be followed when designing a riverine bridge. These steps involve reviewing the history of the site and the stream, analyzing appropriate hydrologic and hydraulic factors, considering the potential for scour and erosion, and performing a risk analysis. These steps have been developed based on standard engineering practices and the experience of the Department. This does not preclude the use of engineering judgment by the designer; however, any procedure that is outside the procedures described herein should be reviewed by the Department's hydraulic design engineer before it is used.

<u>Step 1</u> Flood History and Hydrology

A. Flood History

The first and one of the most important parts of the study is to find all of the available flood history of the stream. Sources for this information include, but are not limited to:

- Road and bridge plans on file in the Department's Central Office should have high-water data and in some cases related discharges indicated on the plan and profile sheets.
- High-water information on specific flood events is on file in the Department's Hydraulic Design Support Office.
- Interviews with local residents can produce useful information. The resident's name, address, and the length of time as a resident in the area should be obtained. The resident should be asked to point out specific high-water marks, if possible, which could be tied to the survey datum. They should also be asked to identify when the flood occurred and how frequently the general area is flooded. If possible, the information should be verified through independent sources. Local Department Maintenance personnel may also be able to furnish similar information.
- The USGS gage records cover many streams in the State. A search of these records may furnish valuable and accurate data. The USGS office may have unpublished information on high-water information that can be obtained.
- Local newspaper files may have stories on previous floods.
- Refer to the flood insurance study produced by FEMA. The narrative portion contains a history of flooding problems for most streams.

If enough data is available, a stream profile showing all of the data should be developed. The profile can be used to determine the starting elevations of flood flow slopes for computer models and as a check or calibration of computed flow profiles.

B. Bridge Site Scour History

Assemble all available information on the scour history of bridges at or near the site. Some sources of information are:

- The Bridge Inspection and Maintenance files
- A comparison of the original bridge plan and profile with the currently surveyed profile
- Aerial photographs covering as long a time span as available
- The history of sand or gravel mining on the stream

Based on the information collected, an indication of the long-term channel stability and aggradation or degradation can be estimated. An evaluation of the existing bridges' performance can also be made.

C. Hydrology

For all types of bridge or bridge-sized culvert studies, the 2-, 10-, 25-, 50-, 100-, and 500-year frequency discharges are needed. The steps used to compute the flows depend on the size of the drainage area, land use, the availability of gage data, the location in the State, and topography of the drainage basin. These steps are discussed below.

1. Using topographic maps and, if necessary, a field inspection, determine the boundaries of the drainage basin and measure the area. Determine the land usage from aerial photography, topographic maps, and a site visit. If the area encompasses a developing urban area, check with the local planning commission to determine future growth patterns.

2. Check the USGS gage record annual reports to see if there is or has been a gage on the stream. Reference 5 has a listing of most of the historic gages in South Carolina. Some gages have been removed, so a number of annual reports need to be referenced to make this determination. If there is a gage, compile a complete listing of the annual peaks for the gage. Analyze the data using the Log-Pearson Type III frequency distribution. The results should be regionalized using the USGS regression equations.

3. For ungaged streams in rural drainage areas, use the USGS regression equations (Reference 19) to determine the discharge. If the drainage area has urban or developed areas, use the peak discharge method described in Reference 20, or the latest revision.

4. If the drainage area contains a Carolina Bay, has a significantly large pond or lake, has a culvert(s) with significant storage volume upstream, or is very flat, the runoff must be determined by routing the floods through the basin and taking into account the storage. Unit hydrographs should be developed using the methods in references 22 and 23. The flows should be routed using an acceptable routing program. The flat areas, storage areas behind culverts, and the Carolina Bays should be treated as reservoirs. The outflow ratings must be estimated based on the site conditions.

D. Develop Comparative Data
Find the road and bridge plans for all crossings on the stream that have drainage areas ranging in size from half to twice the drainage area of the study site. Record the following information on the Comparative Data Sheet (a sample form is in Section 1.6.2):

1. Channel length from comparative bridge to proposed bridge in miles
2. Drainage area
3. Physiographic region
4. 2-, 10-, 25-, 50-, 100-, and 500-year discharges
5. Bridge length
6. Average finished grade of bridge
7. Opening furnished under design high water
8. Velocity
9. Elevation of stream bottom
10. High-water elevation
11. Date of high water
12. Observed water elevation
13. Observed water date
14. File or Docket No. of the Plans
15. Tie to MSL datum if available

The information listed in items 5 through 15 above can usually be obtained from plans for existing roads and bridges in Departmental files. Physiographic region refers to four physiographic regions in South Carolina: the lower coastal plain, the upper coastal plain, the piedmont, and the blue ridge. These are delineated in Reference 5. The opening furnished is the cross-sectional area of the bridge opening under the high-water elevation. Observed water is defined as the water elevation observed by the survey party and is assumed to be a water elevation that is not affected by flooding and reflects the condition at the time of the survey.

Step 2 Evaluate Hydraulic Conditions

A. Evaluate Field Conditions
One of the first and most important aspects of any hydraulic analysis is a field evaluation. This involves an in-depth inspection of the proposed bridge site. A less detailed site visit should be made to the bridges included on the comparative data sheet.

Any dams located in the reach that will affect the bridge should also be field evaluated. Sufficient survey data must be obtained on the spillway to develop a rating for the dam. Storage behind the dam can usually be estimated from USGS topographic maps. However, if the maps are not sufficient and storage is a significant factor, the reservoir boundary should be surveyed to establish the stage-storage relationship for the reservoir.

Any natural hydraulic controls such as rock shoals, waterfalls, or beaver dams as well as man-made controls such as bridges, dams, or other constrictions that have taken place in the floodplain should be evaluated. If these controls have any effect on the high-water profile, they should be taken into account in the high-water modeling. The controls should be surveyed to hydraulically define their effects on the high-water profile.

1. Comparative Bridge Sites

The main purpose of inspecting comparative bridge sites is to evaluate the performance of other bridges on the stream. The information to be observed and recorded is shown on the site inspection form, shown in Section 1.6.3.

The form is to be used for railroad bridges as well as highway bridges. The profile of the railroad bridge opening should be measured. If the railroad crossing is close enough to the subject bridge site to affect or be affected by the hydraulics of the proposed crossing, sufficient information should be obtained to include the railroad in the hydraulic model. This may require a survey of the railroad crossing.

The hydraulic design engineer should pay special attention to the scour at the existing bridge end fill areas and should check for debris accumulation on piles and piers. This may be an indicator of the amount of debris loads that a stream may carry. If the existing bridge has a steel truss over the channel, this may be another indicator that the stream has a high debris carrying potential. If debris accumulation appears to be an issue, then span lengths and configurations should be adjusted accordingly.

2. Job Site Inspection

The purpose of field inspecting the proposed bridge site is to evaluate the stream characteristics and hydraulic properties, evaluate the performance of the existing bridge (if applicable), evaluate the channel and floodplain topography, and evaluate the adequacy and accuracy of the survey data.

The first task is to verify the geomorphic factors determined in the Level 1 analysis, using a copy of the geomorphic factor sheet shown in Figure 3, Section 1.6.4. If there is an existing bridge, the performance of the bridge should be evaluated using the Job Site Inspection Forms, shown in Section 1.6.3. Manning's "n" values should be determined using the method described in Reference 15 and documented using the appropriate forms. The hydraulic design engineer should walk along the channel both upstream and downstream at a distance at least equal to the floodplain width, if possible, depending on the terrain and ground cover. One of the most important items to note is the presence of hydraulic controls such as other bridge crossings, dams (either man-made, by beavers, or from debris), shoals, waterfalls, and sewer or water lines suspended across the channel.

A sketch should be made of the site illustrating the existing structure, the direction of flow, the channel alignment, and anything else that will influence the design of the proposed structure. All visible utilities should be indicated on the sketch. At culvert sites, indicate if and how much silt has been deposited in the barrels of the culvert. Sketch the culvert and illustrate the sediment deposits. Also, indicate the presence and size of scour holes at the ends of the culvert. A copy of the site inspection form is to be included in the documentation for the study report. Photographs should be taken of the site and included in the study report.

Site inspection is one of the most important aspects of bridge hydraulic design. The effectiveness of the inspection depends on the experience and knowledge of the hydraulic design engineer. It involves an understanding of geology, fluvial geomorphology, hydrology, open channel hydraulics, floodplain plant and animal life, and structural aspects of the bridge. Combining these areas of knowledge with a study of aerial photography, topographic mapping, and the hydraulic analysis will help the hydraulic design engineer evaluate how well the computer modeling actually reflects real hydraulic conditions at the bridge site. If the modeling does not effectively reflect actual conditions, a Level 3 analysis with a two-dimensional model may be required. Also, the site inspection with an understanding of fluvial geomorphology will reveal the natural progression of channel movement, which may impact the bridge or roadway in the future. HEC-20 and the *River Engineering for Highway Encroachments* (Highways in the River Environment/Reference 14) present a background in fluvial geomorphology.

B. Hydraulic Analysis

Water Surface profiles are to be computed for the 2-, 10-, 25-, 50-, 100-, and 500-year flood events. The 2-year flood profile is computed because it is approximately the mean annual flood or the dominant, bank-full flood that shapes the channel. The velocities from the 2-year flood can be used to evaluate the stresses that are modifying the channel.

If the stream does not contain a floodway, the high-water profiles will be computed using the HEC-RAS bridge routine. Data for the program will be developed from available survey data and USGS or other topographic mapping. If sufficient data is not available, additional survey data will have to be obtained. The limits of the profile computation should be extended downstream to the point where a change in starting elevation will not affect the computed high-water depth at the bridge. This downstream limit can be determined by computing a sensitivity analysis. The HEC-RAS model can be executed starting at normal depth then subsequent runs can be started three feet below and above normal depth to see if the model converges before the location of the proposed bridge. The upstream limit should extend to the limit of backwater from the bridge. The backwater is defined as the difference in the water surface elevation for the proposed conditions and the water surface elevation at the same location for natural conditions. Backwater should be measured at the cross section where the maximum backwater occurs. The model should be calibrated using known flood data if sufficient reliable data is available.

If the hydraulic design engineer suspects that there have been violations in the State Water Law or State or local drainage regulations by property owners along the stream that will be detrimental to the Department, the engineer shall report the violation to the Department's legal staff for consideration of legal action. All reports shall be supported by documentation verifying the violation, including reference to the specific law or regulation. Documentation should consist of:

- Photographs
- Videos
- Aerial photography, preferably showing before and after conditions
- Engineering studies
- Maps with the violation delineated
- Documentation of conversations with local residents
- Plotted surveys

For culverts, develop the natural or unrestricted high-water profiles using HEC-RAS. Using the results of these computations to determine tailwater ratings, evaluate the culvert hydraulic performance using the HY-8 program or the HEC-RAS culvert routine.

C. FEMA Analysis

If the stream contains a designated floodway, use the HEC-RAS / HEC-2 program and data from the floodway study. The survey data from the project should be superimposed on the model data from FEMA. The floodplain and floodway must be remapped following FEMA's LOMR procedures if:

- There are any major discrepancies between the FEMA data and highway survey data, requiring modification of the floodway, or
- There are physical conditions in the floodplain not modeled in the floodway that would change the floodway or impact the bridge.

If the study is for bridge scour only, no floodway remapping is required.

Compare the discharges in the floodway study with the computed discharges. If the floodway study values are significantly different than the computed values, then consideration should be given to modifying the floodway. A significant difference means that the published values will affect the design of the bridge. In all cases, the safety of the public and property at the design discharge should be the deciding factor.

When a floodway is being remapped, the published floodway boundaries and widths should be matched as closely as possible. These boundaries affect the property development rights of local property owners. To do this, HEC-RAS encroachment Method 1 should be used. This method holds the floodway location by exact station and allows the high-water profile to vary. This method is acceptable as long as the increase in water surface above the base flood does not exceed 1.0 foot. The base flood is defined by FEMA as the 100-year flood profile at the time the floodway was adopted.

When any construction occurs in a floodway, a No Impact Certificate must be submitted to the local community or a CLOMR with supporting documentation must be submitted to FEMA and the local community. Under the latest FEMA regulations, a No Impact Certificate can be issued if there is no change in the flood profile or floodway width at any cross section. All application forms and supporting documentation required by 44CFR, Part 65, should be submitted to the appropriate entity.

All application forms and supporting documentation required by 44CFR, Part 65 should be submitted to the appropriate entity.

- A copy of the original computer runs with the floodway table.
- A copy of the original computer runs with 10-, 50-, 100-, and 500-year flood profiles.
- A copy of a floodway computer run for existing conditions using the original floodway data with the additional cross-section data from the project survey data superimposed. Any change in roughness values should be included. Comment cards should be included in the input data set to identify changes and bridge locations.
- A copy of the computer runs for the 10-, 50-, 100-, and 500-year flood profiles using the modified data.
- A copy of computer runs modeling the proposed construction with a floodway analysis and the 10-, 50-, 100-, and 500-year flood profiles.
- A copy of the published floodway map and the published flood profiles.

If a Conditional Letter of Map Revision (CLOMR) is required, then the same supporting documentation should be submitted with:

- documentation of the changes that were made,
- a revised Flood Insurance Rate Map (FIRM) showing the revised floodplain and floodway boundaries,
- revised flood profiles,
- certification of notification of property owners,
- CLOMR application form, and
- A copy of the proposed construction plans.

For bridge scour studies, the profiles should be computed for existing conditions only. No FEMA submittals are required unless mitigation measures are required. The discharges used for scour studies should be computed values rather than the FEMA study discharges. Many FEMA studies were done years ago and do not have the benefit of additional years of record as do the methods described in the section on hydrology in Section 1.3.1, Step 1.C.

For the design of new bridges or bridge replacement projects, profiles should be run for the natural or unrestricted condition, for the existing condition (that is with the existing bridge), and for the proposed structure. Bridge widening projects need a scour study and, if the stream has a designated floodway, a FEMA submittal. The following factors should be considered in determining if a Level 3 analysis, using a two-dimensional model, is required:

- The floodplain is wide and has significant lateral flow requiring more than one bridge
- There is significant lateral flow in the vicinity of the bridge
- The bridge will be in close proximity to a major meander or bend
- A stream junction of sufficient size that may affect the hydraulics is present

Step 3 Bed and Bank Material

An understanding of the properties of the material that makes up the bed and banks of the stream is essential in determining the stability of the channel and the scour potential of a bridge or culvert.

Soil borings will be taken by the boring crews at pier or bent locations as specified by the Geotechnical Engineer. Additional borings may be requested if needed. Samples should be taken at each boring site for each type of material encountered. A laboratory sieve analysis of samples should be performed to determine the size distribution of the material.

Step 4 Evaluate Watershed Sediment Yield

The important aspect in evaluating the watershed sediment yield is estimating changes in the sediment supply. This is largely dependent on changes in land use in the drainage area. Evidence of a change in sediment supply will be signs of recent aggradation or degradation. If a large rate of change in the sediment yield is found, a Level 3 analysis may be needed to evaluate the impact on the bridge or culvert structure. The computer program BRI-STARS, version 3 of FESWMS, or the Corps of Engineer’s sediment routing program can be used for the Level 3 analysis.

Step 5 Incipient Motion Analysis

An evaluation of the incipient motion of the stream’s bed material can give an indication of the stability of the channel. Procedures for performing this analysis using the equation for incipient motion based on the Shields Diagram, are discussed in HEC-20. This step should be accomplished using streambed samples obtained in Step 3. If channel instability is indicated, a more in-depth study and channel stabilization using the methods in HEC-11 and HEC-20 may be needed.

Step 6 Evaluate Armoring Potential

The potential for armoring occurs when there is material present in the bed material that is too large for flood flow to move. Over a sufficiently long time, floods will leach out the smaller material. The larger material that is left will accumulate in a layer in the bottom of the stream. This layer may armor the stream and prevent further scouring. The determination of the potential for this layer to occur is based on the incipient motion analysis. Armoring is generally rare in South Carolina.

<u>Step 7</u> Evaluating Rating Curve Shifts
If there is a USGS gage on the stream, an evaluation of long-term rating curve shifts should be made. Rating curves may shift due to changes in the factors that determine the relation between stream stage and stream flow. These factors are:

- Slope of the stream (affects velocity)
- Roughness of the channel
- Area of the channel at each stream stage
- Backwater effects (when a tributary enters a larger river)
- Filling in, scouring out, or channel changes of river banks
- Development in the watershed or structural changes such as the construction of nearby stream crossings or creation of dams.

Review of the rating curve shifts will give some indication of the long-term stability of the stream. The USGS will have a record of the long-term rating for the gage. If the stream bed is changing in elevation, a more detailed study will be needed.

<u>Step 8</u> Design Bridge
A. Establish the minimum low chord elevation based on the hydraulic design criteria. This information should be given to the road designer to establish the finished grade. Road requirements may dictate a higher grade than the hydraulic requirements.

B. Establish the orientation of the bridge substructure by determining the high flow angle. This should be based on topographic maps, aerial photographs, and field inspection. If a two-dimensional hydraulic model is used, it will compute the velocity vectors, which will show the high flow angle directly. The piers and bents should be oriented to present a minimum cross-sectional area to the flow. There will be some sites where the flow direction is significantly variable between low flow and high flow or the flow angle is variable throughout the bridge so a uniform skew angle cannot be used. At these sites, it may be necessary to have single circular shafted hammerhead piers for the substructure.

C. Set the bridge geometry. This step should involve consultation with the Regional Production Group (RPG) structural design engineer. The span over the channel should be set first. The controlling factors are the potential for debris and the substructure cost. If debris potential is significant, the debris can accumulate on the substructure causing two possible failure modes:

- The debris can effectively block the flow through the bridge structure. The flow will be diverted either around the debris, causing abutment failure, or downward beneath the debris, causing scour failure of the substructure.
- The debris can cause structural failure of the substructure by side pressure.

Locate the spans using a copy of the bridge plan and profile. If possible, the channel should be completely spanned. The substructure should be set a sufficient distance behind the channel banks so the banks will not be damaged by heavy equipment during construction. The approach span lengths should be set based on the structure height and the compatibility with the channel span. If the bridge is parallel to an existing bridge, the bents and piers should be aligned with the existing bridge bents and piers, if practical. Review the bridge span setup with the RPG structural design engineer. Superstructure depths for bridge spans can be found in Table 1.

Table 1: Superstructure Depth for Bride Spans

SUPERSTRUCTURE DEPTH FOR BRIDGE SPANS			
TYPE		**SPAN LENGTH (ft)**	**SUPERSTRUCTURE DEPTH (ft)[*]**
Flat Slab[**]		22– 30	1.33(continuous)-1.58(simple)
		40	1.67 (continuous)
Cored Slabs[***]		30 – 40 – 50	1.75.[***]
		60	2.00[***]
		70	2.00[***]
Prestressed Beams	Type II	40 – 60	3.92
	Type III	60 – 90	4.67
	Type IV	90 – 100	5.42
Bulb T Beams	54in.	90– 100	5.50
	63in.	100– 120	6.25
	72in.	120 – 140	7.00
Rolled Steel Beams		60 – 90	3.92
Steel Plate Girder		> 90	L/25 minimum

[*] Crown drop must be calculated and added to ‘superstructure depth’ to obtain minimum F.G. elevation.

[**] Flat slab lengths are limited to 22 ft., 30 ft., and 40 ft. with the 30 ft. span being the preferred length.

[***] All cored slabs must have 2 inches added to slab depth in addition to crown drop for asphalt overlay. Cored slab units are not allowed on Interstate, National Highway System routes, or any road with average daily traffic of 3,000 v.p.d. or greater without prior approval.

NOTE: This table is for preliminary bridge span selection. All final selections must be discussed and approved by the appropriate RPG structural engineer.
Revised 3/16/09

D. Set the location of the bridge ends. If the bridge is being replaced in the same location, the new bridge ends should not be located inside the existing bridge ends unless the cost is prohibitive. It is difficult to achieve compaction of the extended fills. Settling will occur, resulting in the development of a bump at the end of the new bridge. If the new bridge is to be longer than the existing bridge, the end fills should be excavated back to the designed end fill slope. Also, when replacing a bridge at the same location the existing pile or pier type and placement should be considered when setting the location of the proposed piles or piers.

If the bridge is parallel to an existing bridge, the end fills should be aligned. If the existing bridge is longer than the hydraulic design requires, the new bridge may be shortened as justified by the hydraulic computations. Consideration should be given to the impacts of the shorter bridge accelerating flow through the existing bridge.

If the stream is skewed to the roadway or if it has a bend in the roadway area such that the fills may spill into the channel, drawing a detailed contour map showing the fill slopes will be very helpful in setting the ends of the bridge. A projection of the bridge fills should not extend into the channel and should be at least 5.0 feet behind the channel banks. A CADD operator should be able to produce the contour map relatively easily using roadway cross sections, stream traverse cross sections, and the 25 feet left and right profile survey data.

A first estimate for the length of a bridge on a new location should be based on the length of bridges upstream and downstream. On the plan and profile sheet, project the end fill slopes from the finished grade elevation at the end of proposed spans, using 2:1 slopes perpendicular to skew. This projection will be accurate for cored slab and flat slab bridge. On all other type bridges (prestressed beams, girders, etc.) this estimate will be somewhat conservative due to the use of parallel wing walls. The final end fill projection must be reviewed by the appropriate RPG structural design engineer.

E. Evaluate the hydraulic performance of the proposed bridge by inserting the necessary bridge data information into the computer data set used to compute the high water. Then compute the profiles for the design discharge. If the performance does not meet the required criteria, adjust the length accordingly and re-evaluate the results. Continue to try different bridge lengths until the design criteria are satisfied.

F. For both bridges and culverts, it may be technically desirable in some instances to construct a channel change to improve the hydraulic performance of the structure. However, channel changes are objectionable to environmental resource agencies. Mitigation will be required if channel changes are used. Proposed channel changes are to be avoided to the maximum extent practical and should be discussed early in the design process with the Environmental Management Office.

<u>Step 9</u> Scour Analysis

A scour analysis will be performed for each bridge. This shall be accomplished using the procedures in FHWA's Hydraulic Engineering Circular No. 18, *Evaluating Scour at Bridges*. If the soil is a cohesive material such as clay, the procedures described by Dr. Jean-Louis Biraud (Reference 40) should be utilized. Other references that should be consulted are HEC-20 *Stream Stability at Highway Structures* and Reference 14, *River Engineering for Highway Encroachments* (Highways in the River Environment and Environmental Design Considerations).

In the scour study, special emphasis should be placed on areas where problems or failures have occurred at bridge sites in the past. These are:

1. One of the primary scour failure modes is scouring of the abutment. This is due to insufficient bridge opening or a large discharge in the overbank area. High velocities occur adjacent to the abutment, requiring properly sized riprap to be placed on the spill-through abutments. Abutment scour causes a scour hole to develop just off the abutment. The overbank flow has to make a severe turn at the abutment to get through the bridge opening. This creates an eddy current, which will scour around the end of the abutment. This scour often occurs in the vicinity of the first interior bent. Guide banks (spur dikes) should be considered for protection against this type of failure.

   This type of scour can occur in wide floodplains because of the lack of over flow bridges or the spacing between bridges is too far. In general, spacing of bridges should not exceed ½ mile in wide floodplains. Two-dimensional hydraulic modeling is necessary to adequately design bridge spacing in wide floodplains. All bridge abutments should be protected from scour by riprap unless the velocity for the design flood is below the scour threshold. Riprap will protect bridge abutments from scour in most cases.

2. Scour can be caused or increased by debris accumulation on a bent. The debris will cause the flow to be diverted downward and/or laterally. Significant scour damage can occur. To prevent this, the channel should be spanned. Tower bents should not be used in the channel or on the channel banks.

   Scour analysis for riverine bridges should be performed using USGS envelope curves. Engineering judgment and limitations for applying the curves are well documented (Reference 48, 49, & 50). In all cases, read these manuals carefully before applying the curves. These curves are limited to the 100-year scour computations. For this reason it is necessary to compute the 100-year and 500-year scour using HEC-18 procedures. The percent increase from the 100-year scour to the 500-year scour from HEC-18 procedures will be applied to the 100-year scour obtained from the USGS envelope curves to predict the 500-year scour.

   This method may be modified once the USGS scour manual is completed. Please be aware of this potential change in 2010.

3. If the bridge crossing is located in or near a channel bend, channel migration will probably occur during the life of the bridge. Channel stabilization should be considered using the

methods in HEC-11 and HEC-20. Placing the bridge foundations deep enough to withstand channel scour would be a viable alternative if the rate of migration would be such that it would not reach the bridge abutment during the lifetime of the bridge (75 to 100 years).

4. If gravel or sand mining occurs on a stream, it may cause channel degradation. This will be added to the other scour components in determining scour depth.

5. Scour on tidal streams is a special case. The scouring events may be associated with normal tidal flow, weather fronts, or a tidal surge from a hurricane. Channel migration of tidal streams is a particular problem. Historic aerial photographs dating back as early as possible should be studied to determine direction and speed of channel migration in the vicinity of the proposed bridge.

The results of the scour analysis may affect the design of the bridge's substructure. The foundation is to be designed for the 100-year frequency storm and have a safety factor of 1.0 for the 500-year flood or for a flood that will cause a critical condition such as the overtopping flood. This means that all bearing should be achieved below the predicted scour depths. No lateral support should be considered above this depth.

A scour analysis should also be prepared for bottomless culverts. Use the equations found in *Bottomless Culvert Scour Study: Phase I Laboratory Report - Publication No. FHWA-RD-02-078 when* performing scour analysis calculations for bottomless culverts. This publication can be found at the following website: www.fhwa.dot.gov/engineering/hydraulics/

Step 10 Risk Assessment
When the bridge hydraulic design is selected, a risk assessment will be performed to determine if a more economical design approach should be considered. The risk assessment involves answering a series of questions that will determine the need for a full risk analysis. Forms for this procedure begin in Section 1.6.4.

1.3.2 Level 2 Procedures for Tidal Bridges.

The hydraulic design procedures for tidal bridges are fairly new and relatively little experience has been gained in this area. The latitude of permissible procedures is consequently greater than the latitude for riverine hydraulics. However, any procedure that is outside the procedures described herein should be reviewed with the Department's hydraulic design engineer before it is used. See Reference 41 for a more complete discussion of tidal hydraulics.

Tidal hydraulics are produced by astronomical tides and storm surges and are sometimes combined with riverine flows. Storm surges are produced by wind action and rapid changes in barometric pressure. The driving force in riverine hydraulics is the gravitational force down the topographic slope of the stream. In tidal hydraulics, the driving force is the rapidly changing elevation of the tide and wind setup.

Storm surges in South Carolina are produced by two types of storms: hurricanes and northeasters. The northeaster is a long duration storm and is usually not as intense as a hurricane. Hurricanes are the storms that will be used for design purposes in most cases. The hurricane storm intensity and duration are described by four factors:

- The central core pressure
- The radius of maximum winds
- The forward speed
- Direction of forward motion

Tides are created by the combined gravitational forces of the moon and the sun. When the moon and the sun are lined up so that their gravitational forces are both pulling in the same direction, they produce a spring tide. This is the highest gravitational tide during the 28-day lunar month. When the moon and the sun are pulling in opposite directions they produce a neap tide or the lowest tide of the lunar month. The average or mean tide range along the South Carolina coast varies from about 5 to 7 feet. This much change in water surface elevation at the mouth of an estuary or inlet over a six-hour period can cause significant flows in tidal streams. The volume and stage of the water that flows past any point on a tidal stream during a tide cycle depends on:

- The volume of stream and marsh area contained between the elevations of high and low tide to the landward side
- The rate of change in tidal elevation
- The conveyance of the tidal channel
- Wind direction, duration, and intensity

Storm surges can cause much higher impacts. The published 100-year stillwater tide elevations along the South Carolina coast range from 12 to 14.5 feet mean sea level (msl). For the 500-year tidal surge, the range is 16 to 19.1 feet msl. The maximum surge height produced by Hurricane Hugo was 22 feet, based on high-water marks along the Cape Romain Wildlife area.

Wind effects will have a significant impact on tide levels and the flow volume. Techniques to determine wind effects are available to the highway hydraulic design engineer. At the present time, advice from an expert in this field is recommended. Computer programs that take into account wind effects are RMA-2V and DYNLET. FESWMS-Flo2DH, has the option of considering wind capability in both the one-dimensional and in the two-dimensional modes.

In tidal areas, bridge lengths are generally controlled by wetland considerations rather than hydraulics. The primary purpose of hydraulic analyses for bridges in tidal areas is typically to establish the grade of the bridge and determine the scour depths around the substructure. Exceptions to this rule are where an opening is being created or increased in an existing causeway or where a culvert is used. In these cases, the opening must be sized so that the velocities through the opening will not create scour problems. A significant head difference can develop across a causeway due to either the tide or wind setup. Sufficient opening should be provided to relieve this difference. A detailed analysis should be conducted to correctly size the opening.

1.3.2.1 Establish Minimum Bridge Grades.

The recommended height for the bottom of the bridge superstructure as specified in Section 1.1.5.2 is the 10-year tidal surge plus wave height plus 2.0 feet. There are three primary sources for storm surge frequency data: ADIRC stations, NOAA Technical Report NWS-16, and the FEMA Flood Insurance Studies. The Corps of Engineers has produced a data set of tidal surge hydrographs for historic hurricanes that have hit the Atlantic and Gulf coasts. These events were simulated on a hydrodynamic storm surge simulator developing the surge hydrographs independent of the gravitational tides. The data are available at near coastal tide stations called ADCIRC stations. There are several ADCIRC stations along the South Carolina coast. *The Tidal Hydraulic Modeling For Bridges Users Manual*, Reference 41, lists site locations and gives the 50-, 100-, and 500-year surge heights for each stations. The 10-, 50-, 100-, and 500-year surge heights are given in the NOAA Technical Report NWS-16 for the State of South Carolina, Reference 42. Surge heights are also given in the FEMA Flood Insurance Studies for coastal counties. Care should be exercised when using the FEMA data because the tide heights are given along transects but the specific point where the height applies is not clearly defined. The tide data in the NOAA report applies to the coast so the surge must be translated upstream to the bridge site utilizing either one- or two-dimensional flow analysis.

Wave height computations should be based on the Corps of Engineers' *The Shore Protection Manual*, Reference 38. Generally two types of wave height analyses may be encountered. For bridges located right on the coast and exposed directly to the ocean, the wave height will generally be based on the height of the breaking wave for the minimum depth between the ocean and the bridge. For bridges not directly exposed to the ocean, the wave height is given by a series of forecasting curves for shallow water waves. The input variables are the water depth, fetch length, and wind speed. The fetch length is the maximum distance of wind exposure over water in direct line with the bridge. Determining the wind speed associated with the 10-year tidal surge has proven to be somewhat difficult. The best approach is to study the historic record for the nearest weather station along the coast. Find a hurricane that produced a 10-year tidal surge and then obtain the wind records for that storm. *The Shore Protection Manual* has a method for translating the wind velocity inshore to a different location from the wind gage.

1.3.2.2 Historic Storm and Site Data.

Historic hurricane storm data for the proposed bridge site should be investigated. Two possible sources of information are NOAA Technical report NWS-16, *Storm Tide Frequencies on the South Carolina Coast,* Reference 42 (http://tidesandcurrents.noaa.gov/products.html), and a NOAA storm data base available on diskette from NOAA entitled *North Atlantic Storms 1886-1994*. Other data sources may be newspaper accounts, NOAA weather records, and library resources.

The site data at tidal bridge sites can be recorded on the same forms as the riverine bridges. The same type of data should be gathered. The historic aerial photograph data is of significant importance because tidal streams and estuaries tend to have more movement than riverine streams in South Carolina. The movement of sandbars and the thalweg in tidal streams should be visible on aerial photographs.

1.3.2.3 Develop the Tidal Hydrograph.
As indicated in Section 1.3.2.1, 50-, 100-, and 500-year tide surge heights are given for the South Carolina ADCIRC stations in Reference 42. The ADCIRC data and the NOAA data are for stillwater heights; that is, no wave height is included. In some portions of the FEMA Flood Insurance Studies, the wave height is included in the tidal surge and in some of their tables stillwater heights are given. If FEMA data is being used, the designer should be very careful to make sure that stillwater heights are being used. The tidal hydrograph should be developed using the following equations:

$$S_{tot}(t) = S_p\left(1 - e^{-\left[\frac{D}{t}\right]}\right) + H_t(t)$$

$$D = \frac{R}{f}$$

Where:	S_{tot}	=	storm tide (combined surge and daily tide)
	t	=	time
	S_p	=	the known stage for selected return period
	D	=	Storm duration
	R	=	radius of maximum winds
	f	=	forward speed
	H_t	=	height of daily tide

Appropriate values for *R* and *f* can be determined from Appendix A of the NOAA report NWS-38 (1987). Figures giving these values are in Reference 41 and 42. It is recommended that the 50 percent values of *R* and *f* be used. Use of the 50 percent values produces a duration, *D*, which is very similar to that derived from an analysis of historical storm surge hydrographs. The 50 percent values are regarded as the most probable values given a surge height of any recurrence interval. The 100-year storm surge hydrograph, for example, should be developed using the 100-year surge height and the 50 percent values of *R* and *f*. Using a curve for *R* and *f* other than the 50 percent curve would lead to a hydrograph with a recurrence interval greater than 100 years.[1]

The basic equation for the tidal surge hydrograph is:

$$S_{tot}(t) = S_t(t) + H_t(t)$$

[1] Reference 42, pp. 42

To incorporate the full storm surge in the hydrograph, it is suggested that the hydrograph be set up so that the peak of the storm surge occurs at hour 50. The equation for $S_t(t)$ should be expressed as follows:

$$S_t(t) = s_p\left[1 - e^{-\left|\frac{D}{50-t}\right|}\right]$$

recognizing that $S_t(t) = S_p$ for $t = 50$.

More information on these equations can be found at the following internet website:

http://www.fhwa.dot.gov/engineering/hydraulics/hydrology/hec25c2.cfm (Eq. 2.6)

The next consideration is at what stage of the daily tide the peak tidal surge should occur: high tide, low tide, mid-rising tide, or mid-falling tide. The best approach is to compute hydrographs for each condition. The most conservative approach is to have the peak of the surge coincide with the mid-rising tide.

1.3.2.4 Tidal Hydraulic Analysis and Scour Analysis.

Reference 43, provides three approaches for developing the boundary conditions for tidal hydraulic modeling, the Corps of Engineers' method, the empirical simulation technique (EST), and the single design hydrograph. The single design hydrograph method is recommended for design purposes and is described herein. For a description of the other methods, see the Final Report, Phase II of Reference 43.

In the single design hydrograph method, the storm hydrograph is developed as described in Section 1.3.2.3. The resulting hydrograph is then applied as the downstream boundary condition for the hydraulic model. In the Level 2 analysis, the one-dimensional model HEC-RAS is used. If discharges from fresh water streams are to be considered, these are inserted in the form of a hydrograph that corresponds to the timing of the tidal hydrograph for the upstream boundary conditions.

A method for developing the fresh water hydrograph is suggested herein. Using historic rainfall data associated with a hurricane, approximating the magnitude of the storm under study, develop the rainfall hyetograph for the area. Timing of the rainfall coinciding with the approach of the hurricane is essential. Use an acceptable routing program to develop the fresh water inflow hydrograph based on this rainfall. Only consider that part of the stream's drainage area that would have a time of concentration corresponding to timing of the rainfall hyetograph and the duration of the tidal surge hydrograph. To account for the flow from upstream areas or base flow, use daily stream records to arrive at a normal daily flow for the stream. The fresh water hydrograph should be added to this daily flow. The resulting hydrographs would then be used as the flow at the upstream boundary of the hydraulic model.

Before the HEC-RAS model is run for design purposes, it should be calibrated with real tide data. NOAA has had several hundred tidal gages in South Carolina tidal waters. There are also several reference stations that have been in place for much longer. The data from these gaging stations can be used to calibrate the model. The data required is at least two tide gages on the tidal stream or estuary with data for the same time period. The tide gages and the project survey should be on the same datum. Twenty-four to forty-eight hours of tide data should be simulated for calibration. Wind setup can affect the gage readings. Using weather records, the designer should verify whether there is a problem with the gage data before using it.

Historical tidal elevations can be found on the website referenced in Section 1.3.2.2.

If the tide data is not available for the study site, two or more continuous recording tide gages should be placed on the tidal stream to obtain calibration data. One of the gages should be located near the mouth of the stream where the downstream boundary condition applies. At least one of the other gages should be located at or near the proposed construction site. Other gages may need to be installed depending on the complexity of the stream or estuary. The gages should be operated on a common time reference.

For scour analysis for tidal bridges, use the HEC-18 procedures.

1.3.2.5 Culverts in Tidal Streams.

Culverts in tidal streams should be analyzed with HEC-RAS.

The rising and falling tidal surges will each have a point of maximum outlet velocity, which will occur approximately mid-way between high and low tide. The exact timing of both points needs to be determined so that outlet scour protection may be designed for both ends of the culvert under maximum velocity conditions.

1.4 Level 3 Analysis

A Level 3 analysis is a more detailed analysis. It will generally be a two-dimensional hydraulic analysis using FESWMS with SMS or a physical model. SMS is a pre- and post-processor that helps facilitate the development of the FESWMS input data set and manipulate the output. It can also be used with a number of other programs including the Corps of Engineers' two-dimensional model RMA-2V and their sediment transport model. For a problem involving stream stability and sediment transport, the analysis may involve the use of the Corps of Engineers' sediment transport model or the FHWA model BRI-STARS. Version III of FESWMS will have sediment transport analysis capabilities. A full risk analysis is a Level 3 engineering economic analysis of a proposed bridge crossing. All computer models in the Level 3 analysis should be calibrated using real hydraulic data.

If the estuary is large, it may be necessary to run an unsteady HEC-RAS model first to establish upstream and downstream boundary conditions for the two-dimensional model. In this case, the two-dimensional model should cover an area of the estuary of sufficient extent to model the flow in the vicinity of the bridge.

In tidal analysis, use of the Corps of Engineers' method for tidal surge hydrographs or the EST approach would be considered a Level 3 analysis even if the one-dimensional model HEC-RAS is used due to the extra level of effort and the statistical analysis required in these approaches.

1.5 Information to Be Shown on Plans and Hydraulic Design Study Report

The following information for bridges, culverts, and bridge-sized culverts shall be shown on plans. All forms contained in this document are also located on the SCDOT Intranet/Internet pages. Those forms are updated regularly, and take precedence to those in the document. A formal study report with a title sheet and an index page will be prepared and both signed and sealed by a registered professional engineer of the State of South Carolina. The title and date of the edition of the *Requirements for Hydraulic Design Studies,* used for design purposes, shall be shown on the title sheet of plans and included on the cover page of the "Hydraulic Design Study Report." A reference of the document used should be shown on the Title Sheet of plans. It should also be included in the "Hydraulic Design Study Report."

1.5.1 For Bridges.

The form found in section 1.5.4 should be completed with all data clearly noted and appropriate comments made. Please note that the form contains a section titled, "Hydrology Data for Tidal Bridges" and a section titled, "Hydrology Data for Riverine Bridges". Only the appropriate section for the subject bridge needs to be completed. The data entered onto the form should be shown on the plans. In addition the 100- and 500-year scour lines should be plotted on the plan and profile sheet.

1.5.2 For Bridge-Sized Culverts (≥ 20' Opening).

The form found in section 1.5.5 should be completed with all data clearly noted and appropriate comments made. Please note that the form contains a section titled, "Hydrology Data for Tidal Culverts and a section titled, "Hydrology Data for Riverine Culverts". Only the appropriate section for the subject culvert needs to be completed. The data entered onto the form should be shown on the plans. In addition, the plans should indicate the direction of flow for maximum velocity.

1.5.3 For Culverts (<20' Opening).

The form found in section 1.5.6 should be completed with all data clearly noted and appropriate comments made. Please note that the form contains a section titled, "Hydrology Data for Tidal Culverts and a section titled, "Hydrology Data for Riverine Culverts". Only the appropriate section for the subject culvert needs to be completed. The data entered onto the form should be shown on the plans. In addition, the plans should indicate the direction of flow for the maximum velocity.

1.5.4 Hydrology Data Sheet for Bridges

MEMORANDUM TO:

Submittal Date: ______________

Supersedes Submittal Date: ______________

RPG ROAD DESIGN TEAM LEADER: ____________________

RPG STRUCTURAL ENGINEER: ____________________

From: Hydraulic Design Squad / Engineer ____________________

Subject: Hydrology Data for Bridge over ____________________

County: ____________________ **Rd/Rte:** ____________

Structure No: ____________________ **Const. Pin:** ____________

Bridge Data:

Bridge Length: __________ ft. Bridge Width: __________ ft.

Beg. Station: __________ Ending Station: __________

Pier/Pile Type: __________ ft. Pier/Pipe Width: __________

Skew Angle: __________ °

Bridge Span Configuration: ____________________

Bridge Span Type: ____________________

Min. F.G. Elev.: __________ ft. Min. Low Steel Elev. __________ ft.

Min. Bottom Interior Bent Cap Elev. (For Tidal Bridges Only) __________ ft.

Br. End Fill Slope: ________ Riprap Req'd: Yes ☐ No ☐ To Elevation: ______ ft.

Comments: ____________________

Historic High Water Information:

Elevation of High Water: __________ ft. Discharge: (if available) __________ ft.

Date of occurrence: ____/____/____ Source of Data: ____________________

Page 1 of 2

Design High Water and Backwater Information: (Show high water elevations including backwater on plans)

If 'Secondary Road' provide 25-yr high water elevation including backwater: __________ ft.

If 'Primary Road' provide 50-yr high water elevation including backwater: __________ ft.

For all roads provide 100-yr high water elevation including backwater: __________ ft.

Hydrology Data for Tidal Bridges: (Only complete this section if tidal flow is the dominant flow) (show on plans)

Mean Higher high tide elevation = __________ ft.

Mean Lower low tide elevation = __________ ft.

10-year tidal surge height = __________ ft. (includes wave height)

100-year stillwater height = __________ ft.

500-year stillwater height = __________ ft.

Maximum vel. within bridge = 100-yr. tidal surge velocity: _____ fps 500-yr. tidal surge velocity: _______ fps

Hydrology Data for Riverine Bridges: (Only complete this section if riverine flow is the dominant flow) (show on plans)

D.A. = __________ sq. mi. (or acres)

Q_{Design} = __________ cfs

Vel. $_{Design}$ = __________ ft./sec.

Design Headwater Elevation = __________ ft.

Including __________ ft. backwater

Q_{100} = __________ cfs

Vel_{100} = __________ ft/sec

100 Year Headwater Elev. = __________ ft.

Overtopping Flood:

Q = __________ cfs Probability = __________ %

cc: Environmental Engineer ____________________

Note: Probability may be determined by plotting the 2-, 10-, 25-, 50-, 100-, and 500-year discharges on Gumble paper and reading the probability corresponding to the overtopping discharge. For discharges greater than 500-year, the probability should be stated as less than (<) 0.002.Profiles of the computed scour for the 100-year and 500-year floods should be shown on the bridge plan and profile sheet. The shape of these profiles should be based on the methods described in the HEC-18. A plot of the 100- and 500-year scour lines on a bridge plan and profile sheet must be provided.

Revised 3/16/09

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1.5.5 Hydrology Data Sheet for Bridge-Sized Culverts (≥ 20' Opening)

MEMORANDUM TO: **Submittal Date:** ____________

Supersedes Submittal Date: ____________

RPG ROAD DESIGN TEAM LEADER: ____________________

RPG STRUCTURAL ENGINEER: ____________________

From: Hydraulic Design Squad / Engineer ____________________

Subject: Hydrology Data for Bridge Sized Culvert over ____________________

County: ____________________ **Rd/Rte:** ____________

Structure No: ____________________ **Const. Pin:** ____________

Culvert Dimensions: Span: ____________ ft. Rise: ____________ ft.

Extension: Right: ____________ ft. Left: ____________ ft.

Estimated Length: ____________ ft.

No. of Barrels: ____________ Material Type: ____________

Centerline Station: ____________ Skew Angle: ____________ °

Inlet Invert Elev: ____________ ft. Outlet Invert Elev.: ____________ ft.

Riprap Required (In Addition to Typical): Yes ☐ No ☐

Comments: ____________________

____

____

____

____

Historic High Water Information: (Show highwater on plans)

Elevation of High Water: ____________ ft. Discharge: (if available) ____________ ft.

Date of occurrence: ____/____/____ Source of data: ____________

Page 1 of 2

Design High Water and Backwater Information: (Show high water elevations including backwater on plans)

If 'Secondary Road' provide 25-yr high water elevation including backwater: __________________ ft.

If 'Primary Road' provide 50-yr high water elevation including backwater: __________________ ft.

For all roads provide 100-yr high water elevation including backwater: __________________ ft.

Hydrology Data for Tidal Culverts: (Only complete this sections if the culvert is tidally influenced) (show on plans)

Mean Higher high tide elevation = __________________ ft.

Mean Lower low tide elevation = __________________ ft.

10-year tidal surge height = __________________ ft. (includes wave height)

100-year stillwater height = __________________ ft.

500-year stillwater height = __________________ ft.

Maximum vel. within culvert = 100-yr. tidal surge velocity: _______ fps 500-yr. tidal surge velocity: ______ fps

Hydrology Data for Riverine Culverts: (Only complete this sections if the culvert is NOT tidally influenced) (show on plans)

D.A. = ________________________ sq. mi. (or acres)

Q_{Design} = ________________________ cfs

Vel. $_{Design}$ = ________________________ ft. / sec.

Design Headwater Elevation = ________________________ ft.

Q_{100} = ________________________ cfs

Vel_{100} = ________________________ ft. / sec.

100 Year Headwater Elev. = ________________________ ft.

Overtopping Flood:

Q = ___________ cfs Probability = _____________ %

cc: Environmental Engineer ____

Note: Probability may be determined by plotting the 2-, 10-, 25-, 50-, 100-, and 500-year discharges on Gumble paper and reading the probability corresponding to the overtopping discharge. For discharges greater than 500-year, the probability should be stated as less than (<) 0.002. A plot of the 100- and 500-year scour lines on a bridge plan and profile sheet must be provided. Revised 3/16/09

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1.5.6 Hydrology Data Sheet for Culverts (< 20' Opening)

MEMORANDUM TO:

Submittal Date: ____________

Supersedes Submittal Date: ____________

RPG ROAD DESIGN TEAM LEADER: ________________________________

RPG STRUCTURAL ENGINEER: ________________________________

From: Hydraulic Design Squad / Engineer ________________________

Subject: Hydrology Data for Culvert over ________________________

County: ____________________ **Rd/Rte:** ____________

Const. Pin: ____________________

Culvert Dimensions: Span: ____________ ft. Rise: ____________ ft.

Extension: Right: ____________ ft. Left: ____________ ft.

Estimated Length: ____________ ft.

No. of Barrels: ____________ Material Type: ____________

Centerline Station: ____________ Skew Angle: ____________ °

Inlet Invert Elev: ____________ ft. Outlet Invert Elev.: ____________ ft.

Riprap Required (In Addition to Typical): Yes ☐ No ☐

Comments: ____

____

____

____

____

Historic High Water Information: (Show highwater on plans)

Elevation of High Water: ____________ ft. Discharge: (if available) ____________ ft.

Date of occurrence: ____/____/____ Source of data: ____________

Page 1 of 2

Design High Water Information: (Show high water elevations on plans)		
If 'Secondary Road' provide 25-yr high water elevation:	____________________	ft.
If 'Primary Road' provide 50-yr high water elevation:	____________________	ft.
For all roads provide 100-yr high water elevation:	____________________	ft.

Hydrology Data for Tidal Culverts: (Only complete this sections if the culvert is tidally influenced) (show on plans)

Mean Higher high tide elevation	=	________________________	ft.
Mean Lower low tide elevation	=	________________________	ft.
10-year tidal surge height	=	________________________	ft. (includes wave height)
100-year stillwater height	=	________________________	ft.
500-year stillwater height	=	________________________	ft.

Maximum vel. within culvert = 100-yr. tidal surge velocity: ______ fps 500-yr. tidal surge velocity: _______ fps

Hydrology Data for Riverine Culverts: (Only complete this sections if the culvert is NOT tidally influenced) (show on plans)

D.A.	=	________________________	sq. mi. (or acres)
Q_{Design}	=	________________________	cfs
$Vel._{Design}$	=	________________________	ft. / sec.
Design Headwater Elevation	=	________________________	ft.
Q_{100}	=	________________________	cfs
Vel_{100}	=	________________________	ft. / sec.
100 Year Headwater Elev.	=	________________________	ft.

cc: Environmental Engineer ______________________________

Note: Probability may be determined by plotting the 2-, 10-, 25-, 50-, 100-, and 500-year discharges on Gumble paper and reading the probability corresponding to the overtopping discharge. For discharges greater than 500-year, the probability should be stated as less than (<) 0.002. A plot of the 100- and 500-year scour lines on a bridge plan and profile sheet must be provided.
Revised 3/16/09

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1.6.1 Title Sheet

HYDRAULIC DESIGN AND RISK ASSESSMENT FOR

BRIDGE / BRIDGE REPLACEMENT OVER

(*enter stream name here*)

ROUTE / ROAD NUMBER: ______________________

FILE NO.: ______________________

PROJECT NO.: ______________________

PIN: ______________________

COUNTY NAME: ______________________

DATE: ________ / ________ / ________

PREPARED BY: ______________________

CHECKED BY: ______________________

Hydraulic Design Reference for this study is the :

2009

Edition of SCDOT's "Requirements for Hydraulic Design Studies."

(Place stamp and signature in this space)

Signed and Sealed

1.6.2 Comparative Data Sheet

COMPARATIVE DATA

PROJECT DESCRIPTION

County:	________	Rt. / Rd. No.:	________
Stream:	________	File No:	________
Project No:	________	PIN:	________
Charge Code:	________	Road Squad:	________
Project Engineer:	________		

By:	________	Date:	________
Checked By:	________	Date:	________

	ROUTE/ROAD NO.'s				
DISTANCE FROM NEW BR. (mi.)					
DRAINAGE AREA (sq. mi.)					
ZONE					
Q_{10} (cfs)					
Q_{25} (cfs)					
Q_{50} (cfs)					
Q_{100} (cfs)					
Q_{500} (cfs)					
BRIDGE LENGTH (ft.)					
AVG. FINISHED GRADE (ft.)					
OPENING FURNISHED (sq.ft.)					
VELOCITY (ft./sec)					
HIGHWATER ELEV. (ft.)					
HIGHWATER DATE					
HIGHWATER DEPTH (ft.)					
OBSERVED WATER ELEV. (ft.)					
OBSERVED WATER DATE					
OBSERVED WATER DEPTH (ft.)					
FILE/DOCKET/PROJECT NO.					
DATUM/DATUM TIE					

1.6.3 Site Inspection Form

SITE INSPECTION FORM

PROJECT DESCRIPTION

County: ____________________ Rt. / Rd. No.: ____________________

Stream: ____________________ File No: ____________________

Project No: ____________________ PIN: ____________________

By: ____________________ Date: ______ / ______ / ______

Note: All references to left and right are looking in the direction of flow.

EXISTING BRIDGE

Length: ______ ft. Width: ______ ft. Max. Span Length: ______ ft.

Alignment: Tangent ☐ Curved ☐

Bridge skewed? Yes ☐ No ☐ Angle: ____________________

End Abutment Type: ____________________

Riprap on Fills? Yes ☐ No ☐ Condition: ____________________

Superstructure Type: ____________________

Substructure Type: ____________________

Utilities Present? Yes ☐ No ☐ Describe: ____________________

____

Debris Accumulations on Bridge: Percent Blocked (Horizontal): ______ %

Percent Blocked (Vertical): ______ %

Hydraulic Problems? Yes ☐ No ☐ Describe: ____________________

____

Draw Sketch of Bridge and Stream Below: (Show north arrow and direction of flow)

1.6.3.1 Site Characteristics Form

SITE CHARACTERISTICS FORM
General Topography ____
Stream Type (circle one)
Straight   Braided   Anabranched   Meandering
Are channel banks stable?   Yes ☐   No ☐
If No, describe:____ ____ ____
Soil Type ____
Exposed Rock?   Yes ☐   No ☐
If Yes, give description and location: ____ ____ ____
Describe potential for debris:____ ____ ____
Give description and location of any structures or other property that could be damaged by backwater:____ ____ ____
Describe any other features that might affect or be affected by the hydraulic performance of the proposed bridge:____ ____ ____

1.6.3.2 Manning's "n" Values – for Channels

MANNING'S "n" VALUES – FOR CHANNELS

$$n = [(n_b + n_1 + n_2 + n_3 + n_4)\, m]$$

Channel	**n_b -- *Base n for soil***	**Channel**	**n_1 -- *Degree of Irregularity***
Earth	.020	Smooth	.000
Rock Cut	.025	Minor	.001-.005
Fine Gravel	.024	Moderate	.006-.010
Course Gravel	.028	Severe	.011-.020

	n_2 -- *Variations of Channel Cross Sections*		**n_3 -- *Relative Effect of Obstructions***
Gradual	.000	Negligible	.000-.004
Alternating Occasionally	.001-.005	Minor	.010-.015
Frequently	.010-.015	Appreciable	.020-.030
		Severe	.040-.060

	n_4 -- *Vegetation*		**m -- *Degree of Meandering***
Low	.002-.010	Minor	1.00
Medium	.010-.025	Appreciable	1.15
High	.025-.050	Severe	1.30
Very High	.050-.100		

SITE OBSERVATIONS FOR CHANNELS

Channel Depth	n_b	n_1	n_2	n_3	n_4	m	***Computed n***

1.6.4 Risk Assessment

SOUTH CAROLINA DEPARTMENT OF TRANSPORTATION FLOODPLAIN AND RISK ASSESSMENT
Regulation 23 CFR 650 shall apply to all encroachment and to all actions which affect base floodplains, except for repairs made with emergency funds. (See HEC-17) Note: These studies shall be summarized in the environmental review document prepared pursuant to 23 CFR 771.

Project Description: ____

____

____

____

A. Narrative Describing Purpose and Need for Project:

a. Relevant Project History: ____

____

____

b. Project Location (attach Location and Project Map):

____

____

____

c. Major Issues and Concerns: ____

____

____

B. Are there any floodplain(s) regulated by FEMA located in the project area?

Yes ☐ **No** ☐

C. Will fill be placed within a 100-year floodplain?

Yes ☐ **No** ☐

Page 1 of 5

D. Will the existing profile grade be raised within the floodplain?

Yes ☐ **No** ☐

E. If applicable, please discuss the practicability of alternatives to any longitudinal encroachments.

____
____

F. Please include a discussion of the following: commensurate with the significance of the risk or environmental impact for all alternatives containing encroachments and those actions which would support base floodplain development:

i. What are the flood-related risks associated with implementation of the action?

____
____
____

ii. What are the impacts on the natural and beneficial floodplain values?

____
____
____

iii. Will the bridge entice people to build in floodplains?

____
____
____

iv. What measures were used to minimize floodplain impacts associated with the action?

____
____
____

v. Were any measures used to restore and preserve the natural and beneficial floodplain values impacted by the action?

____
____

G. Please discuss the practicability of alternatives to any significant encroachments or to support of incompatible floodplain development.

____
____
____
____

H. List local, state, and federal water resources and floodplain management agencies consulted to determine if the proposed highway action is consistent with existing watershed and floodplain management programs. Describe any information obtained on development and proposed actions in the affected area. Please include agency documentation.

____
____
____

I. BACKWATER DAMAGE FORM

Major flood damage applies to shopping centers, hospitals, industrial facilities, residential areas, schools, farming operations, etc.

1. *Does the maximum flood cause major damage to upstream property?*

Yes - (Go to 2.) No - (Go to 3.)

2. *Would this damage occur if the road were not there?*

Yes - (Go to 3.)

No - (Perform a limited Least Total Expected Cost (LTEC) (HEC-17) analysis to see if the bridge opening should be increased and/or grades raised to minimize the damage potential. Go to II.)

3. *Was this a bridge replacement? If so, was the bridge opening increased enough to increase the discharge passed through the bridge?*

Yes - (Go to 4.) No - (Go to II.)

Page 3 of 5

4. *Does the increased flow cause major damage downstream?*

Yes - (Perform a limited LTEC analysis to determine if the bridge opening should be reduced, the floodway redefined, and flood easements purchased upstream or if flood easements should be purchased downstream. Go to II.)

No - (Go to II)

II. TRAFFIC RELATED LOSSES

1. *Is the overtopping flood greater than the 100-year flood?*

Yes - (Go to III.) No - (Go to 2.)

2. *Does the ADT exceed 50 vehicles per day?*

Yes - (Go to 3.) No - (Go to III.)

3. *Does the duration of road closure in days, multiplied by the difference in length, in miles between the normal route and the detour, exceed 20?*

Yes - (Go to 4.) No - (Go to III.)

4. *Does the annual risk cost for traffic related costs exceed 10% of the estimated annual capital costs?*

Yes - (Perform a limited LTEC analysis to compare the cost to raise the grades and if necessary increase the bridge length with the traffic related costs. Go to III.)

No - (Go to III.)

III. ROADWAY AND/OR STRUCTURE REPAIR COST

1. *Is the overtopping flood less than the 100-year flood?*

Yes - (Go to 2) No - (Go to 3)

Page 4 of 5

***2.** Is the overtopping flood less than 0.5 foot over the low point on the roadway and duration no more than 1.0 hour?*

Yes - (Go to 3)

No - (perform a limited LTEC analysis to determine if the grades should be raised and/or the bridge opening increased or that the repair cost for embankment erosion are less significant. Traffic cost should be included in this evaluation.)

***3.** Is the proposed bridge or culvert structure subject to potential damage due to debris?*

Yes - (Go to 4) No - (Go to 5)

***4.** Perform a limited LTEC analysis to determine if the structure should be modified. (Go to 5.)*

***5.** The risk assessment has determined the most economical design for the crossing within the design constraint*s.

Revised 3/16/09

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Figure 1: Level 1: Qualitative Analyses Flow Chart

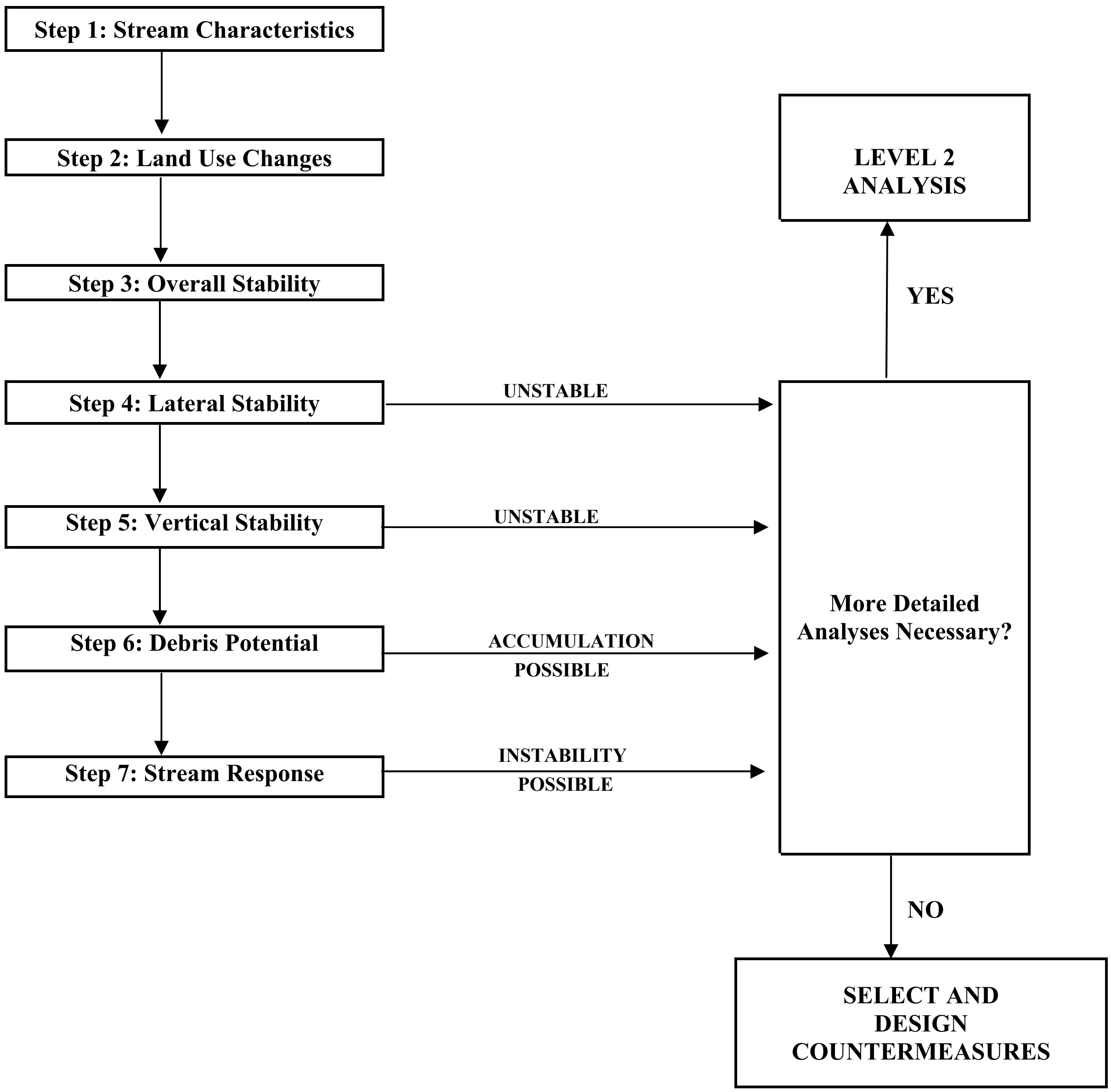


Figure 2: Level 2: Basic Engineering Analysis Flow Chart

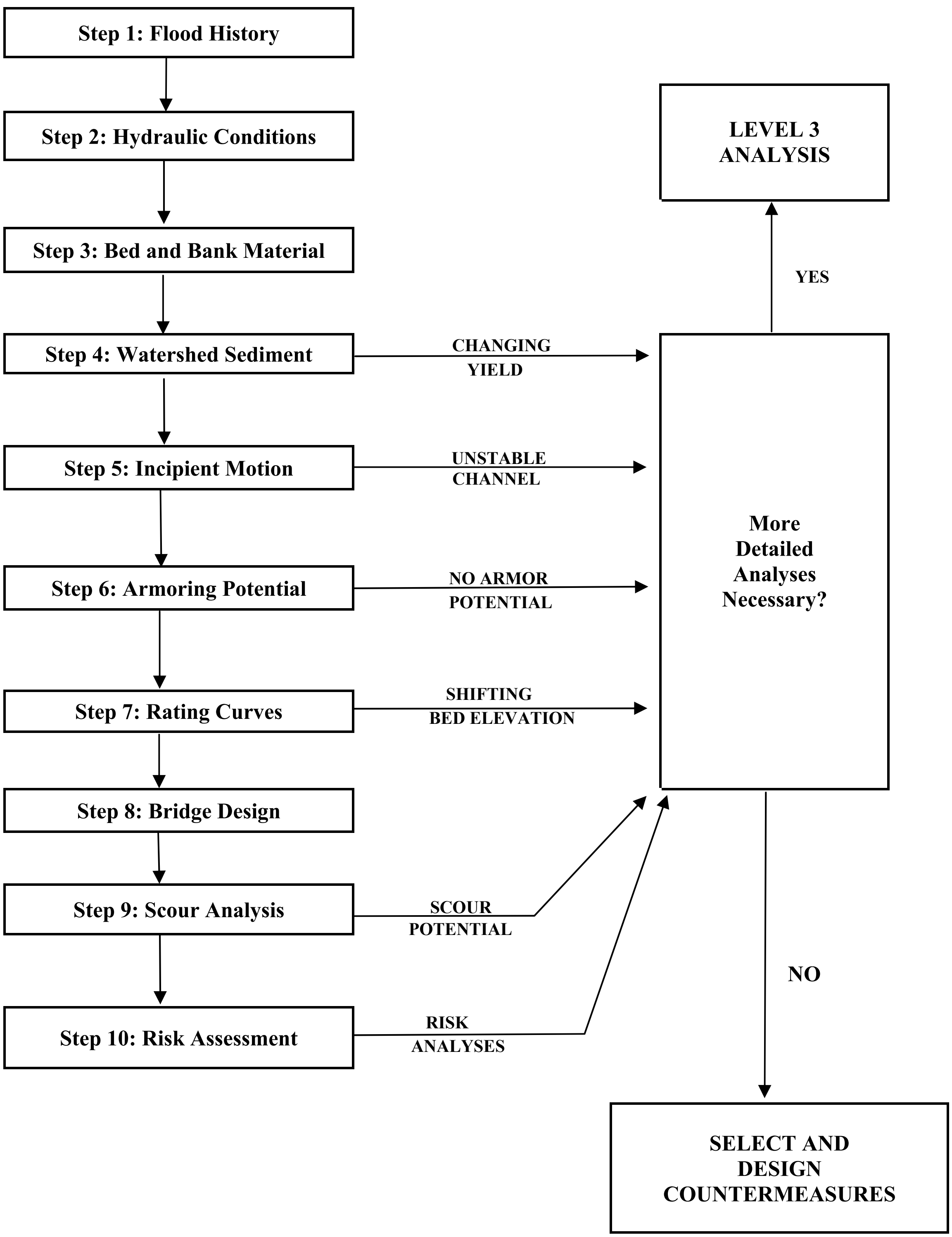
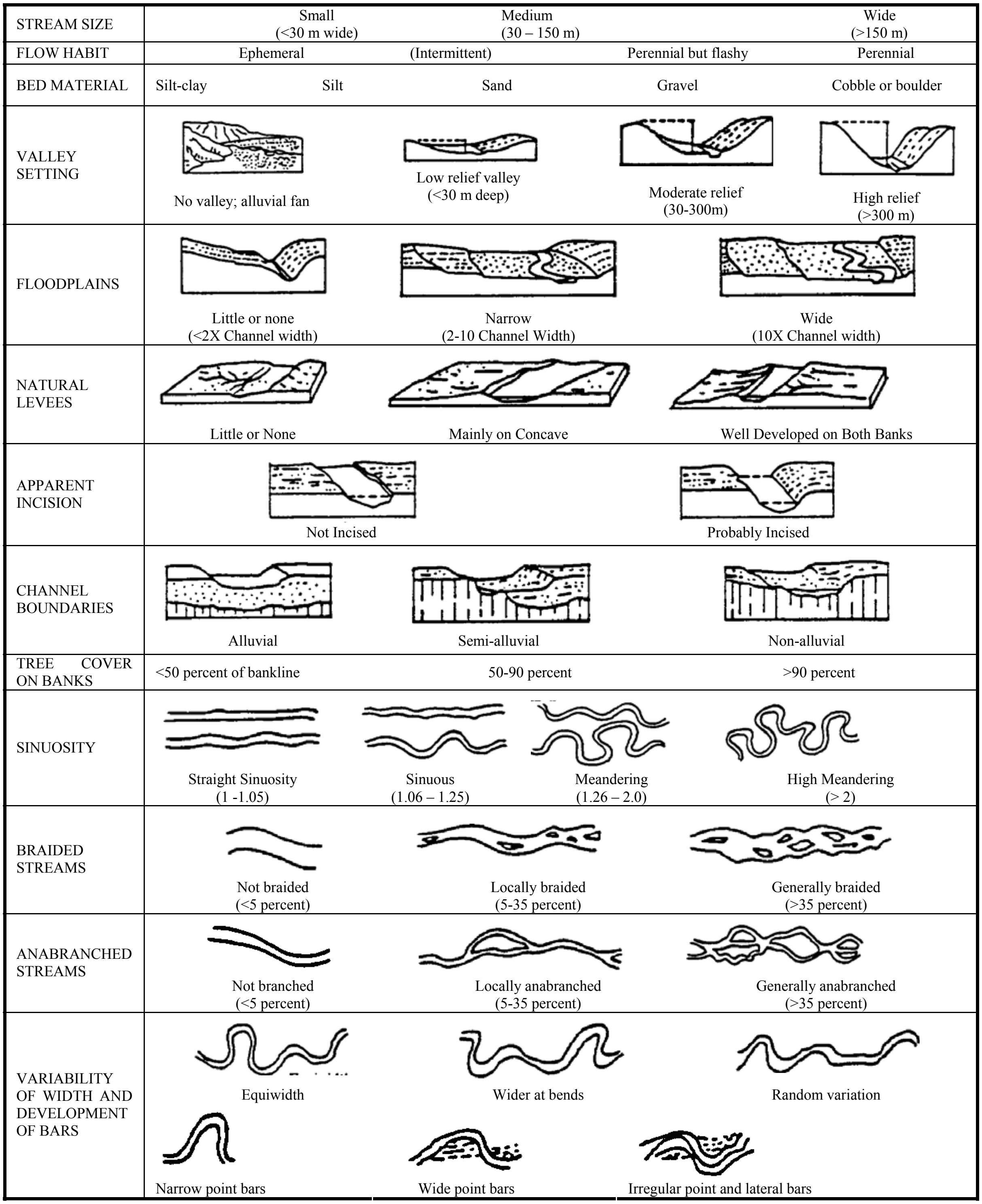


Figure 3: Geomorphic Factors Chart



For more information on the above chart see *Countermeasures for Hydraulic Problems at Bridges, Volume 1 Analysis and Assessment* (FHWA) (HEC 23).

PART 2: REQUIREMENTS FOR ROADWAY DRAINAGE

2.1 Analysis Procedures

Roadway drainage will be designed with procedures to comply with the requirements of the Stormwater Management and Sediment and Erosion Control regulations 72-405 and the NPDES General Permit for Stormwater Discharges from Large and Small Construction Activity (NPDES Construction General Permit). These procedures are outlined below. For information on alternate pipe designs see Instructional Bulletin 2009-4, or latest revision. A formal study report with a title sheet and an index page will be prepared and both signed and sealed by a registered professional engineer of the State of South Carolina as indicated in Part 1. The title and date of the edition of the *Requirements for Hydraulic Design Studies*, used for design purposes, shall be shown on the title sheet of plans and included on the cover page of the "Hydraulic Design Study Report." All forms contained in this document are also located on the SCDOT Intranet/Internet pages. The forms on the Intranet/Internet are updated regularly and take precedence over those in this document. A sample title sheet is shown in Section 1.6.1. The report will also include a topographic layout map, soils information, hydrologic and hydraulic design computations, computer printouts, a plot of the hydraulic grade line for all storm sewers, and all other pertinent information supporting the designed systems.

The design guidelines give general procedures that should be followed in developing drainage designs. However, each hydraulic problem is unique and must be solved by the hydraulic design engineer using the design approach that best meets the particular site conditions involved.

2.2 Design Criteria

2.2.1 Freeboard for Road Subgrades.

To protect the pavement, it is recommended that road subgrades be 1.0 foot above the design high-water level.

2.2.2 Cross-Line Pipes.

The design discharge for all cross-line pipes for primary roads (SC or US designation) and interstate routes is the 50-year peak discharge. For secondary roads, the design discharge for cross-line pipes is the 25-year peak discharge. The designer should analyze the 100-year or overtopping flood, whichever is less. This analysis does not change the design criteria.

2.2.3 Storm Drains and Roadside Ditches.

The design storm for storm drain systems and roadside ditches is the 10-year storm for drainage areas from 0 to 40 acres, the 25-year storm for drainage areas from 40 to 500 acres, and the 50-year storm for drainage areas greater than 500 acres.

2.2.4 Inlet Spacing.

Inlet spacing will be based on the spread criteria in the AASHTO Model Drainage Manual as modified below. For Type 16, 17, and 18 inlets, refer to the Department's website for spacing charts. Recommended maximum spacing is 900 feet and recommended minimum spacing is 150 feet unless specified by the hydraulic design engineer. A 100-foot spacing will be used at sag points to flank the low point in the roadway. Spread criteria can be found in Table 2.

Table 2: Spread Criteria

Roadway Classification		Design Frequency	Design Spread
High Volume	≤45 mph	10 year	Shoulder + 3ft
	≥45 mph	10 year	Shoulder
	*Sag point	50 year	Shoulder + 3ft
Collector	≤45 mph	10 year	½ driving lane
	≥45 mph	10 year	Shoulder
	*Sag point	10 year	½ driving lane
Local Streets	Low ADT	5 year	½ driving lane
	High ADT	10 year	½ driving lane
	*Sag point	10 year	½ driving lane

Note: This criteria applies to shoulder widths of 6 feet or greater. Where shoulder widths are less than 6 feet, a minimum design of 6 feet should be considered.
*For sag points in cuts where the inlets are the only drainage outlet.

Inlets in grassed medians will be spaced so that the 10-year stormwater level in the median will be below the edge of the shoulder. Maximum inlet spacing will be 750 feet.

2.2.5 Minimum Ditch and Pipe Grades.
Minimum grade on ditches, gutters, and pipes in a storm drainage system is recommended to be 0.3 percent where possible. The recommended minimum velocity for the design discharge in a pipe should be 3.0 feet per second. This will promote self-cleaning of the pipe. The controlling factor is velocity rather than grade.

2.2.6 Minimum Pipe Size.
Minimum pipe size in storm drainage systems and for cross-lines is 18 inches. A 15-inch pipe may be used to connect yard drains to a storm drainage system and for driveway pipes.

2.2.7 Minimum Cover for Pipes.
Consideration should be given to the type of inlet or manhole into which the pipe is connecting. For minimum cover requirements refer to SCDOT's *Standard Drawings for Road Construction.*

2.2.8 Precast Manholes.
Precast manholes and basins will be used for the following conditions:

- Where the depth is greater than 12.0 feet
- Where the flow line elevation of the inlet pipe is higher than the soffit of the outlet pipe
- At the engineer's discretion

2.2.9 Storm Drain Systems.

Storm drain systems will be designed for free surface flow. Design flow depths in pipes should equal to 0.94 times the pipe diameter for maximum free surface flow capacity. Storm sewer systems should not be designed for pressure flow.

Table 3 of Manning's roughness coefficients for various pipe types and sizes should be used to assist in the design of storm drain systems. The hydraulic designer should check with vendors to determine if size of certain types of pipe are available.

2.2.10 Process for Selecting Alternate Pipe Types to Include in Final Construction Plans.

A methodology for selecting alternate pipe types can be found in Instructional Bulletin 2009-4. The system of pipes may consist of either "smooth wall pipe" or "corrugated wall pipe." "Smooth wall pipe" includes reinforced concrete pipe (RCP), high density polyethylene pipe (HDPE), and spiral ribbed aluminum pipe (SRAP). "Corrugated wall pipe" includes corrugated aluminum alloy pipe (CAAP).

2.2.11 Outlet Protection for Culverts.

All culvert outlets will be investigated for scour potential using the methods in HEC-14. If there is a potential for scour on the design storm, appropriate outlet protection will be designed. This may be riprap channel lining or an energy dissipator depending on the Froude number of the flow at the outlet of the culvert and the soil type in the channel bottom and banks.

The National Resource Conservation Service (NRCS) Soil Surveys contain descriptions of soil types and their respective engineering properties. Soils with 'K' values higher than 0.4 are highly erodible and will require outlet protection. Soils with 'K' values 0.25-0.4 are moderately erodible and may also require outlet protection. Additional soils information may be found at: http://www.sc.nrcs.usda.gov/soils_x.html

2.2.12 Drainage Outfalls and Stormwater Management.

Outfalls will be designed using the following criteria:

- Outfalls that are to be constructed channels shall be designed for the road drainage design storm.
- For drainage outfalls that are natural watercourses, no modifications will be made to the channel except as necessary to prevent scour or erosion, accommodate highway drainage structures, or to provide positive drainage.
- For projects with 1 or more acres of disturbed area, a formal stormwater management design study will be prepared as a part of a NPDES study report. A number of community ordinances have stricter requirements. Although the Department is a State level government agency and does not have to comply with local ordinances that are from a lower level of government, Departmental policy is to comply with them if it is practical and economical.
- Other requirements may apply to the State's eight coastal counties (Beaufort, Berkeley, Charleston, Colleton, Dorchester, Georgetown, Horry, and Jasper). For instance, the disturbed area threshold drops to ½ acre within ½ mile of a receiving water body. SCDHEC-

OCRM requirements should be reviewed prior to proceeding with final design in these coastal counties.

Table 3: Manning's Roughness Coefficients

Manning's Roughness Coefficients ("n" values) for Various Pipe Types and Sizes		
+Corrugated Aluminum Alloy Pipe (Corrugated Wall Pipe)		
Corrugation	**Nominal Diameter (in.)**	**Manning's "n" Value**
2 2/3" x 1/2"	12	*
2 2/3" x 1/2"	15	*
2 2/3" x 1/2"	18	0.015
2 2/3" x 1/2"	24	0.015
3" x 1"	30	0.024
3" x 1"	36	0.024
3" x 1"	42	0.024
3" x 1"	48	0.024
3" x 1"	54	0.024
3" x 1"	60	0.024
3" x 1"	66	0.027
3" x 1"	72	0.027
3" x 1"	78	0.027
3" x 1"	84	0.027
3" x 1"	90	0.027
3" x 1"	96	0.027
3" x 1"	108	0.027
3" x 1"	120	0.027

++High Density Polyethylene Pipe – Type S, Spiral Ribbed Aluminum Pipe, and Reinforced Concrete Pipe (Smooth Wall Pipe)	
Nominal Diameter (in.)	**Manning's "n" Value**
12	*
15	*
18	0.012
24	0.012
30	0.012
36	0.012
42	0.012
48	0.012
54	0.012
60	0.012
66	0.012
72	0.012
78	0.012
90	0.012
96	0.012
108	0.012
120	0.012

*See Manufacturer – For Manning's "n" Value
+ See Standard Drawings 714-810-01, and 714-810-02 for Fill Height details, etc.
++ See Standard Dravings 714-205-01, 714-205-02, 714-605-01, 714-605-02, and 714-705-01 for Fill Height details, etc.

2.2.12.1 Drainage Regulation Violations by Others.

Where Departmental engineering personnel suspect that landowners adjacent to highway right of way or on the outfall channel have violated the laws or regulations governing drainage and such violation is detrimental to the Department, the suspected violations shall be reported to the Department's Legal Division for consideration as to whether legal action is advisable. A violation is considered detrimental if it causes damage to Departmental property, if it violates the Department's MS4 permit, or if it would increase the Department's liability for damage to another's property. All reports of suspected violations shall be supported by documentation verifying the violation, including reference to the specific law or regulation violated.

2.2.12.2 Basic Design Requirements:

- For those areas where stormwater management is required, determine the 10-year pre- and post-construction peak discharges for each outfall point. The outfall channel will be evaluated for both discharges to determine the effects of the proposed construction. If there is potential for damage to property from flooding, stormwater management procedures will be initiated to minimize the damage.
- The outfall channel should be analyzed with the design discharge to determine (a) that there is no anticipated damage caused to property and (b) the channel is stable. If there is potential for property damage, (1) the channel will be improved, (2) detention storage will be designed to prevent property damage, or (3) a combination of channel improvement and detention will be used to prevent property damage. If the channel is unstable, protective measures will be designed using the design methods in FHWA's Hydraulic Engineering Circulars HEC-15 or HEC-11.
- Detention may be used to address issues of water quantity and/or water quality. Specifically, a detention pond may be required by 72-405 or local ordinances. Careful consideration should be given to downstream impacts of the design and to safety issues.
- When a permanent detention pond is proposed, an analysis of the effects on the outfall shall be made for the 10- and the 100-year storm events. The analysis shall include hydrologic and hydraulic calculations necessary to determine the impact of hydrograph timing caused by the proposed land disturbing activity, with and without the pond. The results will be used to assess the design. If there are adverse downstream impacts, then consideration should be given to redesigning or eliminating the pond. The issue of timing of peaks should be considered starting downstream to where the drainage area to the pond is less than 10 percent of the total watershed area. Consideration should be given to the downstream effects of placing larger openings under the roadway.
- At sites where existing cross-line pipes are undersized and must be enlarged to meet design standards, it will be determined why the existing pipe is undersized. If the cause is due to upstream development, an investigation will be conducted to determine if any drainage law or regulation has been violated. Documentation of violations will be furnished to the Department's legal staff for consideration of possible legal action.
- If the downstream channel has been blocked or restricted by downstream property owners and it appears that violations of State drainage laws or State or Federal regulations have occurred, documentation of the violation will be made and forwarded to the Department's legal staff for consideration of possible legal action.

- If the outfall does not meet the South Carolina legal definition of a water course, as defined in the *SCDOT's Responsibility for Maintenance of Outfall Ditches in Municipalities (Reference 47)* manual and the natural path downstream has been blocked by a land owner, one of the following actions should be taken:

  1. An alternate outfall should be located,
  2. An alternate outfall route around the blockage should be located,
  3. Right of way negotiations should be conducted to locate the outfall through the blockage, or
  4. Failing on the preceding, condemnation proceedings should be initiated to locate the outfall in its natural location.

- When documentation of violations is prepared, it should consist of photographs; videos; aerial photography, preferably showing before and after conditions; engineering studies; maps with the violation delineated; documentation of conversations with local residents; and plotted surveys. The specific law or regulation being violated should be referenced.

2.2.13 Stormwater Management for "C" Projects.
All "C" projects must be designed in accordance with this manual and must have a Stormwater Pollution Prevention Plan (SWPPP) prior to construction.

2.2.14 Sediment and Erosion Control.
All projects must meet the requirements of the SCDOT NPDES MS4 permit and South Carolina Regulation 72-405, as applicable. For sediment and erosion control and other water quality best management practices design criteria, refer to the SCDOT Stormwater Quality Design Manual and the SCDOT Supplemental Technical Specifications, latest revisions.

2.2.15 Hydrologic Analysis.
A hydrologic analysis should be performed using the appropriate method as described below. If the recommended method is not used, prior approval should be obtained from the Department's hydraulic engineer.

When the time of concentration is used, it will be computed using the SCS method described in the WinTR-55 manual. The minimum time of concentration is 5 minutes. Rainfall intensities will be determined from the values in the rainfall depth tables published by the SCDHEC.

2.2.15.1 Rational Method.
For drainage areas up to 100 acres, the rational method using the modifications shown below should be used:

$$Q = CIAC_f$$

Where: Q = discharge in cubic feet per second (cfs)
C = the runoff coefficient
I = the rainfall intensity in inches per hour
A = the drainage area in acres

C_f is defined by:

Recurrence Interval (Years)	C_f
2 -10	1.0
25	1.1
50	1.2
100	1.25

Runoff factors can be seen in Table 4.

Table 4: Runoff Factors for Rational Method

RUNOFF FACTORS FOR RATIONAL METHOD			
	Flat	**Rolling**	**Hilly**
	0% - 2%	**2% - 10%**	**Over 10%**
Pavements & Roofs	0.90	0.90	0.90
Earth shoulders	0.50	0.50	0.50
Drives & Walks	0.75	0.80	0.85
Gravel Pavements	0.50	0.55	0.60
City Business Areas	0.80	0.85	0.85
Unpaved Road, Sandy Soils	0.34	0.45	0.59
Unpaved Road, Silty Soils	0.35	0.47	0.61
Unpaved Road, Clay Soils	0.40	0.53	0.69
Apartment Dwelling Areas	0.50	0.60	0.70
Suburban, Normal Residential	0.45	0.50	0.55
Dense Residential Sections	0.60	0.65	0.70
Lawns, Sandy Soils	0.10	0.15	0.20
Lawns, Heavy Soils	0.17	0.22	0.35
Grass Shoulders	0.25	0.25	0.25
Side Slopes, Earth	0.60	0.60	0.60
Side Slopes, Turf	0.30	0.30	0.30
Median Areas, Turf	0.25	0.30	0.30
Cultivated Land, Clay & Loam	0.50	0.55	0.60
Cultivated Land, Sand & Gravel	0.25	0.30	0.35
Industrial Areas, Light	0.50	0.70	0.80
Industrial Areas, Heavy	0.60	0.80	0.90
Parks & Cemeteries	0.10	0.15	0.25
Playgrounds	0.20	0.25	0.30
Woodland & Forest	0.10	0.15	0.20
Meadows & Pasture Land	0.25	0.30	0.35
Unimproved Areas	0.10	0.20	0.30
Rail Yards	0.25	0.30	NA
Expressways & Freeways *	0.60*	0.70*	0.75*

* The designer can also calculate weighted 'C' values for expressways and freeways using the values in the table for pavement, side slopes and planted medians.

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2.2.15.2 The NRCS WinTR-55 Method.

Use the modified NRCS WinTR-55 method for the conditions listed in Table 5. The time to peak (Tp) should be determined by using the NCRS velocity (segmental) method.

Table 5: NRCS WinTR-55 Method

Physiographic Region	Drainage Area in Acres
Lower Coastal Plain	100 - 640
Upper Coastal Plain	100 - 640
*Piedmont	100 - 640
Blue Ridge	100 - 640

The standard Peak Rate Factor (484) should be used. Some areas of the State, such as the Lower Coastal Plain and Blue Ridge, may warrant the use of a different Peak Rate Factor. For additional guidance, refer to Table 6 below and references 22, 23, and 24.

Table 6: Hydrograph Peaking Factors and Recession Limb Rations

General Description	Peaking Factor	Limb Ratio (Recession to Rising)
Urban areas; steep slopes	575	1.25
Typical SCS	484	1.67
Mixed urban/rural	400	2.25
Rural, rolling hills	300	3.33
Rural, slight slopes	200	5.5
Rural, very flat	100	12.0

For more information on the above Table, see Reference 52.

2.2.15.3 USGS Regression Equations.

For rural drainage areas greater than the limits listed below, use the USGS Rural Regression Equations (Reference 5, latest revision).

NOTE: At the time of this revision, the USGS is rewriting the Rural Regression Equations.

Table 7: USGS Regression Equation

Physiographic Region	Drainage Area in Square Miles
Lower Coastal Plain	1.0
Upper Coastal Plain	1.0
Piedmont	1.0
Blue Ridge	1.0

2.2.15.4 USGS Urban Peak-Discharge Frequency Method.
For urban drainage areas greater than the limits listed under Section 2.2.15.2., use the USGS Regression Equations as modified by the method described in *Determination of Flood Hydrographs for Streams in South Carolina: Volume 2, Estimation of Peak-Discharge Frequency, Run-off Volumes, and Flood Hydrographs for Urban Watersheds*, latest edition, Reference 20.

2.2.15.5 Log Pearson Type III.
Where the stream has or has had a USGS gage, a Log Pearson Type III analysis of the gage data will be performed and the results regionalized using the method described in Reference 5.

2.2.15.6 Hydrograph Methods.
If a hydrograph is needed, use the methods in Reference 19 for rural drainage areas and Reference 20 for urban drainage areas that exceed the minimum drainage areas listed in Section 2.2.15.3. For smaller drainage areas, use one of the acceptable hydrographic computer models listed at the end of these requirements.

2.3 Hydrologic and Hydraulic Design for Stormwater Management
All roadway drainage will be designed using stormwater management procedures as defined by the *Stormwater Management and Sediment and Erosion Control Regulations* (Reference 26). The "Hydraulic Design Study Report" will be prepared in a specific format outlined below in the step-by-step procedure. Full detail on the design procedure is not included in this document. For more detailed information, refer to the AASHTO Model Drainage Manual and the AASHTO Highway Drainage Guidelines.

<u>Step 1</u> Scoping Review
The hydraulic design engineer's first involvement on a project is to participate in the preliminary field scoping review that is used to define the project. Prior to this review, the hydraulic design engineer should obtain a location map showing the extent of the project and, if the project is along an existing roadway, a copy of the existing road plans. The hydraulic design engineer should also determine if the project area lies within an identified floodplain and floodway on the Flood Insurance Rate Maps. The hydraulic design engineer should transfer this information to a copy of a topographic map of the area. Major outfalls and stream crossings should be identified. On the field review, the hydraulic design engineer will identify potential hydraulic problem areas

and make recommendations for possible improvements in road alignment from the drainage standpoint. The hydraulic design engineer will also identify and lay out the survey requirements necessary for the drainage design process. Particular attention should be given to outfalls that will require detailed studies to insure that surveys and studies are conducted early on to avoid delays in the project schedule.

Step 2 Gathering Data

Basic information gathering or research for the study should be carried out during this step. This step should begin after the survey is completed and preliminary plans have been furnished to the hydraulic design engineer by the road design engineer.

1. Obtain all available data in the central office:
   - A copy of plans for existing roads that will impact the project area.
   - All survey data on the job. Most of this data should be plotted on the road plan and profile sheets and on the cross-section sheets. Special emphasis should be given to the determination of the locations and elevations of all underground utilities that might conflict with any proposed storm drain system. Both the plans and survey information should be available on CADD files.
   - Aerial photography coverage.

2. Prepare a map of the roadway using Microstation by superimposing the roadway data over a USGS topographic map or an aerial photograph of the area.

3. Determine from the FIRM maps referenced in the scoping phase if there is any involvement in floodways or special flood hazard areas. If there is involvement, a copy of the appropriate FIRM should be included as part of the study report.

Step 3 Preliminary Drainage Design

1. Determine drainage areas and runoff coefficients for each sub-area or inlet. Mark the drainage areas on the topographic map using the contours and information from highway plans. Determine if water quality issues need to be addressed.

2. For curb and gutter sections, lay out a preliminary drainage system including locations of critical catch basins including locations of proposed yard drains per cross-section data.

Step 4 Field Inspection

When preliminary drainage design is complete, a detailed field inspection is performed. The maintenance engineer for the county involved should be a member of the field inspection team. The maintenance engineer is usually familiar with local drainage conditions and problems. To complete a detailed field inspection, the following steps are necessary:

1. Verify the drainage area boundaries.
2. Determine land usage throughout the drainage area. From this information, the rational "C" runoff coefficients or SCS curve numbers can be determined.

3. Determine Manning's "n" values for outfall channels.
4. Each outfall should be inspected to determine the condition of the outfall, including channel stability, potential flooding problems for adjacent properties, and any existing constrictions.
5. Locate additional yard drains.
6. Locate sites for sediment control ponds and for detention ponds, if required.
7. Verify or adjust the locations of catch basins.

Step 5 Complete Drainage Designs

A. Closed Systems with Curb and Gutter Section.

The procedures in this step are to be followed if the roadway surface drainage is to be contained in an enclosed storm drain system. The computer program GEOPAK Drainage will be used for storm sewer design. If the storm sewer system must be evaluated by routing, XP-SWMM or SWMM computer programs should be used. Before using any other computer programs for storm sewer analysis or design, obtain concurrence from the Department's hydraulic engineer managing the project.

The rational method is used to determine the flow rates for storm drain systems provided the drainage area does not exceed 100 acres. For drainage areas exceeding 100 acres, TR-55 procedures should be used. In special situations where storage is a factor, routing procedures should be used. The modified WinTR-55 method will be used to develop inflow hydrographs. To complete a drainage design, use the following steps:

1. Check the locations of catch basins to determine if the design spread criteria from Section 2.2.4 is met. When designing type 16, 17, and 18 catch basins, use the performance graphs located on the DOT website. These charts were developed for the 10-year storm using a runoff coefficient for paved areas and a 5 minute time of concentration. As new catch basin types are added by the Department, the appropriate graphs should be used. Adjust the locations as necessary to meet the design criteria and physical constraints. Adjust the preliminary layout of the storm drain system in GEOPAK or on the plans as necessary in accordance with the findings of the field inspection. Determine the time of concentration of each inlet.

2. Evaluate the performance of the proposed system using one of the recommended computer programs. The pipes should be set with a minimum cover of 1.5 feet and all the soffits or inside tops of pipes at each junction box should be set at the same elevation unless the junction is to be used to dissipate energy through a drop. The programs will set the sizes and flow lines of the pipes necessary to carry the water based on pipe capacity and friction losses.

3. The next step is to compute junction losses using the method described in Reference 4. The hydraulic grade lines are computed and plotted on a profile of the storm sewer system. On this drawing, the flow lines, the pipe soffits, junction boxes, and the ground line at the junction boxes should be plotted. The energy grade line should also be shown. Based on the results of these computations and using sound engineering judgment, adjust pipe sizes and

flow lines as necessary and re-evaluate. A detailed description of the hand procedure is included in the AASHTO Drainage Manual.

The programs will need the drainage area, runoff coefficient, and the time of concentration at each inlet. They will compute the discharge for the entire system using the rational method. The time of concentration at each junction will be based on the longest travel time for runoff from the drainage area at that point in the system.

4. Evaluate the effects of the proposed construction on the flood discharges of the outfall as specified in sections 2.2.11 through 2.2.12.2.

5. Determine if the velocities for the design storm will cause scour at the outlet of the storm drainage structure. If erosive conditions are encountered, appropriate protection should be designed. Compute the Froude number at the outlet to determine if the flow is subcritical or supercritical. If the flow is subcritical, a riprap pad or channel lining should be used at the outlet. The extent of the riprap should be determined by Figure 10.4 (July 2006 Version) in HEC-14. An appropriate type of energy dissipator should be designed for supercritical flow conditions. The type will be based on the tailwater conditions and the Froude number. HEC-14 provides design details and tailwater requirements for energy dissipators.

   The outfall channel should be protected from erosive velocities from the design storm. The procedures for this determination are in HEC-11 and HEC-15. The HYCHL routine in HYDRAIN provides a good analysis for this procedure. Appropriate protection can be designed using HEC-11, HEC-15, and the computer program HYCHL.

B. Design Ditch Section.

The procedures in this step are to be followed if the roadway drainage is to be handled by a ditch section. Determine the design discharge using the rational method. Design the ditch, assuming a uniform slope, with Manning's equation. Ditch design includes selecting the appropriate cross section and channel lining for the ditch. At the design discharge, it is recommended that the water elevation be a minimum of 1.0 foot below the subgrade of the road, if possible. The lining should be selected to stabilize the ditch and meet the requirements for sediment and erosion control.

It is recommended to use the HEC-15 program written in Excel by the author of the HEC-15 manual to design a stable channel. In addition to HEC-15, HEC-11 should be used for design of some types of lining. Preferred channel lining materials in order of preference from the hydraulic design standpoint are:

1. Grass lining
2. Erosion control blanket
3. Turf reinforcement mat
4. Riprap
5. Wire enclosed rock called gabions and mattresses
6. Asphalt paving
7. Articulated pre-cast blocks

As indicated in the Stormwater Management and Sediment and Erosion Control Regulations, the object of the regulations is to control the quality and quantity of runoff from construction sites during and after construction. One of the best methods to improve the quality of runoff is to filter it through grass or other vegetative matter. The grass on shoulders, road fills, and in the roadside ditches will provide the filtering for roadway surface runoff. For additional water quality BMPs, refer to the SCDOT Stormwater Quality Design Manual.

The designer should work with the maintenance engineer to arrive at an acceptable design.

Step 6 Design Cross-Line Drainage
Cross-line drainage provides passage beneath the roadway for drainage originating off-site. The structures discussed in this section range from 18-inch pipes up to but not including bridge-sized box culverts. Design procedures for bridge-sized culverts are contained in Section 1. Pipes and box culverts are all called culverts in that they have the same cross section throughout and usually will be on a constant grade. The amount of effort and detail included in the design should be commensurate with the size and cost of the structure. The process should be fully documented following the design procedure outlined below:

A. Determine Basic Data Needed for Design.

1. Obtain plots of all survey data including:
   - Road plan and profile sheets
   - Road cross sections with road templates plotted
   - Stream traverse and profile and cross sections

2. Obtain the best topographic mapping available for the location. Plot the site location and outline the drainage area on the map.

3. Obtain aerial photographs covering the drainage area.

4. Conduct an on-site inspection to determine the following information:

   - Verify the drainage area boundaries
   - Determine land usage and runoff coefficients
   - Determine Manning's roughness "n" values for the channel and floodplain
   - Determine the condition and capacity of existing drainage structures at the site and upstream and downstream
   - Evaluate the channel stability
   - Identify any property that might be potentially damaged by flooding and request survey of elevations of property that may be affected - this information may be used to set the allowable headwater elevation for the cross-line structure

5. Determine if the stream is a floodway or a flood hazard area.

6. Through local planning agencies, determine if there are any plans to develop the drainage area upstream.

B. Compute Design Discharge.
Compute the design discharge using the method that corresponds to drainage area size and land use in the drainage basin.

C. Develop Rating Curve for Stream Channel.
Use HEC-RAS to compute the stream high water profile. If the channel is a floodway, use the original data from the floodway study. Modify the data with surveyed data from the stream traverse. If the stream is a designated floodway, the FEMA study requirements are the same as for bridges. The limits of the profile computation reach should be determined using the procedure in Reference 44 as indicated in Section 1.3.1, Step 2, B.

D. Design Culvert.
Design the culvert structure using the principles given in FHWA's Hydraulic Design Series No. 5. The computer program HY-8 performs the analysis for this design and is the method required by the Department for this type of structure. Steps to design the culvert are as follows:

1. Select the allowable headwater elevation based on:
   - preventing potential damage to upstream property
   - the headwater must be at least 1.0 foot below the subgrade of the roadway
   - meet FEMA No Impact or Conditional Letter of Map Revision requirements
   - design head should be limited to 1.2 times the height of the culvert barrel

   The allowable headwater elevation will be based on the controlling requirement of the above limitations.

2. Locate the structure so it will be lined up with the approach channel. The outlet should fall in the downstream channel. It may be necessary to have a bend in the culvert to match the outlet channel. If so, use a circular curve for box culverts rather than an angle to achieve the bend. Pipes may have several bend sections to achieve the appropriate bend or a junction box. Appropriate bend losses should be added to the headwater computations.

3. Set the flow line of the circular culvert close to the bottom of the natural channel. The flow line of box culverts should be set 1.0 foot below the channel bottom. In doing this, the conveyance should be evaluated on a site by site basis. If the culvert is in inlet control, an improved inlet should be considered.

4. Select a trial size and type of structure. Analyze the structure using HY-8. If the computed headwater elevation is not close to the allowable headwater elevation, select another size and/or type and re-evaluate the design. Continue until all design requirements are met.

5. Using the outlet velocity computed by the HY-8 design run, determine if the culvert needs outlet protection. The required protection will be based on whether the flow is subcritical or supercritical. Riprap channel lining may be used for subcritical flow and an energy dissipator should be used for supercritical flow. The design procedures are the same as for storm drains.

E. Exceptions to Normal Design.

There are two major exceptions to the conventional design described above. The one most often encountered occurs with pipe culverts when the channel has a steep slope and the road is a divided highway. The Department will frequently use a drop structure in the median to reduce the outlet velocity of the culvert. The upstream half of the pipe culvert will be constructed on a moderate slope to the median. A drop box, usually a precast manhole, will be used to drop the water so that the outlet pipe can be constructed on a mild or subcritical slope. Each section of the culvert will be analyzed independently.

The other exception to the normal culvert design is a broken back box culvert, a culvert constructed with a change in slope part of the way through the barrel. The most often used broken back culvert has the steeper slope at the entrance of the structure. The analysis for this type of structure involves some manipulation of the HY-8 program output. The culvert is analyzed for the downstream portion with the flatter slope. A rating is developed for the bend section by subtracting the entrance loss and the velocity head from the computed headwater elevation. The upstream or steeper portion is then analyzed with this rating as the tailwater.

For the broken back culvert with the steeper slope on the outlet, the analysis depends on whether the steeper slope is subcritical or supercritical. If it is subcritical, the analysis will be as described above. If the downstream slope is supercritical, the headwater is computed by analyzing the upstream portion of the culvert starting at critical depth at the bend. Outlet velocities are computed by analyzing the steeper portion as an independent culvert using HY-8. It may also be analyzed as an open channel using a direct step backwater analysis. If the steep section is sufficiently long, normal depth can be used to determine outlet velocity.

F. Culvert as Stormwater Management Control Structure.

In some cases and particularly when the upstream floodplain is wide or on a flat slope, the culvert may act as a flood control structure. It may be desirable from a stormwater management perspective to use the culvert in this manner. By taking advantage of the storage, the culvert size can be reduced. If these situations are present, the culvert should be analyzed by routing the storm hydrograph through the structure. HY-8 has this built-in feature. Consideration should be given to how the reduction and shifting of the hydrograph peak will affect flooding both upstream and downstream.

G. Information to be Shown on Plans for Cross Line Structures (For Pipes 48”or Larger).

- Show the design, 100-year, and historic headwater elevations
- Show the hydrology data in the format given in Form 4.

<u>Step 7</u> Prepare Hydraulic Design Study Report
Although it is placed last in this procedure, the design study report should be initiated early in the study process and be an integral part of the entire process. Each design study report shall have a title sheet signed and sealed by a registered professional engineer of the State of South Carolina. The report will include an index listing all the separate items in the study. The index will also be signed and sealed by the same professional engineer. The cover page of the "Hydraulic Design Study Report" shall contain the date of the "Requirements for Hydraulic Design Studies" used in the design.

All design studies for roadway drainage must contain some basic information to comply with the Stormwater Management regulations and the NPDES permit regulations. The following will be listed on the report index:

1. A topographic map using the best available topographic mapping. For "C" type projects and bridge replacement projects, the road location will be indicated on the map unless there is extensive storm sewer on the project. For these exceptions and all other projects, the plan view of the road plans will be reduced to the same scale as the map and superimposed on the map. The drainage areas should be outlined on the map. The outfalls for road drainage and storm drain systems should be highlighted. The highlighting should extend to a named stream.

2. A narrative description of the project summarizing what was done in the design, what decisions were made, and how they were made should be included. Also, the total disturbed area in acres should be given in the narrative. The road designer should furnish this information. The outfall stream names should be listed. Runoff coefficients for the area within the construction limits should be given for pre- and post-construction conditions.

3. Summary tables should be provided giving the design of the drainage systems, outfalls, and sediment and erosion control features.

4. For cross-lines, if there are upstream facilities that may be affected by backwater, include a stream profile showing the natural water surface and the backwater profile for the design flood and the 100-year flood.

2.4 Water Quality and Sediment and Erosion Control
For information on appropriate design considerations, methods, and BMPs to address the quality of stormwater runoff and sediment and erosion control, refer to SCDOT Stormwater Quality Design Manual.

2.4.1 Hydrology Data for 48" Diameter or Greater Pipe

Hydrology Data:			
D.A.	=	______________________	sq. mi. (or acres)
Q_n	=	______________________	cfs (n = design period)
Q_n-year headwater elev.	=	______________________	ft.
Q_n Outlet vel.	=	______________________	ft. / sec.
Q_{100}	=	______________________	cfs
Q_{100} year headwater elev.	=	______________________	ft.
Q_{100} Outlet vel.	=	______________________	ft. / sec.
Q	=	______________________	cfs
Probability	=	______________________	%

2.5 Title Sheet – Hydraulic Stormwater Management Design Study

STORMWATER MANAGEMENT
DESIGN STUDY
FOR
THE PROPOSED CONSTRUCTION
OF
(Project description)

ROUTE / ROAD NUMBER: ______________________

FILE NO.: ______________________

PROJECT NO.: ______________________

PIN: ______________________

COUNTY NAME: ______________________

DATE: ________ / ________ / ________

PREPARED BY:	______________________
CHECKED BY:	______________________

Hydraulic Design Reference for this study is the : 2009 Edition of SCDOT's "Requirements for Hydraulic Design Studies."	(Place stamp and signature in this space) Signed and Sealed

REFERENCE LIST

1. The Federal Highway Administration's Hydraulic Design Series (HDS).

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LIST OF ACCEPTABLE COMPUTER MODELS AND DESIGN GUIDANCE

DYNLET - *Dynamic Behavior of Tidal Flow at Inlets*
A powerful 1-D hydrodynamic model for riverine, estuarine, or coastal problems.

FESWMS - *Finite Element Surface Water Modeling System*
The FESWMS package consists of two software packages that can model flows in open channels. FST2DH is a two-dimensional finite element surface water computer program that can compute the direction of flow and water surface elevation in a horizontal plane. FST2DH has the ability to model hydraulic structures commonly used by hydraulic engineers. FST1DH is a one-dimensional finite element surface water model (not yet available) that models unsteady flow and sediment transport in open channels.

Geopak Drainage
Geopak Drainage is full-featured storm sewer design and analysis plug-in program for MicroStation.

HEC-1 - *Hydrology of Bridge Waterways*
This program will produce runoff hydrographs for complex watershed networks using unit hydrograph or kinematic wave methods and incorporating reservoir and channel routing procedures. The program will allow various methods for calculating rainfall hyetographs, basin unit hydrographs and watershed loss rates.

HEC-2 - *Highway Hydraulics*
Discusses the physical processes of the hydrologic cycle that are important to highway engineers. These processes include the approaches, methods, and assumptions applied in design and analysis of highway drainage structures.

HEC-11 - *Design of Riprap Revetment*
The manual includes discussions on recognizing erosion potential; erosion mechanisms and riprap failure modes; riprap types including rock riprap, rubble riprap, gabions, preformed blocks, grouted rock, and paved linings. Detailed design guidelines are presented for rock riprap, and design procedures are summarized in charts and examples.

HEC-14 - *Hydraulic Design of Energy Dissipators for Culverts and Channels*
The purpose of this circular is to provide design information for analyzing and mitigating energy dissipation problems at culvert outlets and in open channels.

HEC-15 - *Design of Roadside Channels with Flexible Linings*
Flexible linings provide a means of stabilizing roadside channels. Flexible linings are able to conform to changes in channel shape while maintaining overall lining integrity. Long-term flexible linings such as riprap, gravel, or vegetation (reinforced with synthetic mats or

unreinforced) are suitable for a range of hydraulic conditions. Unreinforced vegetation and many transitional and temporary linings are suited to hydraulic conditions with moderate shear stresses.

HEC-18 - *Evaluating Scour at Bridges*
Namely, the purpose of this document is to provide guidelines for the following: designing new and replacement bridges to resist scour, evaluating existing bridges for vulnerability to scour, inspecting bridges for scour, and improving the state-of-practice of estimating scour at bridges.

HEC-20 - *Stream Stability at Highway Structures*
The HEC-20 manual covers geomorphic and hydraulic factors that affect stream stability and provides a step-by-step analysis procedure for evaluation of stream stability problems. Stream channel classification, stream reconnaissance techniques, and rapid assessment methods for channel stability are summarized. Quantitative techniques for channel stability analysis, including degradation analysis, are provided, and channel restoration concepts are introduced.

HEC-23 - *Bridge Scour and Stream Instability Countermeasures*
The purpose of this document is to identify and provide design guidelines for bridge scour and stream instability countermeasures that have been implemented by various State departments of transportation in the United States.

HEC-RAS - *River Analysis System*
Models the hydraulics of water flow through natural rivers and other channels. The program is one-dimensional, meaning that there is no direct modeling of the hydraulic effect of cross section shape changes, bends, and other two- and three-dimensional aspects of flow.

HY-8 - *Culvert Analysis*
Culvert Analysis automates the design methods described in FHWA publications HDS-5, *Hydraulic Design of Highway Culverts*, HEC-14, *Hydraulic Design of Energy Dissipators for Culverts and Channels*, and HEC-19, *Hydrology*.

Hydraflow Express
Hydraflow Express Extension provides flexible calculators for solving a wide variety of everyday hydraulics and hydrology problems for culverts, open channels, inlets, hydrology and weirs.

Hydraflow Hydrographs
Hydraflow Hydrographs Extension is a comprehensive solution for watershed analysis and detention pond design, from simple sites to complex watersheds.

Hydraflow Storm Sewer
Hydraflow Storm Sewers are an easy-to-use, full-featured storm sewer design and analysis package.

SEDIMOT - *Sedimentology by Distributed Model Treatment*
A tool used for flood and sediment estimation.

SEDCAD - *Sediment, Erosion, Discharge by Computer Aided Design*
Used to design and evaluate surface water, erosion and sediment control systems for surface mining (coal and hard rock), residential subdivisions, commercial properties, landfills (municipal, hazardous and low level nuclear), and linear developments such as highways and utility lines.

SMS - *Surface Water Modeling System*
SMS is a comprehensive environment for one- and two-dimensional models dealing with surface water applications. The hydrodynamic models cover a range of applications including river flow analysis, rural and urban flooding, estuary and inlet modeling, and modeling of large coastal domains. SMS can import data from a variety of files including text, CAD, and GIS files.

UNET
UNET simulates one-dimensional unsteady flow through a full network of open channels. In addition to solving the network system, UNET provides the user with the ability to apply several external and internal boundary conditions, including: flow and stage hydrographs; rating curves; gated and uncontrolled spillways; pump stations; bridges; culverts; and levee systems.

WMS - *Watershed Modeling System*
WMS is a graphical modeling environment for all phases of hydrology and hydraulics. WMS has tools to automate modeling processes. WMS supports hydrologic modeling with HEC-1, HEC-HMS, TR-20, TR-55, Rational Method, NFF, MODRAT, OC Rational, HSPF, xpswmm, and EPA-SWMM. Hydraulic models supported included HEC-RAS, xpswmm, EPA-SWMM, SMPDBK, and CE-QUAL-W2.

WSPRO - *water surface profile computation*
Used to analyze one-dimensional, gradually-varied, steady flow in open channels. WSPRO can also be used to analyze flow through bridges and culverts, embankment overflow, and scour at bridges.

Win TR-55
WinTR-55 is a single-event rainfall-runoff small watershed hydrologic model. The model generates hydrographs from both urban and agricultural areas and at selected points along the stream system. Hydrographs are routed downstream through channels and/or reservoirs. Multiple sub-areas can be modeled within the watershed.

XPSWMM - *Storm Water Management Model*
XPSWMM is a software package for modeling stormwater, sanitary and river systems. The model is used for the analysis, design and operation of storm systems. XPSWMM simulates flow transport in engineered and natural systems including ponds, rivers, lakes, floodplains and

the interaction with groundwater. XPSWMM is a dynamic unsteady flow model rather than a steady state or standard step model.

Numerical Models Meeting the SCDOT “Requirements for Hydraulic Design Studies”

Computer Models	Design Components											
	Hydraulics	Hydrology	Bridges / Culverts	Channels / Roadside Ditches	Surface Water	Sediment / Erosion Control	Sediment Transport	Scour	Wind	Tidal	2D	Flooding/ FEMA Encroach-ment
DYNLET	X			X					X	X		
FESWMS	X		X		X	X			X	X	X	
RMA-2V	X			X			X		X	X	X	
HEC-1		X			X							
HEC-2	X		X	X								X
HEC-11	X			X		X		X				
HEC-14	X			X		X		X				
HEC-15	X			X		X						
HEC-18	X		X					X				
HEC-20	X		X	X		X	X	X				
HEC-RAS	X		X	X			X					X
HEC-HMS		X			X							
HY-8	X		X									
SEDIMOT						X						X
SEDCAD					X	X						
SMS		X	X		X		X	X		X	X	X
UNET	X		X	X						X		
WMS	X	X	X									
WSPRO	X		X	X								
XPSWMM	X	X	X	X	X							
BRI-STARS	X		X	X		X		X				
HYCHL	X			X								
Geopak Drainage	X											
Hydraflow Hydrographs		X										
Hydraflow Storm Sewer	X											
Hydraflow Express			X	X								