SOUTH CAROLINA
DEPARTMENT OF TRANSPORTATION

REQUIREMENTS
FOR
HYDRAULIC DESIGN STUDIES

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SOUTH CAROLINA DEPARTMENT OF TRANSPORTATION REQUIREMENTS FOR HYDRAULIC DESIGN STUDIES

Introduction

This document delineates the requirements for hydraulic design studies performed for the South Carolina Department of Transportation (SCDOT). Its purpose is to give guidance to Departmental staff and to consultants performing design work for the Department in the preparation of construction plans. Before the hydraulic design engineer performs a design study utilizing this guideline, he should be thoroughly familiar with the more detailed coverage of the different design procedures given in the manuals in the reference list. Compliance with these procedures will insure that the requirements in the following laws and regulations will be adhered to.

- Federal Highway Administration's Federal-Aid Policy Guide, 23 CRF 650A, December 1, 1991, Transmittal 1, "Bridges, Structures, and Hydraulics",
- The Federal Emergency Management Agency Regulations, 44 CFR Ch.1,
- 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 the 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. The first part is for bridges and bridge-sized culverts (culverts greater than or equal to 20 feet in width 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 twenty feet wide. The second part also includes the guidelines established to insure compliance with the NPDES permit requirements and the State Stormwater Management and Sediment and Erosion Control Act.

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

1. Analysis Procedures

Bridges and bridge-sized culverts 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 will be considered to have 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 Level 3 analysis is needed are:

- Wide floodplains, where curvilinear flow is a major factor and/or multiple-bridges are used.
- Any floodplain where curvilinear flow has a major impact on the bridge.
- Major tidal estuaries.
- In situations where sediment routing is necessary.
- Full risk analysis is required.

The HEC-20 study approach is modified in this document to incorporate the 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 lengths
- Orientation of substructure
- 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 special cases other features may require hydraulic designs. 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 (34), an index, a comparative data sheet (page 35), location maps, site inspection forms (pages 36-40), and a risk assessment (page 41-42). In accordance with South Carolina Code of Laws Title 40, Chapter 22 and Code of Regulations Chapter 49 the title sheet and the index sheet will be signed and sealed by a registered professional engineer.

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.

1.1.2 Floodway - Floodplain Requirements. All floodplain crossings must meet the Federal Emergency Management Agency (FEMA) regulation requirements. If the stream has been designated as a floodway, the structure should be designed, if practical, so that there will be no change in the 100-year flood elevations, floodway elevations, and floodway widths at any cross-section. The 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 in floodway width, rounded to the nearest 1.0 foot, for any cross-section outside the Department’s right of way. Changes greater than 0.1 foot inside the 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 Squad Leader will send one copy to the local community planning commission and one copy will be retained in the Hydraulic Engineering Section’s design files.

If there is a change in the 100-year or floodway profile equal to or greater than 0.1 foot or a change in floodway width equal to or greater than 1.0 foot outside the Department’s right of way, a Conditional Letter of Map Revision (CLOMR) must be prepared using all the appropriate forms. One copy of the CLOMR will be sent to the local community planning commission 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 to 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 is to be submitted to FEMA. The other copy of the CLOMR will be filed in the Hydraulic Engineering Section’s 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 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 so that the hydraulic conditions proposed in the CLOMR, have been met, no hydraulic analysis is necessary. However, if those conditions have been changed, a new hydraulic analysis must be submitted and owners of affected property notified by certified mail.

Bridges in all other floodplains must be constructed so that the 100-year high-water profile will increase no more that 1.0 foot above the existing 100-year flood profile. It is the Department's policy to limit the increase to 1.0 foot above the unrestricted or natural 100-year flood profile, if practical. A letter will be sent to the local community planning commission informing them that a study of the stream has been done and that if they would like to have a copy, they may request it through the Department’s Hydraulic Design Engineer.

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, using the methods in HEC-18. Note: Spread footings should not be used on stream crossings except on firm non-erodible rock. The design flood for scour shall be the 100-year or the overtopping flood if it is less than or equal to the 100-year flood. Maximum scour depths will be produced by the overtopping flood. The bridge foundation should be designed for the normal factor of safety as specified in the AASHTO Standard Specifications for Highway Bridges, most current edition, below the scour depths estimated for the 100-year flood. The footings should be placed below the design flood scour level. Where pile bents are used, the design friction or point bearing should be achieved below the depth of the design scour. There must be sufficient pile penetration below the scour line to provide lateral stability.

Scour should also be computed for the super flood, defined as the 500-year flood, or the overtopping flood if it is greater than the 100-year flood and less than the 500-year flood. The 500-year discharge can be estimated as 1.7 times the magnitude of the 100-year flood if the 500-year discharge cannot be computed by other means. The bridge foundation should have a factor of safety of 1.0 for the scour produced by the super flood.

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

1.1.5.1 Freeboard for Riverine Bridges. There should be a minimum freeboard of 2.0 feet above the design high water for all riverine bridges. For larger streams, the freeboard should be increased to a maximum of 7.0 feet for such rivers 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. Freeboard on other large streams shall be adjusted accordingly. The hydraulic design engineer should evaluate the debris load potential and history of debris accumulation. The hydraulic design engineer, using his best judgement, 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 minimum design freeboard for bridges in these areas will be set at 2.0 feet above the 10-year high-water elevation including wave height. The finished grade of the bridge will be set by considering this requirement, navigation clearances, the approach roadways, topography, and practical engineering judgement.

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 Freeboard for Road Subgrades. To protect the pavement, road subgrades should be 1.0 foot above the design high-water level.

1.1.6 Bridge Abutment Protection. Riprap will be placed on all end fills except where the velocities are well below the threshold for scour as determined by incipient motion analysis. The riprap shall be entrenched 2.0 feet below the ground line and extend to 2.0 feet above the design high-water level. It should be noted that some of the highest velocities experienced in bridges occur around the end fills. Past experience in South Carolina has shown that severe erosion of end fills occurs during high floods, frequently undermining the pavement at the end of the bridge. This creates a hazardous situation with unsupported pavement spanning a gap between the end of the bridge and the remaining fills.

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-2DH should be utilized to evaluate the need for guide banks.

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 can have a maximum headwater depth equal to 1.2 times the height of the barrel (HW/D) providing the FEMA requirements are met. The headwater depth is measured from the entrance invert or the bottom of the culvert opening at the entrance.

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, then evaluates the stream and its geomorphic responses over time. It evaluates in a qualitative way the possible stream responses to the proposed or existing highway structures. The design procedure for the Fast Track Bridge Replacement utilizes 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, page 43. See Chapter 4 of HEC-20 for full details on this procedure.

STEP 1 Stream Characteristics. Describe the stream geomorphic characteristics utilizing the form in Figure No. 3, page 45 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 Department's 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 has aerial photographs dating back almost 60 years. It should be noted that "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 channel is due to flat slopes and thick vegetation on the floodplains. A scour hole resembling a channel, at first glance, will usually form at a bridge site in a swamp. But a more detailed investigation will reveal that the scour hole will only extend roughly 100 feet upstream and 200–300 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 flow in a stream will increase the sediment carrying capacity of the stream, causing stream instability and degradation. Reference 14 and HEC-20 have in-depth discussions on stream responses to changes in sediment supply.

STEP 3 Overall Stability. Assess overall stream stability. Utilize table 6 and figure 20 in HEC-20 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 see 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. A pier on the left bank of the Great Pee Dee River on the southbound lane of I-95 was almost lost for this reason. The channel bank on the Pee Dee was stabilized with old car bodies tied together by cable and anchored to a large concrete anchor. This has proved to be successful. Current EPA regulations would probably not permit this form of bank stabilization because of residual hydrocarbons on the car bodies.

If horizontal channel stability will be 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 a recommended bank stabilization measure is to line the channel bank with synthetic mats and plant grass or other vegetation so that the roots will grow through the mat acting as reinforcement. In some specific cases bioengineering techniques may be utilized. However, caution should be utilized since bioengineering techniques have not had sufficient research to establish ranges of effective use.

STEP 5 **Vertical stability.** Bridges and culverts can be affected by elevation changes of the streambed. Aggradation, a rise in streambed elevation, reduces the openings of both bridges and culverts. This is generally more of a problem to 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 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 causes degradation. Sediment supply can be reduced by urbanization of the drainage area, converting row crops to pasture or forest, construction of a dam upstream, or mining of sand or gravel in the stream bed upstream. The energy grade can be increased by straightening the channel or by mining in the streambed downstream. Channelization of a swamp can also cause degredation.

Location of the bridge or culvert in or near a meander bend can have a combination of both aggradation and degradation. The build up 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.

STEP 6 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 downwards, 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 human 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. Examples are: 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 utilize 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 see if there is any debris accumulation. Any sign of bank instability along the stream is a good indicator of debris potential.

Local maintenance personnel can provide information on the history of debris problems along the stream.

STEP 7 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 II of HEC-20 and Chapter VII of Reference 14 give good discussions of stream responses to change.

At this point in the process, a determination is made whether a more detailed analysis is required or not. 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 to proceed with 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:

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

In order to determine if a bridge meets these criteria, a PS&E team, including an experienced Hydraulic Design Engineer, visits 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 level 2 study shall be performed to handle the FEMA requirements and the project cannot be handled as a fast track bridge. On the site visit the Hydraulic Design Engineer shall, based on his experience and engineering judgement, 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.
- In conjunction with the Bridge Engineer establish the bridge superstructure type, span length, and bridge length.

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 or, if more detailed study is required to determine any of the above information, a Level 2 study must be performed.

1.3 Level 2: Basic Engineering Analysis

Level 2 Analysis involves the basic engineering analysis to hydraulically design a bridge or culvert, to determine the scour depths, and to design necessary counter measures. A diagram of the steps in the procedure is presented in Figure No. 2, page 44. Two different procedures are given below, one for riverine bridges and one for tidal bridges.

1.3.1 Level 2 Procedures for Riverine Bridges. In order to provide coordination with the local communities, when a study is begun, the Hydraulic Design Engineer should inform the local community planning commission that the Department is performing the study. If the planning commission has any input or would like to receive a copy of the study report, they should contact the Hydraulic Design Engineer.

Step 1 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:

1. Road and bridge plans on file in the Department's Central Office will have high-water data and in some cases related discharges indicated on the plan and profile sheets.

2. High-water information on specific flood events is on file in the Department’s Hydraulic Engineering office.

3. Interviews with local residents can produce useful information. The resident's name, address, and the length of time he/she has resided 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. Also, they should be asked when the flood occurred and with what frequency the general area is flooded. If possible, the information should be verified through independent sources. Local Department Maintenance personnel may be able to also furnish similar information.

4. 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.

5. Local newspaper files may have stories on previous floods.

6. Determine if the stream is a designated floodway. If it is, obtain a copy of the original computer program data files used to establish the floodway boundaries.

7. A thorough knowledge of the historic floods in the State is very helpful in evaluating the assembled information. If enough data is available, a stream profile showing all of the data should be developed. The profile can be used to determine flood flow slopes for starting elevations 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:

1. The Bridge Inspection and Maintenance files.

2. A comparison of the original bridge plan and profile with the currently surveyed profile.

3. Aerial photographs taken over as long a time span as available.

4. The history of sand or gravel mining on the stream.

Based on this information, an indication of the long-term channel stability and aggradation or degradation can be estimated. An evaluation of the performance of the existing bridges 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 method used to compute the flows depends on the availability of gage data, the location in the State and topography of the drainage basin. These methods 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 No. 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 checked to make this determination. If there is a gage, compile a complete listing of the annual peaks for the gage. Analyze the data utilizing 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 (Ref. No. 5) to determine the discharge. If the drainage area has urban or developed areas, use the peak discharge method described in Reference 19.

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 taking into account the storage. Unit hydrographs should be developed utilizing the methods in References 22 and 23. The flows should be routed using the Corps of Engineer's routing program, HEC-1. 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, which have drainage areas that range 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 on page 35.

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. Normal water elevation

13. Normal 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.

Normal water is defined as the water elevation observed by the survey party. It is assumed to be a water elevation that is not affected by flooding and reflects the normal condition. It is used to determine the difference between wet and dry excavation for the bridge foundation.

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 the 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 forms, shown on page 40.

This 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.

2. Job Site Inspection. The purpose of the field inspection of 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, utilizing a copy of the geomorphic factor sheet (page 45). If there is an existing bridge, the performance of the bridge should be evaluated using the Job Site Inspection Forms, shown on pages 36 through 39. Manning's "n" values should be determined using the method described in reference 15 and the appropriate forms. The design engineer should walk along the channel both upstream and downstream 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, beaver, or debris), shoals, waterfalls, sewer or water lines suspended across the channel.

A sketch should be made of the site showing the existing structure, the direction of flow, the channel alignment, and anything else which 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 showing 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. 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 designer. It involves an understanding of geology, fluvial geomorphology, hydrology, open channel hydraulics, floodplain plant and animal life, and structural aspects of the bridge. Utilization of all these areas of knowledge combined with a study of aerial photography, topographic mapping and the hydraulic analysis will help the designer 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 "Highways and River Environment", reference 14 present a background in fluvial geomorphology.

B. Hydraulic Analysis. High-water profiles are to be computed for the 2-, 10-, 25-, 50-, 100-, and 500-year floods. If the overtopping flood is less than the 500-year high-water profile, its profile should also be computed. 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 is not a floodway, the high-water profiles will be computed using the computer programs WSPRO or HEC-RAS utilizing the WSPRO 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 that a change in starting elevation will not affect the computed high-water depth at the bridge. The upstream limit should extend to the limit of backwater from the bridge. The backwater is defined as the increase in water surface elevation due to the restriction of flow by the bridge. To determine these distances use the Corps of Engineer's Technical Paper No. 114, *Accuracy of Computed Water Surface Profiles* Reference No.45. This study gives the following regression equations for these distances:

$$Ldc = 6600 \times \frac{HD}{S},$$

$$Ldn = 8000 \times \frac{HD^{0.8}}{S},$$

$$Lu = 10{,}000 \times HD^{0.6} \times \frac{HL^{0.5}}{S}$$

Where: Ldc = downstream study length (along main channel) in feet for critical depth,
Ldn = downstream study length (along main channel) in feet for normal depth starting conditions,
Lu = upstream study length (along main channel) in feet,
HD = average reach hydraulic depth (100-year flow area divided by cross-section top width) in feet,
S = average reach slope in feet per mile, and
HL = backwater or headloss for the bridge or culvert for the 100-year flow.

The model should be calibrated utilizing known flood data if sufficient reliable data is available.

For areas where urbanization has taken place or is expected to take place, the 100-year floodplain will be defined and furnished to the local community planning commission and to the State Coordinator for the National Flood Insurance Program. The 100-year floodplain will be plotted on a topographic map and a profile of the stream showing the channel bottom and the 10-, 50-, 100-, and 500-year high-water profiles. This will be done to indicate to property owners which areas may experience flood damage and to protect the Department from future damage claims for flooding by restricting future development in the floodplain in accordance with FEMA Regulations 44 CFR Part 60.3.

If the 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, and
- Plots of surveys.

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

If the stream is a designated floodway, use the HEC-2 program and data from the floodway study. Under certain conditions FEMA will accept HEC-RAS. The Design Engineer should check with FEMA to see if HEC-RAS is acceptable before using it. The survey data from the project should be superimposed on the HEC-2 data from FEMA. The 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, which are not modeled in the floodway that would change the floodway or impact the bridge.

If the study is for bridge scour only, the floodway re-mapping does not need to be done.

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. By significantly different, it is meant 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.

If the floodway is being changed due to changing conditions other than the roadway improvements and the roadway improvements do not cause any changes, a LOMR will be submitted to FEMA and a "No Impact Certificate" will be submitted to the local community based on the modified floodway.

When a floodway is being remapped, the published floodway boundaries and widths should be matched if reasonable. These boundaries are established by zoning ordinances under the jurisdiction of the local community and effect the flood insurance rates and property development rights of local property owners. To do this, HEC-2 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 is being done in a floodway, a "No Impact" Certificate must be submitted to the local community planning commission or a CLOMR with supporting documentation must be submitted to FEMA and to the local community planning commission. Under the latest FEMA regulations a "No Impact" Certificate can be issued if there is no change in the floodway profile or floodway width at any cross-section. The supporting documentation consists of the following:

- A copy of the original computer run with the floodway table.

- A copy of the original computer run with 10-, 50-, 100-, and 500-year high water 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 run for the 10-, 50-, 100-, and 500-year flood profiles using the modified data.

- A copy of computer runs modeling the proposed construction for the floodway and the 10-, 50-, 100-, and 500-year flood profiles.

- A copy of the published floodway map and the published high water profiles.

For the CLOMR the same supporting documentation should be submitted with:

- documentation of the changes that were made,

- a revised floodway map showing the revised floodway boundaries,

- revised high-water profiles,

- certification of notification of property owners,

- community sign-off sheet, 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. The FEMA studies were done some years ago and do not have the benefit of additional years of record as do the methods described in the section on hydrology in Step 1.C. page 15.

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 been designated as a floodway, a FEMA submittal. A Level 3 analysis, using a two-dimensional model will be required if:

- the floodplain is wide, 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 meander bend, or
- a stream junction of sufficient size to affect the hydraulics.

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 Bridge Design 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 Engineers' sediment routing program can be used for the Level 3 analysis.

STEP 5 Incipient Motion Analysis

An evaluation of the incipient motion of the bed material of the stream can give an indication of the stability of the channel. Procedures for doing this analysis utilizing 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 utilizing 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 will 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 a gage has been on the stream for a period of years, an evaluation of long-term rating curve shifts should be made. This will give some indication of the long-term stability of the stream. A shift in the rating curve implies a change in the bed elevation of the stream or a change in hydraulic control. The USGS will have a record of the long-term rating for the gage. If the bed of the stream is changing in elevation, a more detailed study will be needed.

<u>STEP 8</u> Design Bridge

A. Establish the minimum finished grade based on the 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.

2. Set the span lengths for the bridge. This step should involve consultation with the Bridge Design Section. The span over the channel should be set first. The controlling factors are the potential for debris and the substructure costs. 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.

Substructure costs, for particularly drilled shafts, can be significant.

Using a copy of the bridge plan and profile, locate the spans. 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 Bridge Design Squad Leader.

D. Set the location of the ends of the bridge. 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.

If the bridge is parallel to an existing bridge, the end fills should be aligned. If the existing bridge is too long according to hydraulic requirements, 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 be at least 5.0 feet behind the channel banks. A CADD operator should be able to produce the contour map relatively easily utilizing 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 new location should be based on the length of bridges upstream and downstream. On the plan and profile sheet sketch the end fill slopes from the end of proposed spans, using 2:1 slopes.

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 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. Channel changes are to be avoided if at all practical.

STEP 9 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 soils are cohesive material such as clay, the procedures described by Dr. Jean-Louis Biraud (Referance No. 41) should be utilized. Other references that should be consulted are HEC-20 Stream Stability at Highway Structures and Reference No. 14, Highways in the River Environment.

In the scour study special emphasis should be placed in areas where problems or failures have occurred at the Department's 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, causing spill-through abutments to fail. In some cases piping occurs behind the end bent or for bridges with timber bulkheads behind the bulkhead. This leads to undermining of the pavement and in some cases the fill material may be completely scoured away but the pavement still remains creating a severe traffic hazard. Abutment scour also 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. Often this scour occurs in the vicinity of the first interior bent. Guide banks (spur dikes) should be considered for protection against this type of failure.

   In wide flood plains this type of scour can occur because of the lack of over flow bridges or the spacing between bridges is too far. A good rule of thumb is that bridges should be located every one half mile. Two-dimensional hydraulic modeling is necessary to adequately design bridge spacing in wide flood plains. All bridge abutments should be protected for 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.

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 utilizing 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 to occur. 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 ranging over the maximum time available 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 substructure of the bridge. 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 that flood which 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.

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 risk analysis. Forms for this procedure begin on page 41.

1.3.2 Level 2 Procedures for Tidal Bridges. The hydraulic design procedures for tidal bridges are new and relatively little experience has been gained in this area. The latitude of permissible procedures is consequently greater than 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 utilized. See Reference 42 for a more complete discussion of tidal hydraulics.

Tidal hydraulics are produced by astronomical tides, storm surges, and 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 set-up.

Storm surges in South Carolina are produced by two types of storms, hurricanes and northeasters. The northeaster is a long duration storm and 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 time 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 from 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 just becoming available to the highway hydraulic 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. Version 3 of FESWMS, will have wind capability in both the two-dimensional mode and in the one-dimensional mode.

In tidal areas, bridge lengths are generally controlled by wetland considerations rather than hydraulics. So the purposes of hydraulic analysis for bridges in tidal areas are primarily concerned with establishing the grade of the bridge and determining 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 set-up. Sufficient opening should be provided to relieve this difference. A detailed analysis should be made to correctly size the opening.

1.3.2.1 Establish Minimum Bridge Grades. The minimum 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 sources for storm surge frequency data. The Corps of Engineers has produced a data set of historic hurricanes that have hit the Atlantic and Gulf coasts. The database contains tidal surge hydrographs for 134 hurricanes over 104 years. 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 eight ADCIRC stations along the South Carolina coast. *The Tidal Hydraulic Modeling For Bridges Users Manual*, Reference 41, lists these sites 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 in utilizing 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 is for 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 Engineer's Shore Protection Manual, Reference No. 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 swallow 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, 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. Basically the same type of data should be gathered. The historic aerial photograph data is of significant importance. This is 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 8 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 and make sure that he is using stillwater heights. The tidal hydrograph should be developed utilizing the following equations:

$$S_{tot}(t) = S_p(1 - e^{-[D/t]}) + H_t(t)$$

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

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 the data provided by the NOAA report NWS-38 (1987). Figures giving these values are in Appendix A of Reference 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:

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

In order 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 $t = 50$

The next consideration is at what stage of the daily tide should the peak tidal surge 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. The Final Report, Phase II for the Pooled Fund Study, Reference No. 43, provides three analyses approaches for developing the boundary conditions for tidal hydraulic modeling, the Corps of Engineer's 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 for the Pooled Fund Study.

[1] Reference 42, pp. 42

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 UNET 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. Utilize HEC-1 to develop the fresh water inflow hydrograph based on this rainfall. Only that part of the drainage area of the stream that would have a time of concentration corresponding to timing of the rainfall hyetograph and the duration of the tidal surge hydrograph should be considered. To account for the flow from upstream areas or base flow, utilize 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 utilized as the flow at the upstream boundary of the hydraulic model.

Before the UNET 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, most of which were in operation for less than two years. There are also several reference stations that have been in place for a much longer time period. The data from these gaging stations can be used to calibrate the model. The data that is 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 should be simulated for calibration. Wind setup can effect the gage readings. Utilizing weather records, the designer should verify whether this is a problem with the gage data before using it.

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 UNET.

The rising and falling tidal surge will each have a point of maximum outlet velocity. This 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 Engineer's 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 Engineer's 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 a UNET 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 Engineer's method for tidal surge hydrographs or the EST approach would be considered a Level 3 analysis even if the one-dimensional model UNET is used. This is because of the extra level of effort and the statistical analysis required in these approaches.

1.5 Information to Be Shown on Plans

1.5.1 For Riverine Bridges

1. Historic high-water data including: elevation of high water, date of occurrence, discharge, if available, and source of data.

2. Design high-water elevation, 25-year for secondary, 50-year for primary. The high-water elevation should include the backwater produced. The amount of backwater should be shown.

3. 100-year high-water including backwater; show the amount of backwater.

4. The hydrology data should be shown in the following format:

Hydrology Data:

D.A. = _______ sq. mi.
Q_n = _______ cfs. (n = design period)

Area furnished under Elev. _____ = _____ sq. ft. (Elevation of the water surface within the bridge.)
Vel. _____ ft./sec.
Q_{100} = _______ cfs.
Area furnished under Elev. _____ = _____ sq. ft. (Elevation of the water surface within the bridge.)
Vel. ______ ft./sec.

Overtopping Flood:

Q = ______ cfs.
Probability = _________

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.

5. 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 HEC-18.

1.5.2 For Riverine Culverts

1. Show the same high-water information as for bridges.

2. The hydrology data should be shown in the following format:

Hydrology Data:

D.A. = _____ sq. mi. (or acres)
Q_n = ______ cfs. (n = design period)
n-year headwater elev. = ________
Outlet vel. = __________
Q_{100} = ______ cfs.
100-year headwater elev. = ________
Outlet vel. = __________

Overtopping Flood:

Q = _________ cfs.
Probability = __________

1.5.3 For Tidal Bridges

1. Show the following tide elevations:
   - Mean high and mean low tide elevations,
   - 10-year tidal surge height including wave height, labeled as such,
   - 100-and 500-year stillwater surge height.

2. Show maximum velocity within the bridge for the 100- and 500-year tidal surges.

3. A plot of the 100- and 500-year scour lines on a bridge plan and profile sheet.

1.5.4 For Tidal Culverts

1. Show the following tide elevations
   - Mean high and mean low tide elevations
   - 10-year tidal surge height including wave height-labeled as such
   - 100- and 500-year stillwater surge height

2. Show maximum outlet velocity for the rising and falling tide for the 100-and 500-year tidal surge and indicate the direction of flow at the time of maximum velocity.

1.6.1

HYDRAULIC DESIGN
AND
RISK ASSESSMENT FOR
BRIDGE/BRIDGE REPLACEMENT OVER
stream name
ROUTE/ROAD NUMBER ________
FILE NO.__________ PROJECT NO.__________PIN________
________ COUNTY, SOUTH CAROLINA

DATE

Prepared By ___________
Checked By ___________

____
Signed and Sealed

1.6.2

COMPARATIVE DATA

PROJECT DESCRIPTION

County_______________________ Rt./Rd. No.__________________________________
Stream_______________________ File No. __________ Project No.________________
PIN__________ ______________Charge Code_________________________________
Project Engineer_________________________________ Road Squad________________

By_________________Date_______ Checked by________________ Date_______

ROUTE/ROAD NO.				
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.^{2})				
VELOCITY (ft./sec)				
HIGHWATER ELEV. (ft.)				
HIGHWATER DATE				
HIGHWATER DEPTH (ft.)				
NORMAL WATER ELEV. (ft.)				
NORMAL WATER DATE				
NORMAL WATER DEPTH (ft.)				
FILE/DOCKET/PROJECT NO.				
DATUM/DATUM TIE				

1.6.3

SITE INSPECTION

County____________________________ Rt/Rd No.___________ Date____________
Stream____________________________ PIN______
By____
____
____

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 channel blocked horizontal______ Percent channel 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

General Topography____
Stream Type (circle one) Straight, Braided, Anabranched or Meandering. Are channel banks stable? Yes/No If No, Describe ____
____
____
Are there any hydraulic controls upstream or downstream? Yes/No Describe_______
____
____
____
Soil type___________________ Exposed rock? Yes/No, If so, 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

Channel

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

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

$n2$ -- Variations of Channel Cross Sections	Gradual	.000
	Alternating	.001-.005
	Occasionally	
	Frequently	.010-.015
n_3 -- Relative Effect of Obstructions	Negligible	.000-.004
	Minor	.010-.015
	Appreciable	.020-.030
	Severe	.040-.060
n_4 -- Vegetation	Low	.002-.010
	Medium	.010-.025
	High	.025-.050
	Very High	.050-.100
m -- Degree of Meandering	Minor	1.00
	Appreciable	1.15
	Severe	1.30

Site Observations
for Channel

Channel Depth	n_b	n_1	n_2	n_3	n_4	m	Computed n

1.6.3.3

Manning's "n" Values
For Over Bank Areas

$n = n_b + n_1 + n_3 + n_4$

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

n_3 -- Effect of Obstructions	Negligible	.000-.004
	Minor	.005-.019
	Appreciable	.020-.030
n_4 -- Amount of Vegetation	Small	.001-.010
	Medium	.011-.025
	Large	.025-.Very Large

Site Observations
For Over Bank Areas

Location	depth	n_b	n_1	n_3	n_4	Computed n

1.6.4 COMPARATIVE BRIDGE SITE INSPECTION FORM

County____________________________Rt./Rd No.____________________________
Stream ____________________________Measured bridge length_______________ft.
Maximum span length _______________ft. Superstructure type_________________

Substructure type______________________End Abutment type_________________

Riprap present? Yes/No Condition____

Stream type (circle one) Straight, Braided, Meander, or Anabranched. Alluvial or Rock.

Any visible signs of scour problems (describe)_________________________________

____

Are banks stable (describe)____

Debris blockage; Percent of channel blocked horizontally _______________________

vertically ______________ Describe other signs of debris ______________________

____

Any other problems ____

____

Draw sketch and indicate problem areas. On sketch indicate location of woods, fields and other land uses in the vicinity of bridge. Show north arrow and direction of flow.

1.6.5 Risk Assessment

Risk assessment involves the risk in dealing with the hazards associated with floodwaters and the economic cost of protection against those risks. The objective is to arrive at an acceptable level of protection while minimizing the cost. Only the design decisions controlled by hydraulic design criteria are evaluated. The procedure is to work through the following questions. The answers will lead to the appropriate result.

In this procedure the maximum flood is defined as the 500-year flood or the overtopping flood which ever is smaller.

I. Backwater Damage

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 LTEC 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).

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 Costs

1. Is the overtopping flood less than the 100-year flood? Yes_____ (Go to 2) No_____ (Go to 3).

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 constraints.

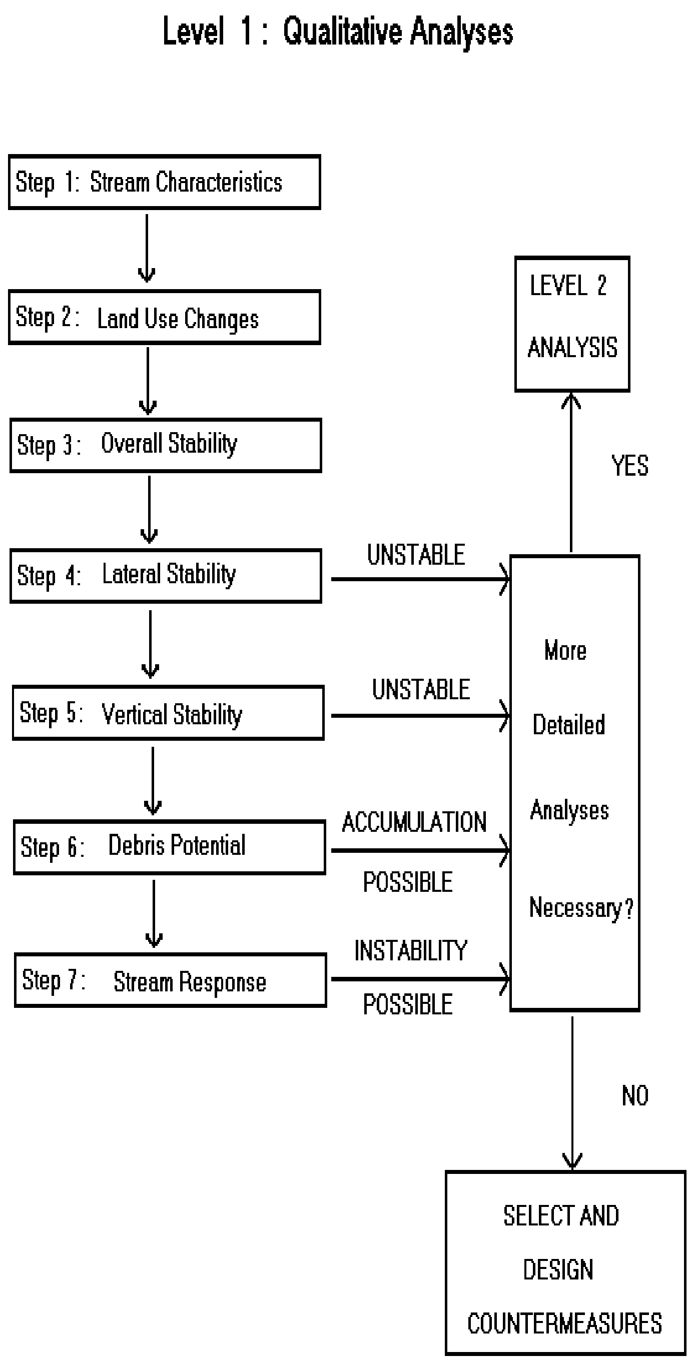
Level 1 : Qualitative Analyses
Step 1: Stream Characteristics
Step 2 : Land Use Changes
Step 3 : Overall Stability
Step 4: Lateral Stability
Step 5: Vertical Stability
Step 6: Debris Potential
Step 7: Stream Response
UNSTABLE
UNSTABLE
ACCUMULATION
POSSIBLE
INSTABILITY
POSSIBLE
More
Detailed
Analyses
Necessary?
YES
LEVEL 2
ANALYSIS
NO
SELECT AND
DESIGN
COUNTERMEASURES

Figure No. 1

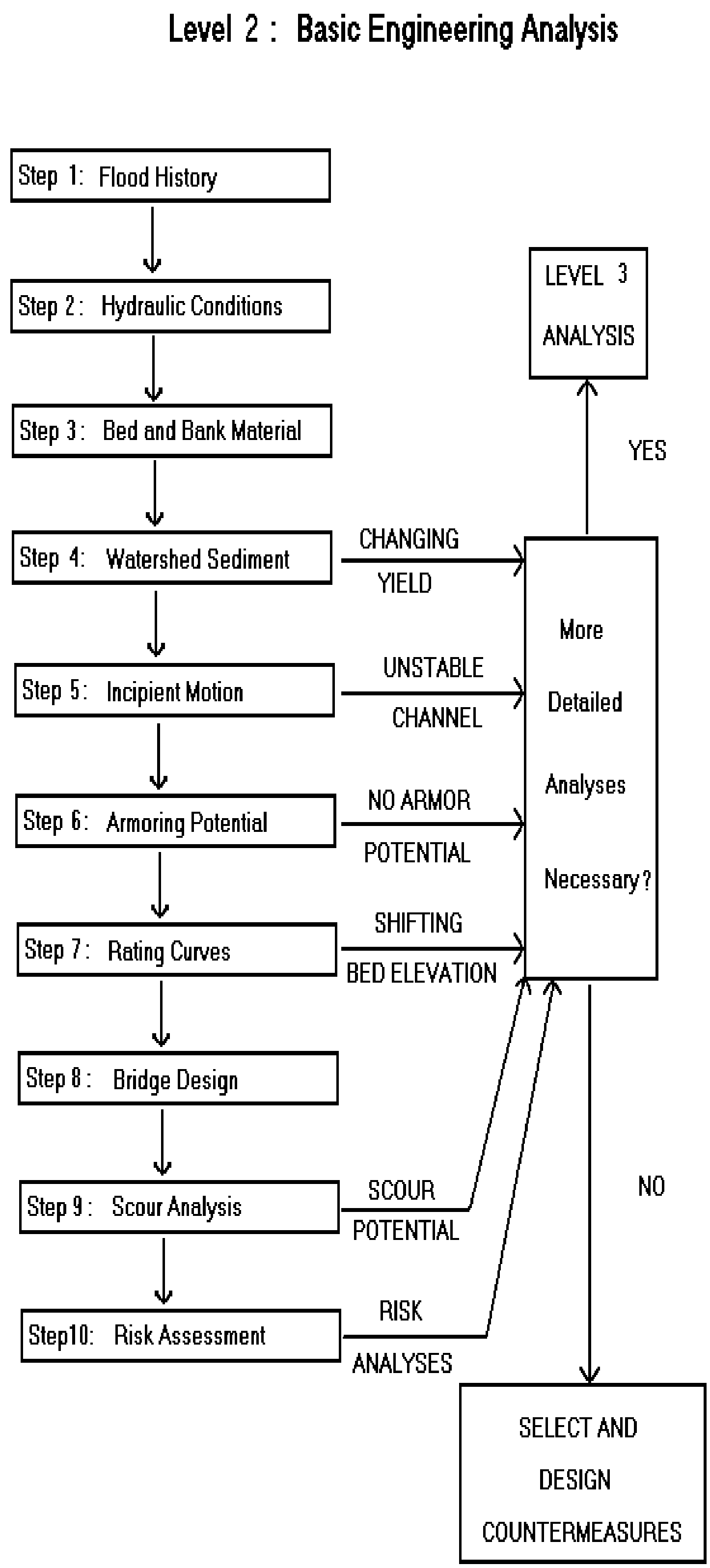
Level 2 : Basic Engineering Analysis
Step 1: Flood History
Step 2 : Hydraulic Conditions
Step 3 : Bed and Bank Material
Step 4: Watershed Sediment
Step 5: Incipient Motion
Step 6: Armoring Potential
Step 7 : Rating Curves
Step 8 : Bridge Design
Step 9 : Scour Analysis
Step10: Risk Assessment
CHANGING YIELD
UNSTABLE CHANNEL
NO ARMOR POTENTIAL
SHIFTING BED ELEVATION
SCOUR POTENTIAL
RISK ANALYSES
More Detailed Analyses Necessary?
YES
LEVEL 3 ANALYSIS
NO
SELECT AND DESIGN COUNTERMEASURES

Figure No. 2

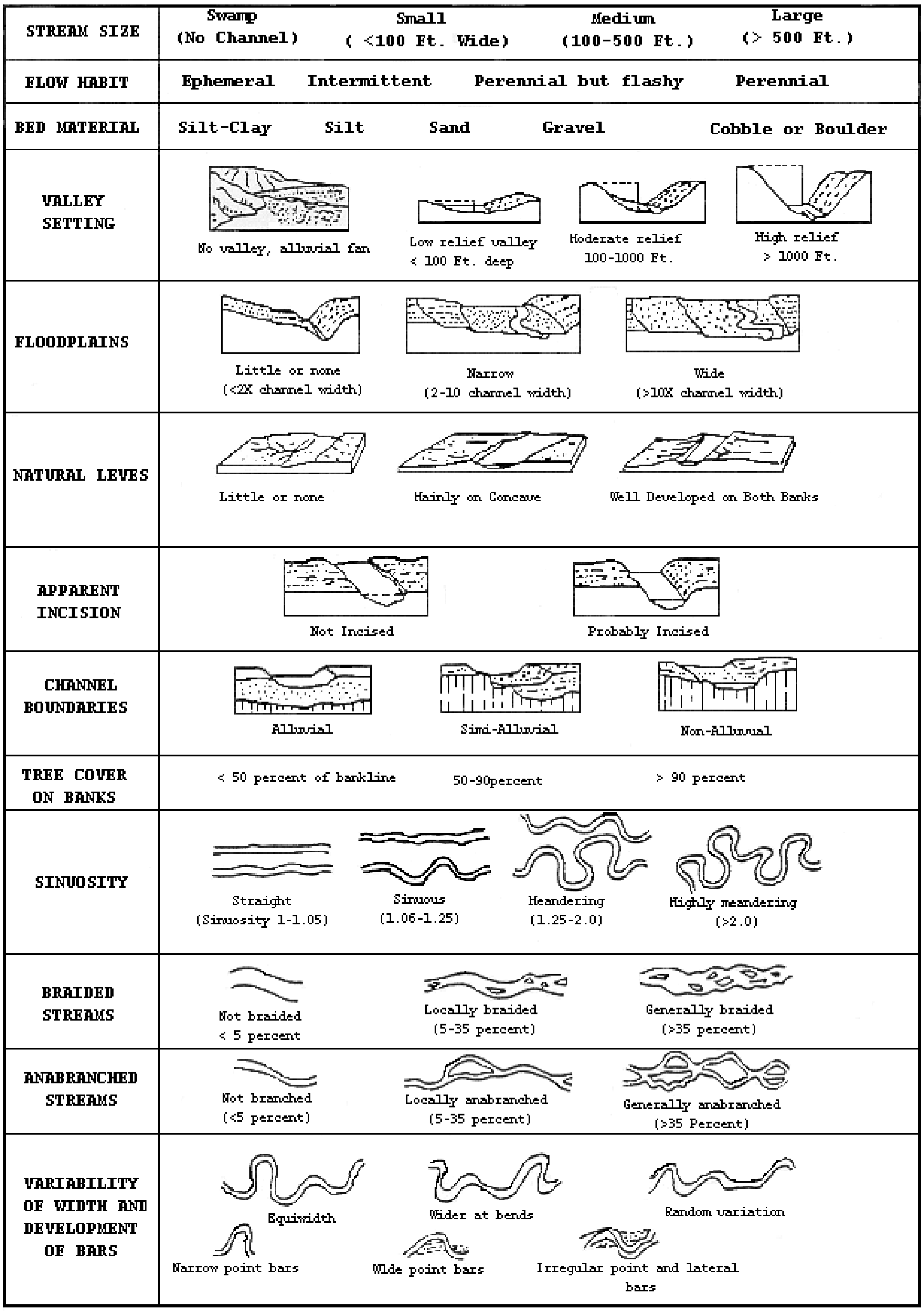

Figure 3. Geomorphic factors that affect stream stability.

2. 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 and the NPDES general permit. These procedures are outlined below. 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 as indicated in Part 1. A sample title sheet is shown on page 63. 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 engineer using the design approach which best meets the particular site conditions involved.

2.2 Design Criteria

2.2.1 Cross-Lines Pipes. The design discharge for cross-line pipes for primary 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.

2.2.2 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.

2.2.3 Inlet Spacing. Inlet spacing will be based on the spread criteria in the AASHTO Model Drainage Manual as modified below. Maximum spacing is 400 feet and minimum spacing is 150 feet unless specified by field personnel.

Road Classification	Design Frequency	Design Spread
Interstate Routes and Primary Routes with ADT > 10,000		
< 45 mph	10-year	Shoulder
> 45 mph	10-year	Shoulder
*Sag point	50-year	Shoulder

Primary Routes with ADT < 10,000		
< 45 mph	10-year	1/2 driving lane
> 45 mph	10-year	Shoulder

Road Classification	Design Frequency	Design Spread
Primary Route ADT < 10,000 *Sag point	10-year	1/2 driving lane
Secondary Road	10-year	1/2 driving lane
Paved shoulder	10-year	Shoulder
*Sag point	10-year	1/2 driving lane

*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 storm water level in the median will be below the edge of the shoulder. Maximum inlet spacing will be 750 feet.

2.2.4 Minimum Ditch and Pipe Grades. Minimum grade on ditches, gutters, and pipes in a storm drainage system should be 0.3% where possible. Minimum velocity for the design discharge in a pipe should be 3.0 feet per second. This will promote self-cleaning of the pipe.

2.2.5 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.

2.2.6 Minimum Cover for Pipes. The minimum cover for storm drain pipe systems is 1.5 feet except for yard drains that have a 0.5 foot minimum cover.

2.2.7 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.

2.2.8 Storm Drain Systems. Storm drain systems will be designed for free surface flow. Design flow depths in pipes should equal 0.94 times the pipe diameter for maximum free surface flow capacity.

2.2.9 Outlet Protection for Culverts. All culvert outlets will be investigated for scour potential utilizing 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.

2.2.10 Drainage Outfalls and Stormwater Management. Outfalls will be designed using the following criteria:

- Outfalls that are not natural channels shall be designed for the 10-year discharge.
- For drainage outfalls that are natural watercourses, no modifications will be made to the channel except as necessary to prevent scour or erosion or to accommodate highway drainage structures.
- For projects with 5 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, Department policy is that we will comply with them if it is practical and economical.

2.2.10.1 Outfalls in Flood Hazard Areas. If the outfall has been designated as a flood hazard area or if it is likely that development will take place along the outfall, the 100-year profile will be computed and the boundaries of the 100-year floodplain will be identified. This information will be forwarded to the local community planning commission and FEMA so that future development in the 100-year floodplain can be restricted in accordance with FEMA Regulations 44 CFR Part 60.3.

2.2.10.2 Drainage Regulation Violations By Others. Where Department 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 Department property 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.

2.2.10.3 Basic Design Requirements:

1. 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.

2. 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 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 utilizing the design methods in FHWA's Hydraulic Engineering Circulars HEC-15 or HEC-11.

3. Detention will only be used when the resulting flattening and elongation of the hydrograph will not cause any significant increase in flow along the outfall channel at any point downstream. A significant increase in flow is an increase in discharge that will cause damage to property. A significant increase in flow can occur if the site of the detention pond is located some distance downstream from the headwaters of the stream. Although the detention pond will reduce the peak flow from the outfall, it will shift the timing of the outflow so that it will add a greater amount to the peak discharge of the main stream causing it to be increased.

4. At sites where existing cross-line pipes are undersized and must be enlarged to meet design standards, a determination of why the existing pipe is undersized will be made. If the cause is due to upstream development, an investigation to determine if any drainage law or regulation has been violated will be conducted. When there is a violation, documentation of the violation will be furnished to the Department's legal staff for consideration of possible legal action.

6. At sites where cross-lines must be increased to meet design criteria, a hydraulic analysis of the channel will be made for the 100-year flood using HEC-2, HEC-RAS, or WSPRO to determine the boundaries of the 100-year floodplain. The study will also determine the impacts on downstream properties due to the increased flows released by the larger pipe. The channel reach that should be included in the study should be downstream to the limit of influence and upstream to the limit of backwater effects caused by the project. A report consisting of the computer printout, a map delineating the floodplain boundary, and a high-water profile will be sent to the local community planning commission, the State Coordinator for the National Flood Insurance Program, and to FEMA to inform them where development should be restricted in accordance with FEMA Regulations, 44 CFR Part 60.3

7. If the downstream channel has been blocked or restricted by downstream property owners and it appears that violations of State drainage law 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.

8. If the outfall does not meet the South Carolina legal definition of a water course and the natural path downstream has been blocked by a land owner, (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.

9. 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 plots of surveys. The specific law or regulation being violated should be referenced.

10. 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 modifications caused by the proposed land disturbing activity, with and without the pond. The results will be used to assess the design. If the downstream impacts are detrimental, the pond may need to be redesigned or eliminated.

2.2.10.4 Stormwater Management for "C" Projects. On "C" projects when roadway surface water is collected in a grass-lined roadside ditch, stormwater management design is not required, provided the ditch is on a non-erodible slope.

Since "C" projects generally cost less than other projects, minimum design efforts are required for these projects. All drainage facilities are to be set by the project engineer using engineering judgment. However, all projects will have a study by the Hydraulic Engineering Section to meet the minimum requirements under the NPDES Regulations. Sediment and erosion control will be handled in the same manner as other projects. If there are special problems, the project engineer may request a design study through the Support Services Engineer. The secondary road design standards will be applied by the Hydraulic Engineer for the requested studies.

2.2.11 Sediment and Erosion Control. All projects that have 5 or more acres of disturbed area will be designed for sediment and erosion control in accordance with DHEC's General NPDES Permit and State Erosion Control regulations. When a construction contract has been awarded, the contractor will prepare a set of plans showing all erosion control best management procedures (BMP's) that he plans to use for the areas within the right of way. The plans will remain at the construction site and be available for inspection by DHEC and Department personnel at all times. All BMP's that require right of way will be designed by the hydraulic design engineer.

All permanent detention ponds (note: this is a modification made on August 28, 2002, from the original document stating all detention or sediment and erosion control ponds) will be designed to store and release the first inch of runoff over the 24-hour period. Sediment storage will be predicted by the Universal Soil Loss Equation or other methods acceptable by DHEC. All ponds will be designed for the 10-year 24-hour storm. They should be able to pass the 100-year 24-hour storm through the emergency spillway.

All sediment and erosion control basins shall be designed to trap 80 percent of the suspended solids or 0.5 ml per liter of settleable solids, whichever is the lesser requirement.

2.2.12 Hydraulic Analysis. Hydrologic analysis should be performed utilizing 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 utilizing the SCS velocity method described in the TR-55 manual. The minimum time of concentration is 5 minutes. Rainfall intensities will be determined utilizing the methods in HYDRO-35 (Ref. No. 27). Coefficients for this method are available for most areas of the state on the Department's web page.

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

$$Q = C\ I\ A\ C_f$$

Where: Q = discharge in cubic feet per second (cfs)
C = the run-off 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

2.2.12.2 The SCS TR-55 Method. Utilize the modified SCS TR-55 method described in References No. 22, 23 and 24 for the following conditions.

Physiographic Region	Drainage Area in Acres
Lower Coastal Plain	100 - 384
Upper Coastal Plain	100 - 2816
*Piedmont	-----
Blue Ridge	100 - 384

* Piedmont regression equation drainage area range overlaps the rational equation range. The modified TR-55 method may be used as an alternate or as a check.

2.2.12.3 USGS Regression Equations. For rural drainage areas greater than the limits listed below, utilize the USGS Regression Equations (Ref. No. 5).

Physiographic Region	Drainage Area in Square Miles
Lower Coastal Plain	0.6
Upper Coastal Plain	4.4
Piedmont	0.1
Blue Ridge	0.6

2.2.12.4 USGS Urban Peak-Discharge Frequency Method. For urban drainage areas greater than the limits listed under Section 2.2.12.3., utilize 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, reference No. 20.

2.2.12.5 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 utilizing the method described in Reference No. 5.

2.2.12.6 Hydrograph Methods. If a hydrograph is needed, utilize the methods in Reference No. 19 for rural drainage areas and Reference No. 20 for urban drainage areas that exceed the minimum drainage areas listed in Section 2.2.12.3. For smaller drainage areas utilize the modified SCS TR-55 method in References No. 22, 23 and 24.

2.3 Hydrologic and Hydraulic Design for Stormwater Management

All roadway drainage will be designed utilizing stormwater management procedures as defined by the Stormwater Management and Sediment and Erosion Control Regulations (Reference No. 26). The design study 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 see the AASHTO Model Drainage Manual and the AASHTO Highway Drainage Guidelines.

STEP 1 Scoping Review. The first involvement on a project for the hydraulic engineering designer 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 be furnished 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. He should transfer this information to a copy of the USGS topographic map of the area. Major outfalls and stream crossings should be identified. The design engineer should determine if any of the involved streams is a regulated floodway. 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. He will also identify and layout the survey requirements necessary for the drainage design process. Particular emphasis should be given to outfalls that will require detailed studies to insure that surveys and studies are made early on so as not to delay the project schedule.

STEP 2 Gathering Data. This step is for basic information gathering or research for the study. This step should start after the survey is completed and preliminary plans have been furnished to the Hydraulic Engineering Section by Road Design.

1. Obtain all available data in the central office.
   - A copy of plans for existing roads that will impact the project area.
   - Obtain 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 utilizing Microstation. This is done by superimposing the roadway data over a USGS topographic map or an aerial photograph of the area.

3. Determine from FEMA floodway and FIRM maps if there is any involvement in floodways or flood hazard areas. If there is involvement, a copy of the appropriate map 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.

2. For curb and gutter sections, layout preliminary drainage system including locations of catch basins, using GEOPAK Drainage.

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. He is usually familiar with local drainage conditions and problems.

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 yard drains.
6. Locate sites for sediment control ponds and for detention ponds, if required.
7. Verify or adjust the locations of catch basins.

<u>STEP 5 Complete Drainage Designs.</u>

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 programs GEOPAK Drainage, THYSYS, and HYDRA (in HYDRAIN) may 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 utilizing any other computer programs for storm sewer analysis or design, obtain concurrence from the Department's Hydraulic Engineer.

The rational method is used to determine the flow rates for storm drain systems providing that the drainage area does not exceed 100 acres. For drainage areas exceeding 100 acres TR55 procedures should be used. In special situations where storage is a factor, routing procedures should be used. The modified SCS TR-55 method will be used to develop inflow hydrographs.

1. Check the locations of catch basins to see if the design spread criteria from Section 2.2.3. are met, utilizing the performance graphs for type 16, 17, and 18 catch basins developed by the Hydraulic Engineering Section. These charts are available on the DOT web page. Using sound engineering judgement, adjust the locations as necessary to meet the design criteria and physical constraints. Determine the time of concentration for each inlet.

2. 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. Evaluate the performance of the proposed system utilizing 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 top 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 flowlines of the pipes necessary to carry the water based on pipe capacity and friction losses. The next step is to compute junction losses utilizing the method described in Reference 4. When THYSYS is used, this step must be done by hand. The hydraulic grade lines are computed and plotted on a profile of the storm sewer system. On this drawing, the flow-lines of the soffits of the pipes, junction boxes, and the ground line at the junction boxes should be plotted. The energy grade line should be shown. Based on the results of these computations and using sound engineering judgement 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.

3. Evaluate the effects of the proposed construction on the flood discharges of the outfall as specified in Sections 2.2.10. through 2.2.10.4.

4. 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 XI-3 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/or 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 the water should be 1.0 foot below the subgrade of the road. The lining should be selected to stabilize the ditch and meet the requirements for sediment and erosion control.

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. The design procedure to meet this requirement is to determine if the grass lining for the ditch will prevent erosion. HEC-15 procedures should be used to design a stable channel. The HYCHL routine in HYDRAIN will design a stable channel using the HEC-15 procedures. 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:

- Grass lining
- Temporary biodegradable lining with grass
- Permanent synthetic lining with grass
- Riprap
- Wire enclosed rock called gabions and mattresses
- Asphalt paving

- Articulated pre-cast blocks

Resident Maintenance Engineers usually prefer asphalt paving over riprap because of the problems they have mowing the areas. The designer should work with them 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 outline.

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 up- and downstream.
    - Evaluate the channel stability.
    - Identify any property that might be potentially damaged by flooding. 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 a rating curve for the stream channel. Use WSPRO or 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 No. 44 as indicated in Section 1.3.1, Step 2, B.

D. Design Culvert. Design the culvert structure utilizing 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.

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 floodway or flood hazard requirements. The maximum head on the culvert should be no more that 1.0 foot above the natural high-water profile if the stream is in a flood hazard area.
    - 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 that 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 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. 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 frequently will 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 head water 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 supercritical or subcritical. If it is subcritical the analysis will be just 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 utilize 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 feature built in. Consideration should be given to how the reduction and shifting of the hydrograph peak will effect flooding both upstream and downstream.

G. Information to be Shown on Plans for Cross Line Structures.

1. Show the design, 100-year and historic headwater elevations.

2. Show the hydrology data in the following format:

Hydrology Data:

D.A. = _____ sq. mi. (or acres)
Q_n = ______ cfs (n = design period)
n year headwater elev. = ________ft.
Outlet vel. = __________ft/sec
Q_{100} = ______ cfs
100 year headwater elev. = ________ft.
Outlet vel. = __________ft/sec

Overtopping flood:
Q=______ cfs
Probability = ________

STEP 7 Prepare 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 certifying that the design complies with the appropriate regulations. 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.

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

1. A topographic map utilizing the best available topographic mapping. For "C" type projects and bridge replacement projects, the road location will be indicated in red 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 in red (orange, if the road is in red). The outfalls for road drainage and storm drain systems should be highlighted in yellow. 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 Design Squad should furnish this information. The outfall stream names should be listed. Runoff coefficients for the area within 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 include a stream profile showing natural water surface and the backwater profile for the design flood and the 100-year flood.

2.4 Sediment and Erosion Control

Sediment and erosion control is based on the philosophy that "an ounce of prevention is worth a pound of cure". The best means of sediment and erosion control is to stabilize disturbed areas as soon as possible by planting grass when work temporarily stops on an area. Regulations require that temporary stabilization must be in place within 7 days after work stops on an area unless work will start back in less than 21 days.

The second level of effort to control sediment and erosion is that all stormwater that comes from the disturbed area must be treated by some structural method or Best Management Practice (BMP) before it is released off site or allowed to mix with off site water. The BMP's used by the Department are discussed below. If possible, the site should be isolated from off-site water so that the sediment control methods can be limited in capacity. This will also involve structural methods of control.

The design procedures for sediment and erosion control are presented in Reference No. 29. Computer programs SEDIMOT and SEDCAD+ can be used for sediment and erosion control design.

2.4.1 Criteria

1. All stormwater from a construction site must be treated before it is released.

2. Sediment and erosion control ponds will be designed for efficiency of 80 percent of suspended solids or 0.5 ml. per liter of settleable solids concentration, whichever is the lesser requirement. The pond's primary spillway should be designed for a 10-year 24-hour storm. The emergency spillway shall be designed for the 100-year 24-hour storm

2.4.2 Sediment Barriers. To contain the sediment that will erode from the construction site, temporary barriers are placed around the site where runoff will be in the form of sheet flow.

2.4.2.1 Silt Fence. A silt fence is a temporary barrier consisting of synthetic geotextile fabric stretched across an area, supported by post, and entrenched into the ground (See Standard Drawing No. 815-1, Ref. No. 46). It acts as a filter. Its use is limited to areas of sheet flow and areas of concentrated flow of less than 1.0 cfs. The sheet flow should have no more than 1/4 cfs per 100 feet of silt fence and the maximum fill slope protected by the fence must not exceed 2:1. Primarily, it is used around the perimeter of a construction project where the run-off flow is in the form of sheet flow. It can also be used around roadway catch basins during construction. However, it should not be used around median catch basins because the flows there will usually exceed the 1.0 cfs limit.

2.4.2.2 Hay Bales. Hay bales can be used as barriers if they are stabilized by driving at least two stakes through them into the ground and they are entrenched at least 6 inches on the up hill side (Standard Drawing No. 815-1). They are used under similar conditions as silt fence. However hay bales are considered effective for only three months. For a more complete discussion on the use of silt fence and hay bales see the AASHTO Drainage Manual and Reference No. 29.

2.4.3 Ditch Protection. During construction, ditches constructed on slopes that will erode the unprotected soil should be protected from erosion by temporary means. The permanent design of the ditch should follow the procedures in Section 2.3 Step 5.

2.4.3.1 Ditch Checks. On steep slopes, the recommended method of controlling erosion in a ditch before it is stabilized, is to effectively flatten the slope of the water surface by the use of ditch checks or check dams. These are small riprap dams in the ditch, constructed so that the water will flow over the top during storm events (Standard Drawing No. 815-3, Ref. No. 46). The ditch checks are placed in the ditch so that the downstream toe of one ditch check is level with the top of the next one downstream. This, in effect, flattens the flow line of the ditch.

2.4.3.2 Temporary Linings. On flatter slopes other temporary methods may be utilized. These may be asphalt tacking of straw, jute mesh, fiberglass roving, or other specially manufactured materials. The hydraulic design engineer should call for this material in his design based on HEC-15 procedures. The computer routine HYCHL in HYDRAIN has procedures for designing temporary channel lining.

2.4.3.3 Sediment Dams. Sediment dams can be used to trap larger sediment particles from disturbed areas of less than 5 acres. The sediment dam consists of a riprap dam with riprap channel lining downstream placed in a ditch (Standard Drawing No.815-6, Ref. No. 46). The ditch should have sufficient storage volume to be effective for trapping sediment. If sufficient volume is not available in the ditch, excavation will be required in accordance with the Standard Drawings. The required storage volume is 67 cubic yards per acre of drainage area. At the time when half of the volume has been filled with silt, the basin should be cleaned out and the sediment disposed of properly.

2.4.3.4 Sediment Traps or Inlet Filters. Sediment traps or inlet filters are designed for sediment protection around drop inlets. Two types of traps or filters are used.

1. Filters or traps used around type 16, 17, and 18 curb inlets should be constructed of hay bales and/or silt fence. The entrance to the inlet should be completely surrounded by the hay bales or silt fence. At the bottom of a sag, a riprap filter Type C should be used if the discharge is more than 1.0 cfs.
2. A riprap filter should be used for drop inlets (Standard Drawings 815-4, 815-5, and 815-5A, Ref. No. 46).

2.4.4 Sediment Ponds or Basins. For larger drainage areas, a sediment pond or basin should be used. A sediment basin is made by constructing a dam or a combination of a dam and excavation to create the required storage volume (Standard Drawing No.815-2). The outlet structures of the basin used by the Department consist of a perforated standpipe connected to an outflow pipe through the dam and an emergency spillway constructed, on undisturbed soil, and protected by riprap. The perforations on the standpipe start 1.0 ft. above the top of the outflow pipe and consist of four ½-inch diameter holes spaced around the pipe. The top of the riser pipe is open but covered by a trash rack. When sediment collected in the basin reaches half way between the top of the outflow pipe and the perforations, the basin should be cleaned out. The Standard Specifications give full details on the construction of the basin.

2.4.5 Detention Ponds. Detention ponds are design to be permanent structures. Their construction details are similar to the sediment pond but the riser is not perforated and the materials in the pipe are of more durable material. Details are also given the Standard Specifications (Ref. No. 46).

The detention pond should be designed so that the 10-year 24-hour storm peak discharge for post-construction is no greater than the pre-construction discharge. The entire outflow from the 10-year storm should pass through the primary spillway. The emergency spillway should safely pass the 100-year storm.

2.5

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

ROUTE/ROAD NUMBER ________
FILE NO.________ PROJECT NO._________CONSTR. PIN__________
______________ COUNTY, SOUTH CAROLINA

DATE

Prepared By ____________________
Checked By ____________________

____
Signed and Sealed

REFERENCE LIST

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