STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION

DRAINAGE HANDBOOK
Bridge Hydraulics

OFFICE OF DESIGN, DRAINAGE SECTION JULY 2012
TALLAHASSEE, FLORIDA

ACKNOWLEDGMENTS

This handbook was commissioned by Central Office Hydraulics to be written by Shawn McLemore at Jacobs Engineering and Dr. Mark Gosselin at OEA, Inc. The project was overseen by Amy Tootle, P.E., CO Hydraulics, and was reviewed by the District Drainage Engineers and Central Office Drainage Section. Comments were solicited from the districts on a chapter by chapter basis, and again for the entire document in its draft final stage of development.

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Chapter 1

Introduction

1.1 Purpose

This handbook is intended to be a reference for designers of FDOT projects and to provide guidelines for the hydraulic analysis and design of bridges, including scour. These guidelines were developed to help the hydraulics engineer meet the standards addressed in Chapter 4 of the Drainage Manual and incorporate pertinent sections of the 1987 Drainage Manual.

The guidance and values provided in this handbook are suggested or preferred approaches and values, not requirements, nor standards. The values provided in the Drainage Manual are the minimum standards. This handbook does not replace the standards and in cases of discrepancy, the Drainage Manual standards shall govern. This handbook neither replaces the need for professional engineering judgment nor precludes the use of information not presented in the handbook.

1.2 Distribution

This handbook is available for downloading from the Drainage Section website at: http://www.dot.state.fl.us/rddesign/Drainage/files/BridgeHydraulicsHB.pdf

1.3 Revisions

Any comments or suggestions concerning this handbook may be made by e-mailing the State Hydraulics Engineer.

1.4 Terminology Used in this Handbook

Refer to the Open Channel Handbook for terminology used to describe open channels. Refer to Appendix A of this handbook for terminology used to describe bridge hydraulics.

Chapter 2

Project Approach and Miscellaneous Considerations

The material in this chapter addresses background material and initial decision making needed in preparation for a bridge hydraulic design. More detailed design guidance will be presented in following chapters.

Most bridge projects in the State of Florida receive funding from FHWA. Even if the project is not planned for Federal funding, the funding situation may change before the project is complete. As a result, much of the hydraulic analyses and documentation required by the Department's standards are tailored to satisfy Federal regulations and requirements.

FHWA 23 CFR 650A outlines the principal hydraulic analysis and design requirements that must be satisfied to qualify bridge projects (as well as any other project involving floodplain encroachments) for Federal Aid. A copy of 23 CFR 650A is provided in Appendix A of the Drainage Manual. The requirements in 23 CFR 650A are very comprehensive and the drainage engineer should become familiar with them.

2.1 Identify Hydraulic Conditions

Before beginning any hydraulic analysis of a bridge, one must first determine the mode of flow for the waterway. For purposes of bridge hydraulics, the FDOT separates the mode of flow into 3 categories of tidal influence during the bridge design flows:

1. Riverine Flow – crossings with no tidal influence during the design storm such as (a) inland rivers or (b) controlled canals with a salinity structure ocean ward intercepting the design hurricane surge. Bridges identified as riverine dominated require only examination of design runoff conditions.

2. Tidally dominated flow – crossings where the tidal influences are dominated by the design hurricane surge. Flows in tidal inlets, bays, estuaries and interconnected waterways are characterized by tide propagation evidenced by flow reversal (Zevenbergen et al., 2004). Large bays, ocean inlets, open sections of the Intracoastal Waterway are typically tidally dominated so much so that even extreme rainfall events have little influence on the design flows in these systems. Tidally dominated with negligible upland influx require only examination of design storm surge conditions.

3. Tidally influenced flow - Flows in tidally influenced crossings, such as tidal creeks and rivers opening to tidally dominated waterways, are affected by both river flow and tidal fluctuations. Tidally affected river crossings do not always experience flow reversal, however backwater effects from the downstream tidal fluctuation

can induce water surface elevation fluctuations up through the bridge reach. Tidally influenced bridges require examination of both design runoff and surge conditions to determine which hydraulic (and scour) parameter will dictate design. For example, a bridge located near the mouth of a river that discharges into a tidal bay (Figure 2-2) may experience a high stage during a storm surge event. However, high losses through the bridge and a relatively small storage area upstream may limit the flow (and velocities) through the bridge. In fact, the design flow parameters (and thus scour) may occur during the design runoff event while the design stage (for clearance) and wave climate occurs during the storm surge event. Given that tidally influenced crossings may require both types of analyses, inclusion of a coastal engineer for these bridge projects is recommended.

The level of tidal influence is a function of several parameters including distance from the open coast, size of the upstream watershed, elevation at the bridge site, conveyance between the bridge and the open coast, upstream storage, and tidal range.

By far, the best indicator is distance from the coast. Comparisons of gage data or tidal benchmarks with distance from the coast will illustrate the decrease in tidal influence with increasing distance (Figure 2-1). The figure shows that with increasing distance, the tidal range decreases, the flow no longer reverses, and, eventually, the tidal signal dies out completely. This illustrates the transition from tidally controlled (gage 2323592), to tidally influenced (gages 2323590, and 2323567, and 2323500), and finally to a riverine dominant system (gage 2323000).

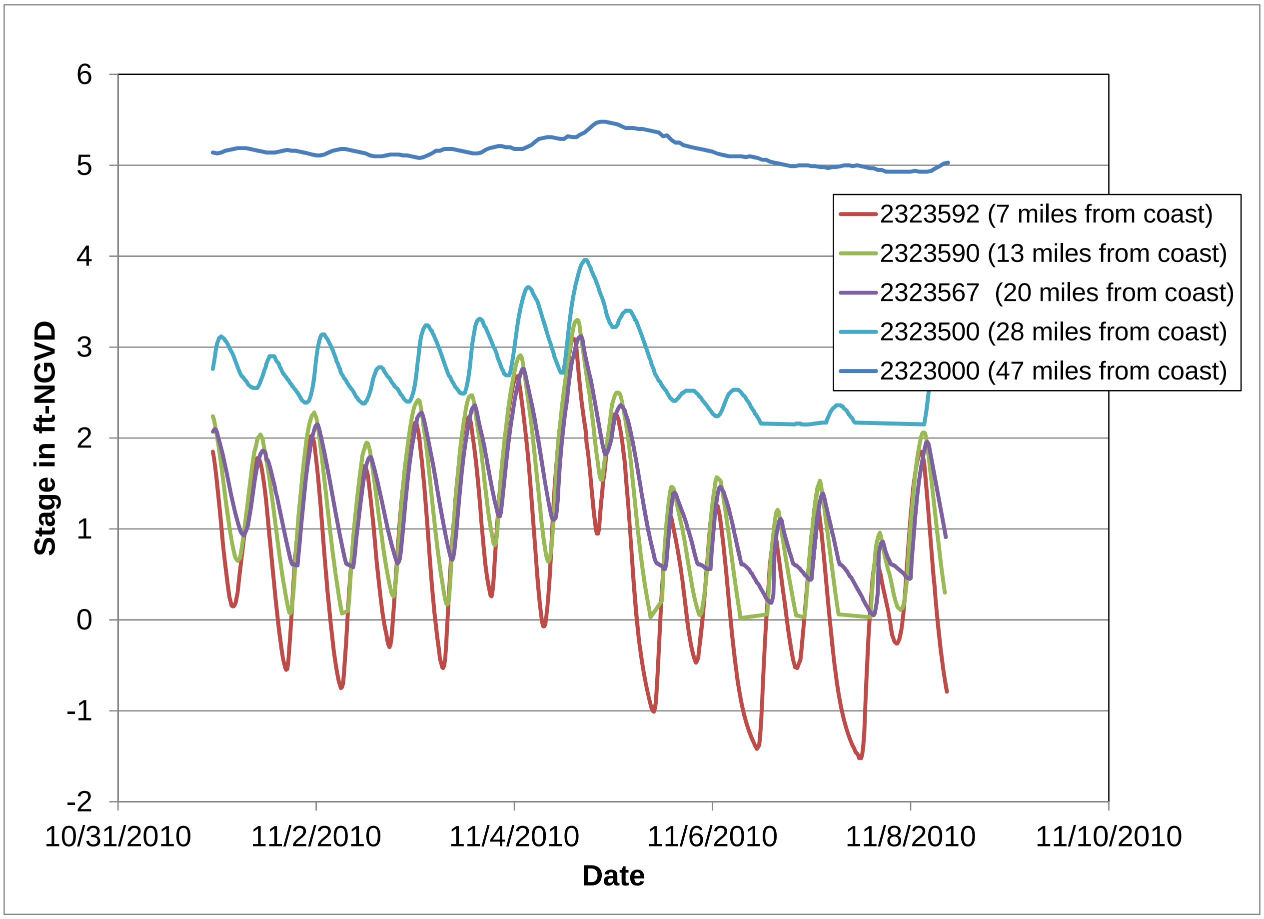


Figure 2-1 USGS Gage Data from the Suwannee River with Increasing Distance from the Coast

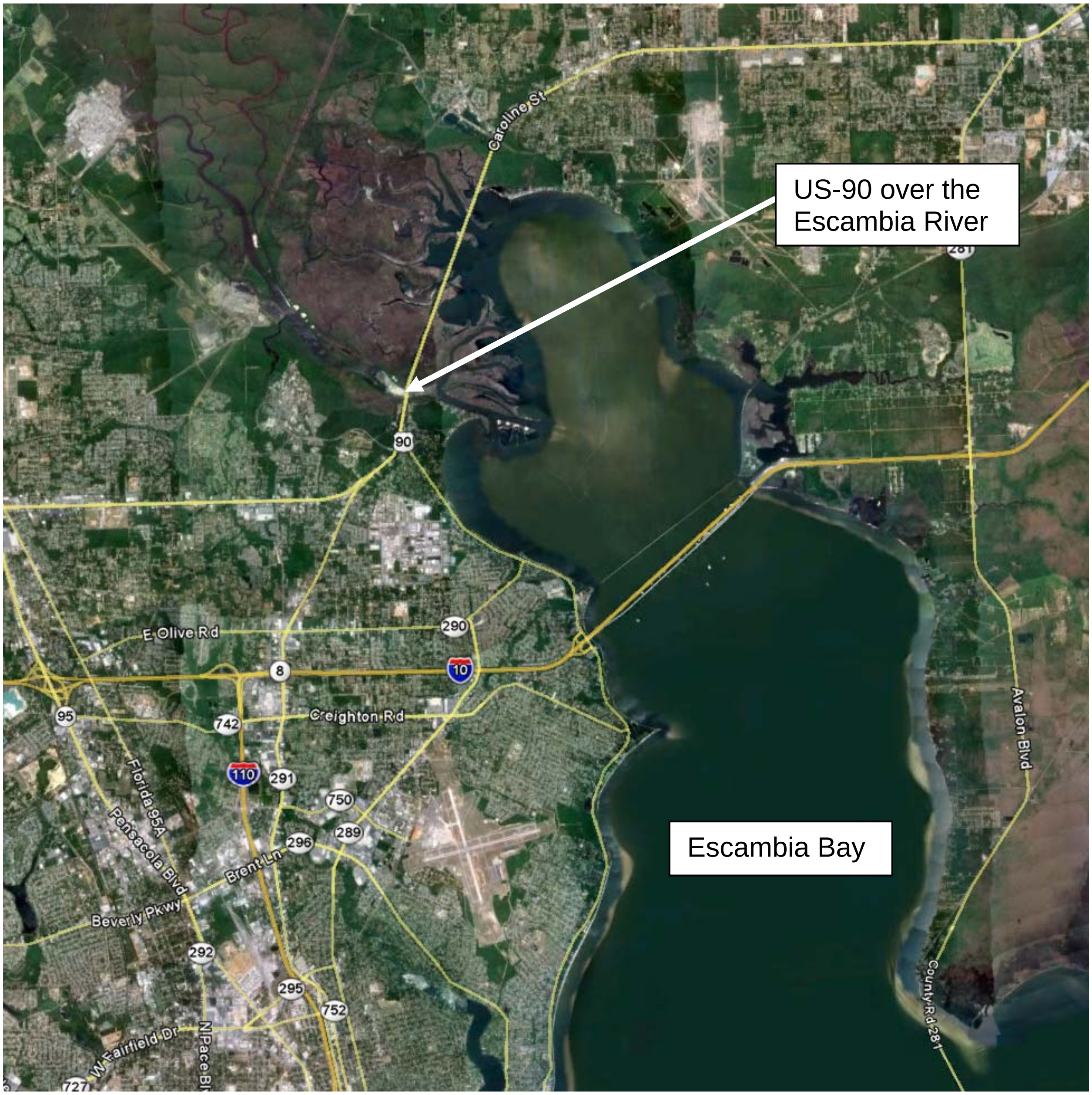


Figure 2-2 Example of a Bridge Requiring both Riverine and Tidal Analyses (US- 90 over Escambia Bay)

For the purposes of FDOT work, a coastal engineer is defined as an engineer who holds a M.S. or Ph.D. in coastal engineering or a related engineering field and/or has extensive experience (as demonstrated by publications in technical journals with peer review) in coastal hydrodynamics and sediment transport processes.

2.2 Floodplain Requirements

Potential floodplain impacts should be addressed during the Project Development and Environment (PD&E) phase of the project. A Bridge Hydraulics Report (BHR) will not usually be prepared during PD&E studies. However, if a BHR is not prepared for a bridge, then the Location Hydraulic Study should address:

- Conceptual bridge length
- Conceptual scour considerations
- Preliminary vertical grade requirements
- The need, if any, for the input of a coastal engineer during final design.

Refer to the PD&E or environmental documents and the Location Hydraulic Report for commitments made during the PD&E phase. Refer to Chapter 24 of the FDOT Project Development and Environment Manual for more information on floodplain assessment during PD&E.

2.2.1 FEMA Requirements

All bridge crossings must be consistent with the National Flood Insurance Program (NFIP). Requirements to be consistent with the NFIP will depend on the presence of a floodway and the participation status of the community. To determine these factors, review:

- Flood maps for the bridge site, if available, to determine if the floodplain has been established by approximate methods or by a detailed study, and if a floodway has been established.
- Community Status Book Report to determine the status of the community's participation in the NFIP.

Both the flood maps and the Status Book are available at the Federal Emergency Management Agency (FEMA) website: http://www.fema.gov/

The Special Flood Hazard Area (SFHA) is the area within the 100-year floodplain (refer to Figure 2-3). If a floodway has been defined, it will include the main channel of the stream or river, and usually a portion of the floodplain. The remaining floodplain within the SFHA is called the floodway fringe. The floodway is established by including simulated encroachments in the floodplain that will cause the 100-year flood elevation to increase one foot (refer to Figure 2-4).

Figure 2-5 shows an example of a floodway on the flood map. The floodway, as well as other map features, may have a different appearance on different community flood maps. Each map will have a legend for the various features on the map.

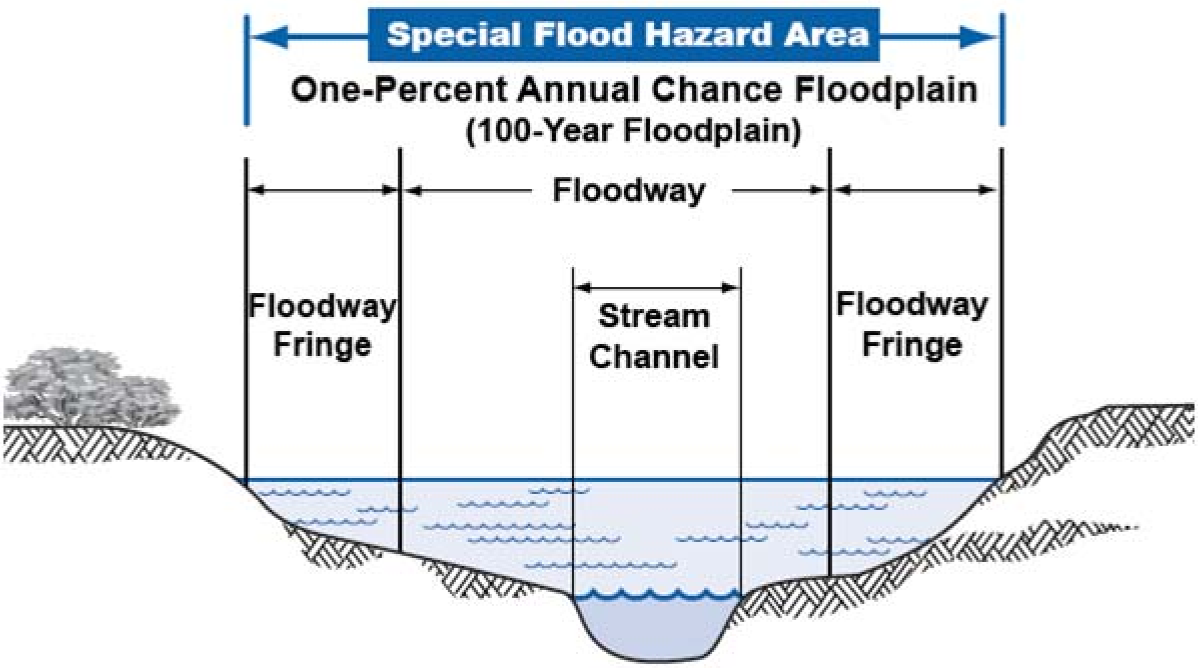


Figure 2-3 Special Flood Hazard Area

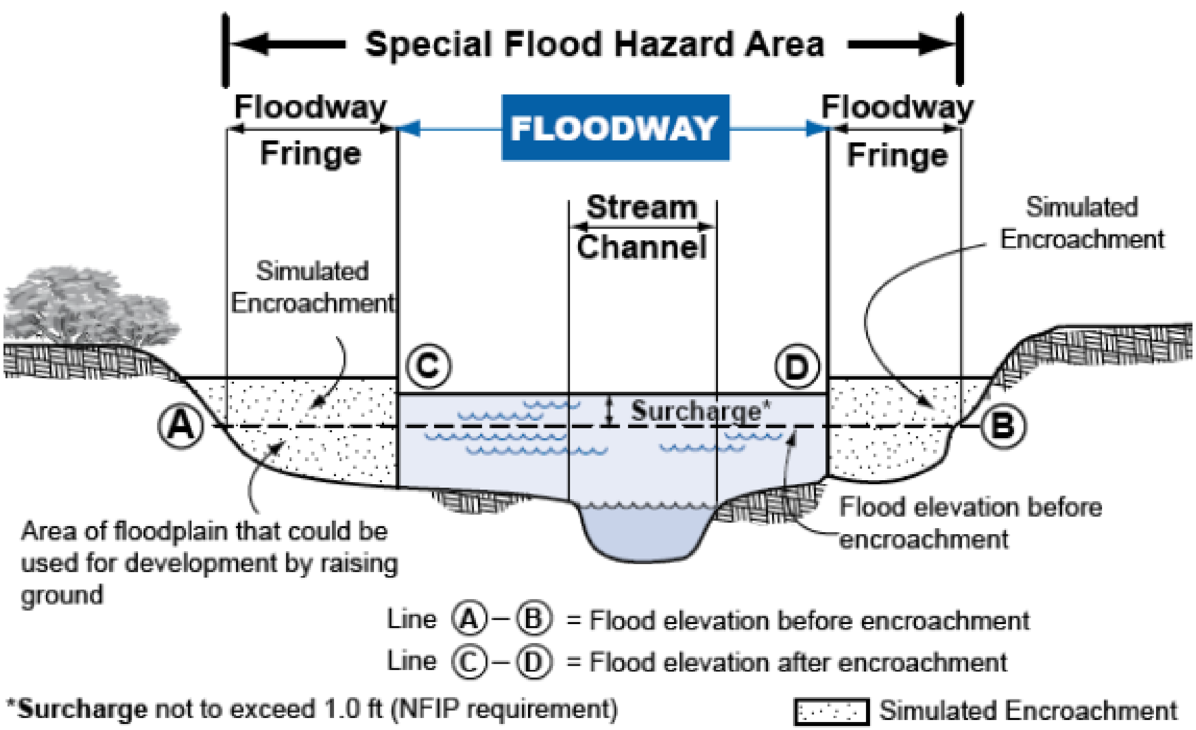
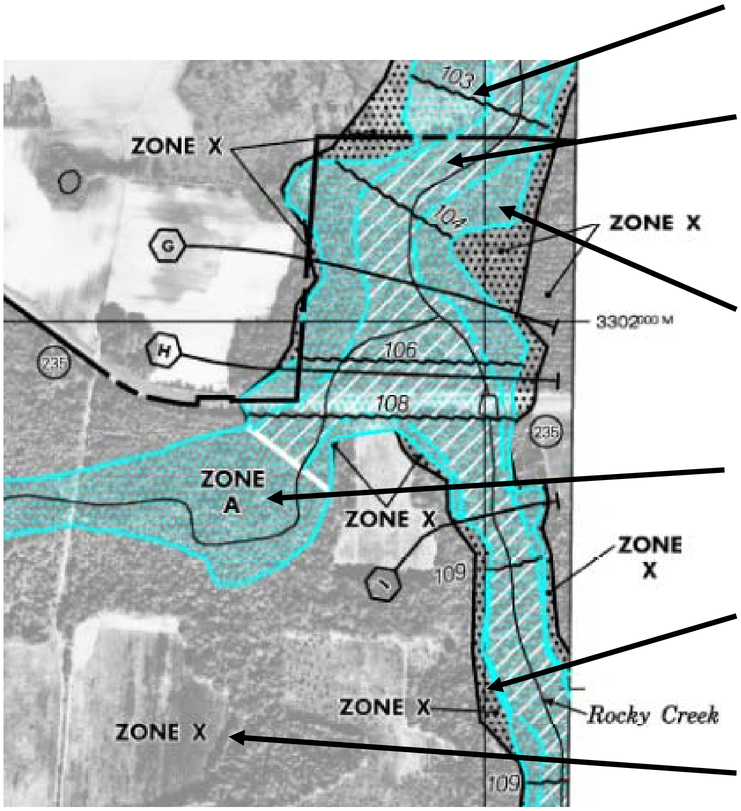


Figure 2-4 Floodway Definitions



Base Flood Elevation
Elevations for the 100-year flood

Floodway
The cross-hatched area. Includes the most conveyance and highest velocities.

Zone AE and Zone A
Zone AE: Subject to flooding by the 100-year flood as determined by a detailed study.
Zone A: Flooding area determined by approximate methods.

Zone X (shaded)
Subject to flooding by the 500-year flood Zone B on some maps.

Zone X (unshaded)
Outside 500-year floodplain.

Figure 2-5 Example Flood Map

The simplest way to be consistent with the NFIP standards for an established floodway is to design the bridge and approach roadways such that their components are excluded from the floodway. If a project element encroaches on the floodway but has a very minor effect on the floodway water surface elevation (such as piers in the floodway), the project may normally be considered as being consistent with the standards if hydraulic conditions can be improved so that no water surface elevation increase is reflected in the computer printout for the new conditions. A No-Rise Certification will need to be prepared and supported by technical data. The data should be based on the original model used to establish the floodway. The FEMA website has contact information to obtain the original model.

A Flood Insurance Study (FIS) documents methods and results of the detailed hydraulic study. The report includes the following information:

- name of community
- hydrologic analysis methods
- hydraulic analysis methods
- floodway data including areas, widths, average velocities, base flood elevations, and regulatory elevations

- water surface profile plots

The FIS can be obtained from the FEMA website. Note that the report does not include the original hydraulic model.

For some rivers and streams, a detailed study was performed, but a floodway was not established (refer to Figure 2-6). The bridge and roadway approaches should be designed to allow no more than a 1 foot increase in the base flood elevation. Information from the FIS and the original hydraulic model should be used to model the bridge, and technical data should be submitted to the local community and FEMA.

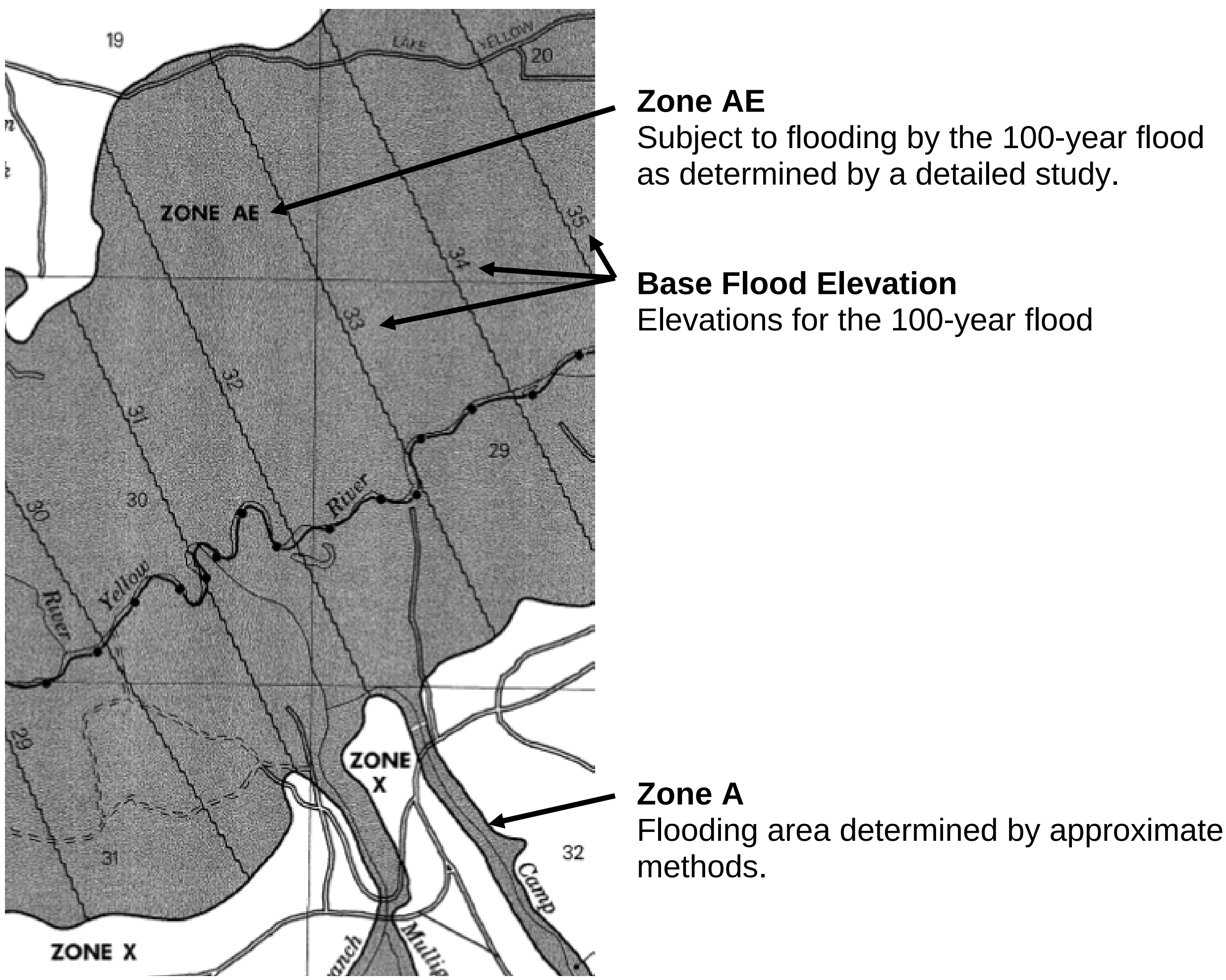


Figure 2-6 Example Flood Map

If the encroachment is in an area without a detailed study (Zone A on Figures 2-5 and 2-6), then technical data should be generated for the project. Base flood information should be given to the local community, and coordination carried out with FEMA where the increase in base flood elevations exceeds one foot in the vicinity of insurable buildings.

2.2.2 Other Government Agency Requirements

Many government agencies (cities, counties, water management districts, etc.) will have additional limitations on backwater conditions in floodplains. The limitations may be designated at multiple distances upstream of the bridge. For example, backwater increase immediately upstream may be limited to one foot, and backwater increase 1000 feet upstream may be limited to 0.1 foot.

Many local agencies have also implemented mitigation requirements for fill within the floodplain. Fill within the floodplain reduces the storage capacity in the floodplain and may increase discharges downstream. Therefore, the local agency may require a compensation area which creates the amount of storage that was lost due to the roadway approach fill.

2.3 Design Frequencies

Design frequency requirements are given in Section 4.3 of the FDOT Drainage Manual. These design frequencies are based on the importance of the transportation facility to the system and allowable risk for that facility. They provide an acceptable standard level of service against flooding.

Criteria that are based on the design frequency include:

- Convey the design frequency without damage (Section 4.2 of the FDOT Drainage Manual)
- Backwater for the design frequency must be at or below the travel lanes (Section 4.4 of the FDOT Drainage Manual)
- Debris clearance

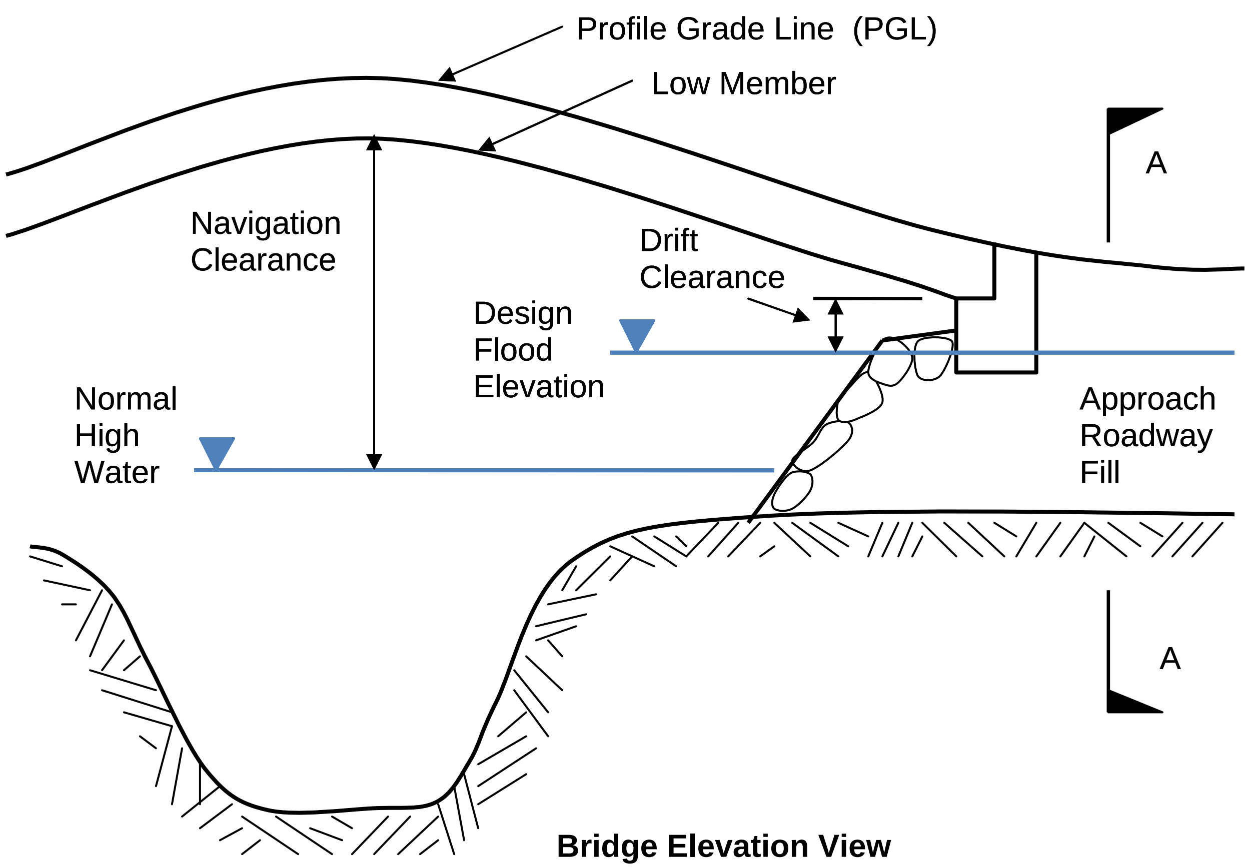
The relationship between the design frequency criteria are shown in Figure 2-7. The criterion naturally tends to create a crest curve on the bridge, with the profile of the approach roadway lower than the bridge profile. This is a desirable profile because the roadway will overtop before the bridge is inundated. Losing the roadway is preferable to losing the bridge.

Backwater criteria also apply for floods other than the design flood:

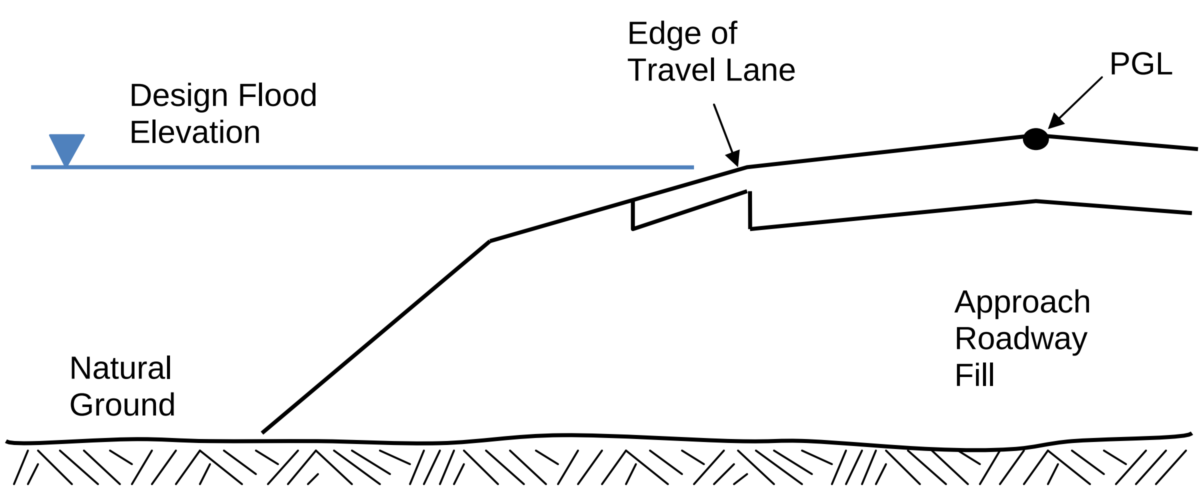
- Backwater must be consistent with the NFIP
- Backwater must not change the land use of affected properties without obtaining flood rights

When the risks associated with a particular project are significant for floods of greater magnitude than the standard design flood, a greater return interval design flood should be evaluated by use of a risk analysis. Risk analysis procedures are provided in FHWA

HEC-17 and discussed briefly in Appendix B, Risk Evaluations of the Culvert Design Handbook. Discuss changing the design frequency with the District Drainage Engineer before making a final decision. In addition, hydraulic design frequency standards of other agencies that have control or jurisdiction over the waterway or facility concerned should be incorporated or addressed in the design.



Bridge Elevation View



Section A - A

Figure 2-7 Bridge and Cross Drain Roadway Grade Controls

Scour analysis and design has a separate design frequency which is discussed in Section 4.9 of the FDOT Drainage Manual. National standards for scour design are found in FHWA HEC-18, Evaluating Scour at Bridges.

The worst case condition for scour will usually occur at overtopping of the approach roadway or another basin boundary. Flow relief is often provided at the bridge due to the overtopping flow and scour conditions will be a maximum at overtopping.

For more guidance on scour computation and design, refer to Chapter 6 of this handbook and the FDOT Bridge Scour Manual.

2.4 Clearances

The span lengths of a bridge affect the cost of the bridge, with longer spans generally increasing the cost. Increased height above the ground increases the cost of the foundations and the earthen fill of the approach roadways. However, minimum clearances both vertically and horizontally must be maintained for the bridge to function properly.

2.4.1 Debris

The two foot minimum debris drift clearance used by the Department traditionally has provided an acceptable level of service. Though this will usually be adequate for facilities of all types, bridge maintenance records should be reviewed for the size and type of debris that may be expected. For example, if the watershed is a forested area subject to timbering activities, sizeable logs and trees should be anticipated. Meandering rivers will also tend to fell trees along its bank, carrying them toward downstream bridge crossings. On the other hand, bridges immediately downstream from pump station may have little opportunity to encounter debris. Also, manmade canals tend to be stable laterally and will fell much less trees than sinuous, moving natural rivers. In such low debris cases, if a reduced vertical clearance is economically desirable, the hydraulic designer should approach the District Drainage Engineer to reduce the drift clearance.

For new bridges, the drainage engineer should advocate for aligning the piers normal to the flow if there is a possibility of debris being lodged between the pilings. The drift clearance is typically shown on the Bridge Hydraulics Recommendation Sheet (BHRS).

2.4.2 Navigation

For crossings subject to small boat traffic, the minimum vertical navigation clearance is set as six feet above the mean high water, normal high water, or control elevation. Notably, other agencies may require different navigational clearances.

For tidally controlled or tidally influenced bridges, the BHR should document the tidal datums for the bridge location. This includes not only the Mean High Water (MHW) for use in navigational clearances, but also any other tidal datums available for the site. If

taken from a tidal bench mark, the BHR should document the bench mark ID as well as the tidal epoch referenced.
Normal High Water is considered to be equivalent to the mean annual flood. The mean annual flood is the average of the highest flood stage for each year. For gaged sites, this information may be obtained from U.S. Geological Survey (USGS). Statistically the mean annual flood is equivalent to the 2.33 year frequency interval (recurrence interval). Therefore, if a synthetic hydrologic method is used to determine the Normal High Water, the 2.33 year event is used. In some cases, stain lines at the site indicating the normal flood levels can be used to estimate the Normal High Water.

Control elevations can be obtained from the regulating agency (water management districts, water control districts, U.S. Army Corps of Engineers, etc.)

2.4.3 Waves

Coastal bridges should be elevated one foot above the design wave crest, as required in the Drainage Manual. If the clearance is less than one foot, which often occurs near the bridge approaches, the bridge must be designed according to AASHTO's Guide Specifications for Bridges Vulnerable to Coastal Storms.

2.5 Bridge Length Justification

The BHR should clearly demonstrate that the proposed structure length and configuration are justified for the crossing. Historical records from the life of the bridge, along with hydrologic and hydraulic calculations, should be used to make recommendations. Using the same length as an existing structure that may have been in place for many years is not justification to use the same bridge length, given that the existing structure may not be hydraulically appropriate and may not have experienced a significant flooding event.

The most effective way to justify the length of a proposed structure is with the analysis of alternate structure lengths. Typical alternative bridge lengths that might be appropriate include:

- existing structure length
- structure length that goes from bank to bank plus 20 feet to provide the minimum maintenance berms
- target velocity structure (for example, an average velocity through the bridge of 2 fps)
- structure that spans the wetlands (the no-mitigation structure length)
- Concrete Box Culvert (CBC) structure
- roadway geometrics structure length

As the analysis proceeds, the need for another length to be analyzed may become apparent, and may turn out to be the proposed structure length.

2.6 Berms and Spill-Through Abutment Bridges

Spill-through abutments are not normally placed in the main channel of a stream or river for several reasons:

- Construction difficulties with placing fill and riprap below water
- Abutment slope stability during and after construction
- Increased exposure to scour
- Environmental concerns
- Stream Stability or Channel Migration
- Maintenance

As stated in Section 4.9 of the Drainage Manual, the horizontal extent of the berms shall be determined using the methods in HEC-23. However, a 10 foot width between the top of the main channel and the toe of spill-through abutment slopes is considered the minimum width necessary to address the above concerns. For stable banks, the horizontal 10 foot measurement should be made from the top edge of the main channel. The use of the minimum berm width does not excuse the drainage engineer from conducting sufficient site analysis to determine the existence of unusual conditions. If the natural channel banks are very steep, unstable, and/or if the channel is very deep, or channel migration exists, additional berm width may be necessary for proper stability. For these conditions the horizontal 10 foot measurement should be made from the point where an imaginary 1V:2H slope from the bottom of the channel intersects the ground line in the floodplain.

In most situations, the structure which provides the minimum berm width will often be the shortest bridge length that will be considered as a design alternative.

The minimum abutment protection is stated in Section 4.9 of the Drainage Manual. The standard rubble riprap was sized in accordance with HEC-23 for flow velocities (average) not exceeding 9 fps, or wave heights not exceeding 3 feet. The horizontal and vertical extent should be determined using HEC-23. A minimum of 10 feet is recommended as a horizontal extent if HEC-23 shows that a horizontal extent less than 10 feet is acceptable. The drainage engineer is advised to review the limits of right-of-way to be sure the apron at the toe of the abutment slope can extend out and along the entire length of the abutment toe, around the curved portions of the abutment to the point of tangency with the plane of embankment slopes. If calculations from HEC-23 show that the horizontal extent is outside the right-of-way limits, the drainage engineer can do the following:

a. Recommend additional right-of-way.
b. Provide an apron at the toe of abutment slope which extends an equal distance out around the entire length of the abutment toe. In doing so, the drainage engineer should consider specifying a greater rubble riprap thickness to account for reduced horizontal extent.

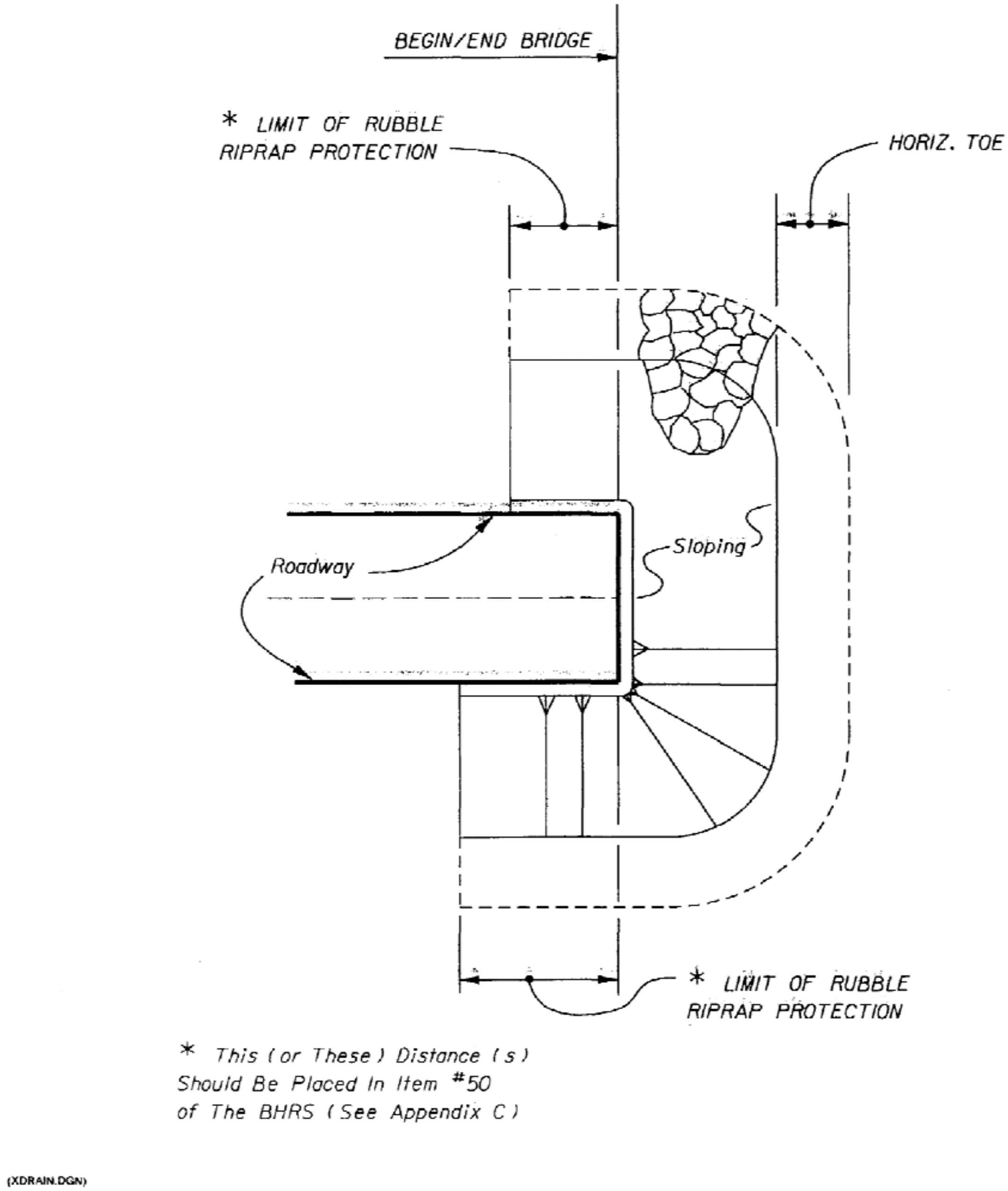


Figure 2-8 – Limits of Rubble Riprap Protection

Figure 2-8 is a plan view which defines the limit of rubble riprap protection. Refer to the FDOT Structures Detailing Manual for the recommended minimum distance.

In contrast, controlled canals in developed areas typically have very low velocities, no stability problems, no overbank flow contracting into the bridge opening, and few abutment maintenance problems. In such cases, the abutment slope usually drops steeply from the abutment directly into the canal.

Use rubble with a specific gravity of 2.65 or other extra heavy revetment where large wave attack is expected, typically in coastal applications. Avoid corrodible metal cabling or baskets in coastal environments; even if coated, the coating may be marred and allow corrosion. Follow USACE Shore Protection Manual for design of coastal revetment.

Bedding stone should be used on all bank and shore rubble installations to guard against tearing of the filter fabric during placement of the rubble. The bedding stone also helps dissipate wave impacts on the revetment.

For revetment installations where wave attack is not expected to be significant, include all options (e.g., fabric-formed concrete, standard rubble, or cabled interlocking block, etc.), which are appropriate based on site conditions. All options shown to be inappropriate for the site should be documented in the BHR. A Technical Specification should be written based on the use of the most desirable revetment material, with the option to substitute the other allowable materials at no additional expense to the Department. This recommendation will help in eliminating revetment Cost Savings Initiative Proposals (CSIP's) during construction.

No matter what options are allowed, the bedding (filter fabric and bedding stone) should be matched to the abutment material. Some of the options are not self-healing, and a major failure can occur if loss of the embankment material beneath the protection takes place.

2.7 Design Considerations for Dual Bridges

When two lane roadways are upgraded to multi-lane divided highways, the existing bridge on the existing roadway often has many years of remaining life. So a new dual bridge is built next to the existing bridge. Years later when the original bridge needs to be replaced, the newer bridge still has years of remaining life. So a cycle of replacing one of the dual bridges at a time is repeated. There is a tendency to keep the bridge ends aligned with the bridge remaining in place. However, consideration should be given to potential lateral migration of the stream and the new bridge end locations should accommodate the stream.

Scour estimates must consider the combined effects of both bridges. Ideally the foundation of the new or replacement bridge will be the same type as the other foundation and will be aligned with the other foundation. In such cases the scour calculations will be similar to that of a single bridge.

In some cases it may not be reasonable to match and align the foundations of both bridges because of such things as economics, geotechnical considerations, and channel migration, etc. If the foundation designs are not the same, or are not aligned, or both, the scour estimates must consider the combined obstruction of both foundations to the flow. The techniques of HEC-18 do not specifically address this situation. If another approach is not available, assume a single foundation configuration that accounts for the obstruction of both foundations and use the techniques of HEC-18. A conservative configuration can be developed by assuming each downstream pile group is moved upstream (parallel to flow) a sufficient distance to bring it in line with the adjacent upstream pile group. Figure 2-9 shows some configurations.

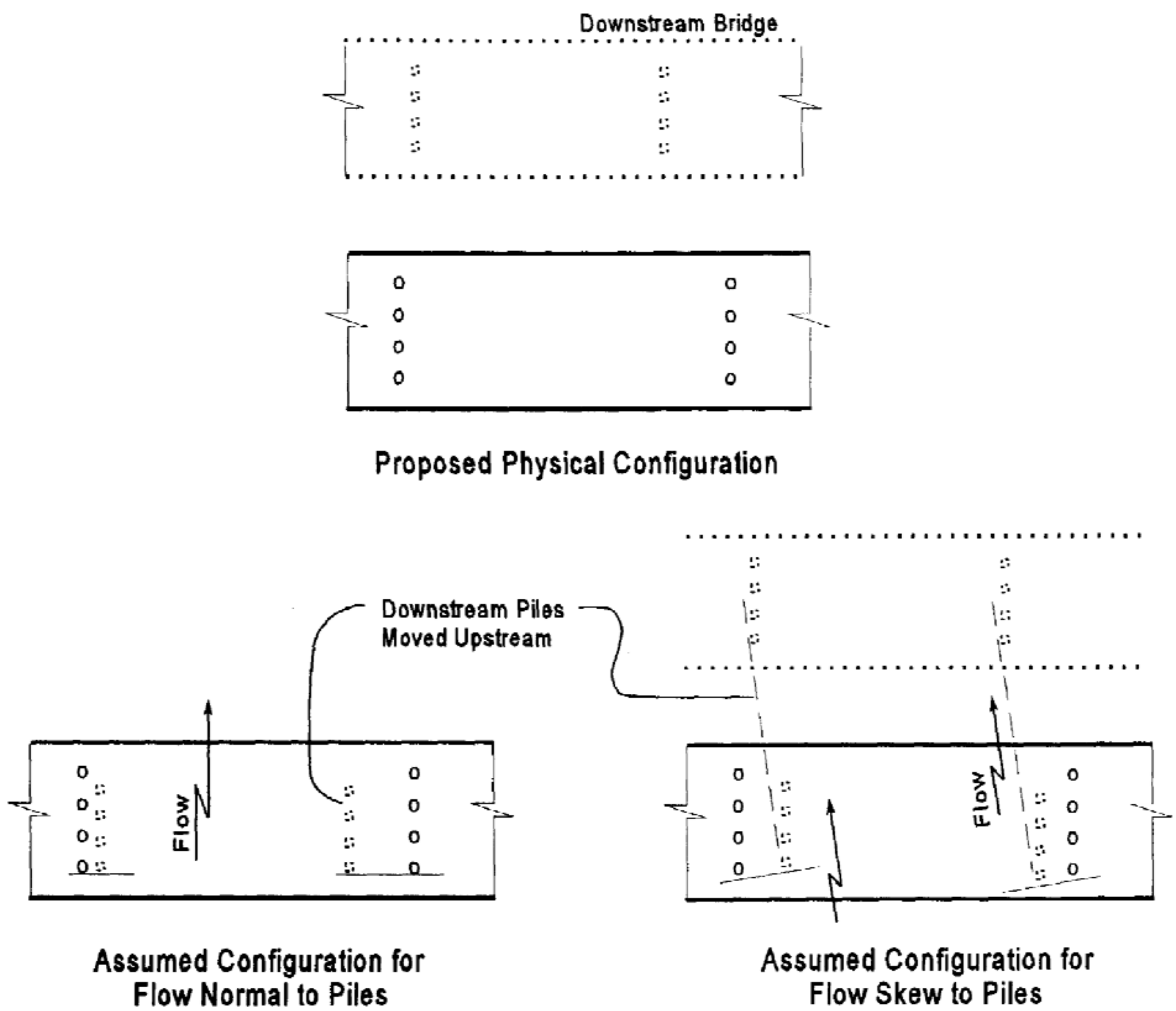


Figure 2-9 Configurations for Computing Scour of Dual Bridges

2.8 Design Considerations for Bridge Widenings

The new substructure or foundations under the widened portion of a bridge are often different than the existing substructure in their shape or depth. If a bridge has been through the Statewide Bridge Scour Evaluation Process and as a part of that process, has been identified as "scour critical", the existing foundation must accommodate the predicted scour. If the existing foundation design cannot accommodate the predicted scour, the first alternative is to reinforce the existing foundation so that it can. If it is not practical to reinforce the existing foundation, the next alternative is to replace the existing structure so that it can be removed from the scour critical list. These approaches are consistent with the goal to remove all bridges from the scour critical list. For minor widening[1] of bridges that have been through the Statewide Bridge Scour Evaluation Process and have not been identified as scour critical, it is acceptable to leave the existing foundation without modification. The foundation under the widened portion must be properly designed to accommodate the predicted scour.

Widening existing bridges will often result in a minor violation of vertical clearances due to the extension of the cross slope of the bridge deck. Consult the District Drainage Engineer in documenting justification for flexing

2.9 Structural Pier Protection Systems

Dolphins and fender systems are two structural systems designed to protect piers, bents, and other bridge structural members from damage due to collision by marine traffic. Dolphins are large structures with types ranging from simple pile clusters to massive concrete structures that can either absorb or deflect a vessel collision. They are typically located on both sides of the structure being protected as shown in Figure 2-10. Fender system types are less variable, consisting usually of pile-supported wales, as shown in Figure 2-11. Fender systems are typically wrapped around the protected piers and run along the main navigation channel.

[1] Minor bridge widening is defined in the FDOT Structures Design Guidelines.

Figure 2-10 Dolphin Pier Protection at the Sunshine Skyway Bridge

Figure 2-11 Fender System at the Old Jewfish Creek Bridge

For design purposes, scour around dolphins can be calculated in the same manner as bridge piers. Typically dolphins are located sufficiently far from the piers so that local scour is calculated independently. However, the engineer should check to ensure sufficient spacing (greater than 10 effective diameters).

Scour at fender systems is typically taken as equal to that of the pier it is protecting. In some cases, fender systems may “shield” bridge piers, reducing velocities and scour at the pier. However, this shielding effect can vanish or be modified if the fender system is lost due to collision or unforeseen scour problems, or if the flow attack angle is skewed so that the pier is not in the hydraulics shadow of the fender system. Piers and fender systems introduced into relatively narrow rivers may cause contraction scour between the fender systems. This scour is usually greatest near the downstream end of the system.

Chapter 3

Riverine Analysis

A riverine analysis applies to inland streams and rivers. Flooding conditions for riverine systems result from runoff from extreme rainfall events. Steady-state flow conditions can usually be assumed.

3.1 Data Requirements

The data collected will vary depending on the site conditions and the data available. Two-dimensional models require substantially more data than one-dimensional models.

3.1.1 Geometric Data

The following steps should be followed to collect geometric data for the analysis:

1. Determine the model domain. The geometric data must extend far enough upstream, downstream, and laterally to provide an accurate representation of the terrain within the domain. Refer to Section 3.4.1 for guidance.
2. Locate available geometric data within the model domain. Liberally estimated boundaries of the domain can be used when the cost of collecting existing data is low.
3. Order survey for those portions of the model domain that do not have adequate coverage from existing geometric data. Survey will be expensive, so the domain boundaries should be more conservatively estimated.

3.1.1.1 Existing Geometric Data

There are many potential sources of geometric data, and new sources of data continually become known. The following is a list of potential sources:

- USGS
    - Quadrangle Maps
        - A public source in both scanned and vector formats is the FDEP Land Boundary Information System (LABINS) located at:
    - http://www.labins.org/ Digital Elevation Models (DEM)
        - Digital Elevation Models are essentially x,y,z coordinate points on a 90 meter grid. They were derived from the Quadrangle Maps
        - DEMs are also available at LABINS.
    - LiDAR
        - Coverage in Florida is not yet complete. Available data can be downloaded at: http://lidar.cr.usgs.gov/
- U.S. Army Corps of Engineers
    - USACE performs hydrographic surveys on navigable waterways which can provide main channel information.

- Mobile District: http://navigation.sam.usace.army.mil/surveys/index.asp
- Jacksonville District: Contact directly

- Florida Department of Emergency Management
    - Data for the Florida Coastal LiDAR project and links to other compatible data: http://www.floridadisaster.org/gis/lidar/
- Water Management Districts
- Cities and Counties
- Old Plans and BHRs
- FEMA studies
    - Refer to Section 3.1.5 for more information on how to determine if a detailed study is available.

USGS Quadrangle Maps and DEMs are available for the entire state of Florida. They may be useful for preliminary analysis and in some circumstances may be used to fill in gaps further away from the site.

The remaining data sources will usually have a level of accuracy that was adequate for hydraulic modeling at the time of collection. However, the age of the data should be considered. If the terrain within the model domain has changed significantly, then newer existing data sources must be found or survey will be required.

Data from different sources may be needed to cover the entire model terrain. Sometimes one source will have data within the overbank and floodplain areas, and a different source will have hydrographic data within the channel. Be sure to convert all data to a common datum and projection.

3.1.1.2 Ordering Survey Data

The FDOT Surveying Handbook (dated October 31, 2003) states that bridge survey and channel survey requirements are project specific. Thus, the hydraulic designer should provide instructions to the surveyors, which are site specific, so that the surveyor does not default to the previously used Location Survey Manual.

Survey can be in either cross section or Digital Terrain Model (DTM) format for one-dimensional models. Although cross sections can be used to develop two-dimensional models, a DTM format is preferable. Discuss the survey format with the surveyor to determine which format is most appropriate.

Survey should always be ordered in the immediate vicinity of the proposed bridge. The accuracy needs in this area are greater than the accuracy needs of the hydraulic model for two reasons:

1. Bridge and roadway construction plans need a higher degree of accuracy
2. The approach roadway and bridge abutment, including abutment protection, must fit within the right-of-way.

The typical roadway survey will be a DTM within the proposed right-of-way, and may extend a minimal distance outside of the proposed right-of-way. Coordinate with the roadway design engineer.

The location of the approach and exit cross sections for the model should be determined, and survey information in the main channel should be extended to these locations. Additional survey information in the adjacent floodplain and further upstream and downstream of these extents will depend upon the other available geometric data.

The hydraulic designer should provide a sketch to the surveyor on a topographic map or aerial showing the limits of the DTM or the location, orientation, and length of cross sections. The surveyor should also be asked for:

- Survey(s) of any adjacent utility crossings.
- Elevations of stains on the existing pilings.
- Any high water marks determined by the hydraulics engineer during the site visit.
- Elevation of the water level on the day of the survey.

When ordering survey, remember that most floodplains in Florida often have dense vegetation. Surveying in these areas will be difficult. Not all cross sections need to be surveyed at the actual location used in the hydraulic model. Surveyed cross sections can be reasonably manipulated into model cross sections, so look for areas that would be easier to survey, such as along power lines and open fields.

3.1.2 Geotechnical Data

In order to calculate scour at bridge foundations, geotechnical information is required to establish the bed composition and its resistance to scour. Near surface bed materials in Florida range from sand and silts to clays to rock. As will be discussed in Chapter 6, the composition of the bed material dictates the procedure employed in the calculation of scour. For scour studies, the required information is a characterization of the near surface bed material: i.e., the layer over which scour will occur. The thickness of this layer will be a function of the expected scour at the site.

For bridges with foundations in cohesionless sediments (sands and silts), the geotechnical data collection should include sieve analyses to characterize the size of the bed sediments. One should obtain a sufficient number of samples to confidently characterize the sediment size both over the length of the bridge as well as over the thickness of the expected scour layer. The parameter from the sieve analyses necessary for scour calculation is the median grain size (D_{50}).

NRCS Soil Surveys can provide an estimated median grain size for preliminary scour estimates.

For bridges with foundations in non-cohesionless sediments (rock or clay), one must establish the bed material's scour resistance. For rock, the FHWA provides guidelines for scourability of rock formations in the technical memorandum HNG-31:

http://www.fhwa.dot.gov/engineering/hydraulics/policymemo/rscour.cfm

For substrates that do not meet these criteria, scour calculation will follow the FDOT Rock Scour Protocol:

http://www.dot.state.fl.us/rddesign/Drainage/Fla-Rockclay-Proc.shtm
The referenced protocol recommends obtaining core borings at each pier for testing at the State Materials Office. It is the responsibility of the engineer to follow the protocol procedure when encountering soils of this type.

For smaller streams where a bridge culvert may be an appropriate hydraulic option, consider obtaining a preliminary soil boring to determine if increased foundation costs for the culvert need to be included in the alternatives cost comparisons.

3.1.3 Historical Data

Historical data provides important information for many aspects of the bridge hydraulics and scour analysis. It provides data for calibration through gage measurements and historical high water marks, data for calculation of long-term scour processes through historical aerial photography and Bridge Inspection Reports, and characterization of the hurricane vulnerability through the hurricane history.

Speak with local residents, business owners and employees, and local officials including fire and emergency services to obtain anecdotal information about past floods. This information can be very important in the absence of other historical data.

3.1.3.1 Gage Measurements

Gage data can be used in a number of ways in bridge hydraulics analysis.

- Gage data can be used to determine the peak flow rates, although the Department usually relies upon agencies such as the USGS to perform statistical analysis of the stream flow data. Refer to Section 2.2 of the FDOT Hydrology Handbook for more information.
- If the gage is downstream of the bridge, the gage data can provide starting water surface elevations, or boundary conditions, for the model. Refer to Section 3.4.1.2 and Section 3.4.4 for more information.
- Gage data can be used to calibrate the model. Refer to Section 3.5.1 for more information.

If the gage is located at a distance from the bridge site, the gage flow rates may not be the same as the bridge flow rates. However, the gage data may still be useful if the flow rates can be adjusted. Refer to Section 4.5 Peak Flow Transposition in FHWA Hydraulic Design Series 2, Highway Hydrology for more information.

USGS gage information can be found at the following website: http://fl.water.usgs.gov/

Gage data may also be available from the water management districts and other local agencies.

3.1.3.2 Historical Aerial Photographs

Historical aerial photographs provide a means to determine the stream stability at a highway crossing. Comparison of photographs over a number of years can reveal long-term erosion or accretion trends of the shorelines and channel near the bridge crossing. Current aerial photographs can also be used as a base for figures in the Bridge Hydraulics Report, showing such things as cross section locations and upstream and downstream controls.

Recent and current aerial photographs can be found at many internet sites. Be careful of copyright infringements when using these aerials in the Bridge Hydraulics Report. For this reason, it is probably best to obtain the photographs from government sites that give free access.

Older aerial photographs can be obtained from the Aerial Photography Archive Collection (APAC), maintained by the FDOT Surveying and Mapping Office. APAC includes aerials dating back to the 1940's. Ordering information can be found at the following link:

http://www.dot.state.fl.us/surveyingandmapping/aerial_main.shtm
The University of Florida also maintains a database of older aerial photographs:
http://ufdc.ufl.edu/aerials

Another useful site to obtain aerial photographs is the FDEP Land Boundary Information System (LABINS) which can be accessed at the following link:

http://www.labins.org/

3.1.3.3 Existing Bridge Inspection Reports

The District Structures Maintenance Office is responsible for the inspection of each bridge in the state, including bridges owned by local agencies, at regular time intervals. The reports will document any observed hydraulically related issues, such as scour or erosion around the piers or abutments. Bridge Inspection Reports can be obtained from the District Structures Maintenance Office. Of particular interest will be the channel profiles that have been collected at the site, which may show any channel bottom fluctuations over time.

The channel profiles are usually created by taking soundings from the bridge deck. Soundings are measurements taken using a weighted tape measure to keep the tape vertical. The measurements are the distance from a consistent point on the bridge (usually the bridge rail) to the stream bed. The measurements are made on both sides of the bridge at each bridge pier and often at midspan.

The Phase 1 Scour Evaluation Report may also be available for existing bridges. This report will plot some of the bridge inspection profiles against the cross section from the original construction, assuming that old plans or pile driving records were available to obtain the original cross section. The example bridge shown in Figures 3-1 and 3-2 has a very wide excavated cross section beneath the bridge. This was a common bridge design practice before dredge and fill permitting requirements brought the practice to an end unless the required wetland impact was justified and mitigated. In the example, the widened channel has filled back in and narrowed since the initial construction in 1963.

The channel profiles can be used to determine long-term bed changes at the bridge site.

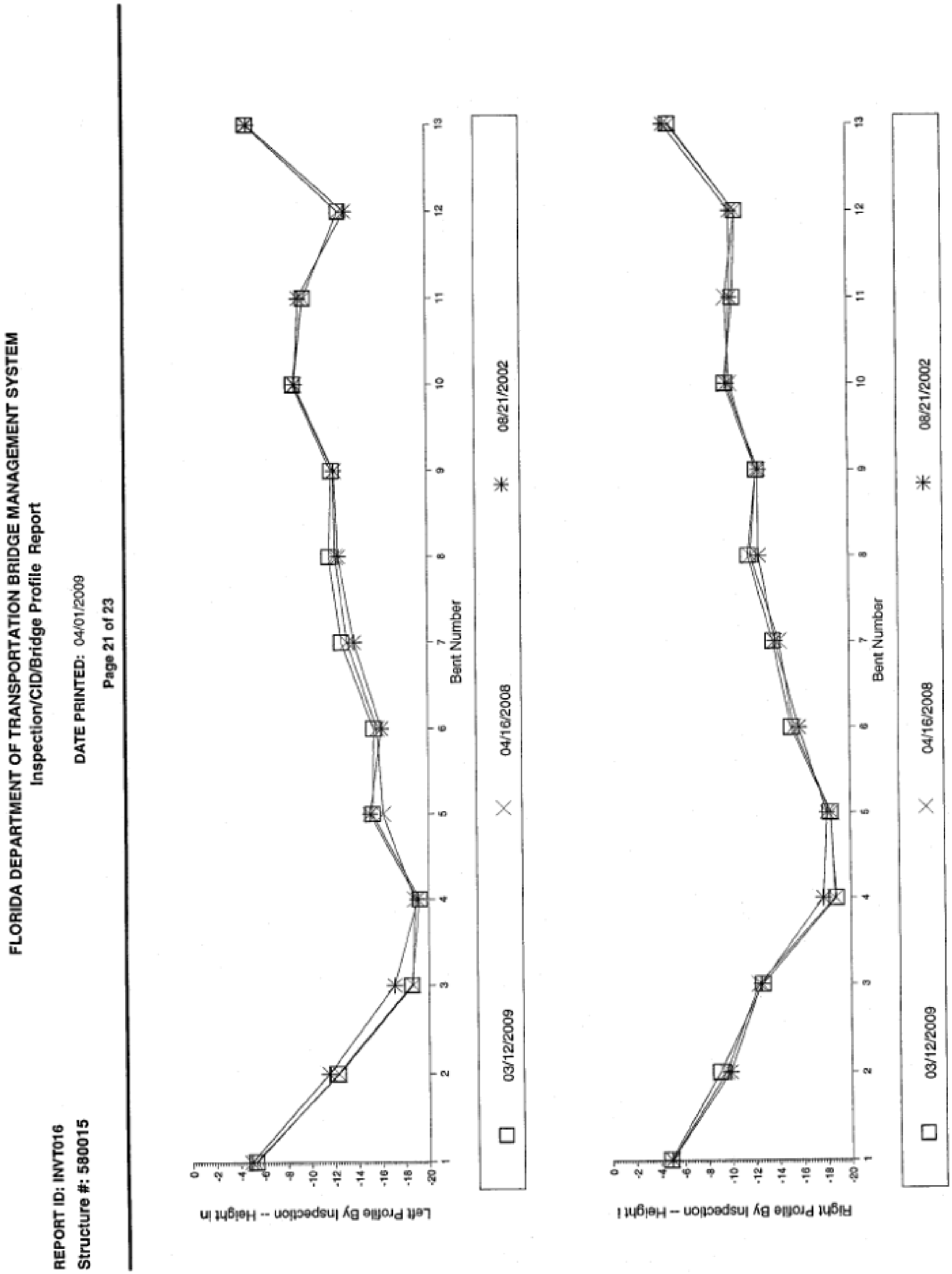


Figure 3-1 Example Bridge Profile from a Bridge Inspection Report

FLORIDA DEPARTMENT OF TRANSPORTATION BRIDGE MANAGEMENT SYSTEM
Inspection/CID/Bridge Profile Report

REPORT ID: INVT016
Structure #: 580015

DATE PRINTED: 04/01/2009

Page 22 of 23

Profile Data - Numerical Summary

Inspection Date and Key: 03/12/2009 CXYM

(All Heights Are In Feet)

Bent #	Left Height	Right Height
1	5.3	4.8
2	12.2	9
3	18.5	12.6
4	19.2	18.8
5	15.3	18.3
6	15.4	15.1
7	12.7	13.6
8	11.7	11.6
9	11.9	12.3
10	8.7	9.7
11	9.6	10.4
12	12.6	10.6
13	4.8	5

Air Temp:
Profile Notes:
Waterway Measurements: Top of rail to water line at Bent 4; 16.5 ft left and right.
Groundline Measurements from top of rail.

Inspection Date and Key: 04/16/2008 HOWV

Bent #	Left Height	Right Height
1	5.2	4.9
2	12.3	9.5
3	18.6	12.3
4	18.8	18.7
5	16.2	18.4
6	15.7	15.3
7	13.2	14.1
8	12.2	11.8
9	12.1	12.3
10	8.8	10.1
11	9.6	9.8
12	12.6	10.6

Figure 3-1 (cont.) Example Bridge Profile from a Bridge Inspection Report

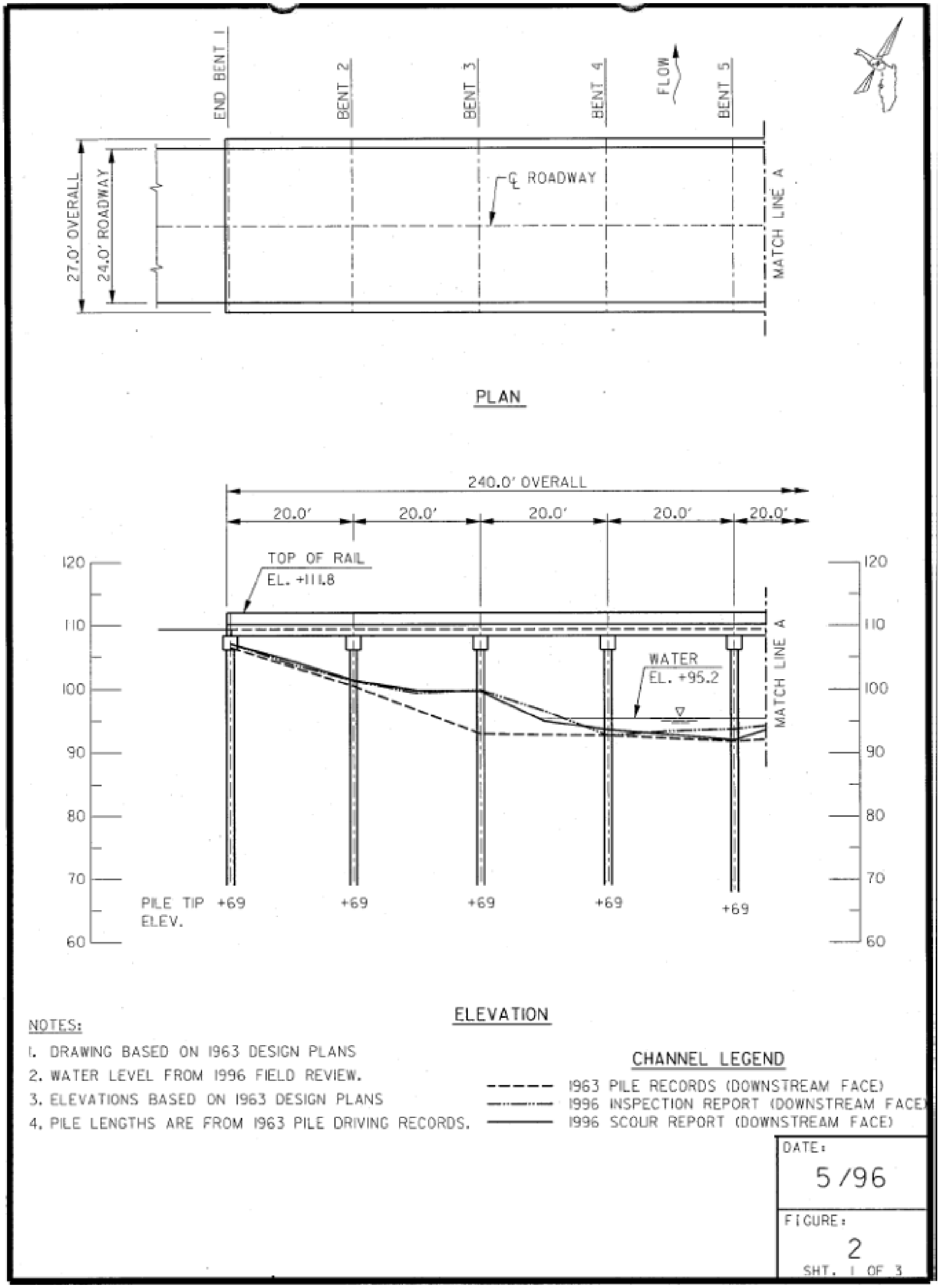


Figure 3-2 Excerpt from Scour Evaluation Report

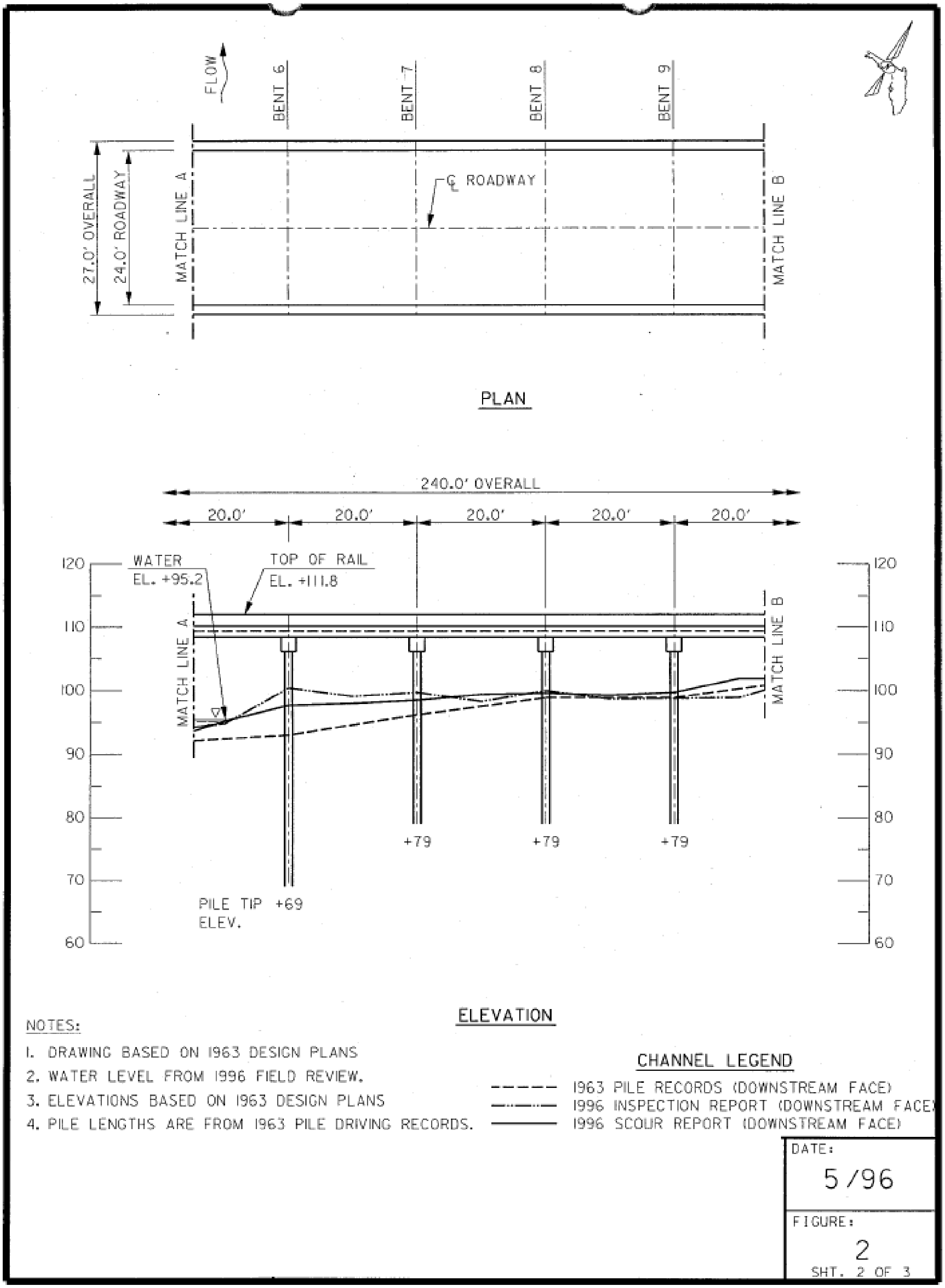


Figure 3-2 (cont.) Excerpt from Scour Evaluation Report

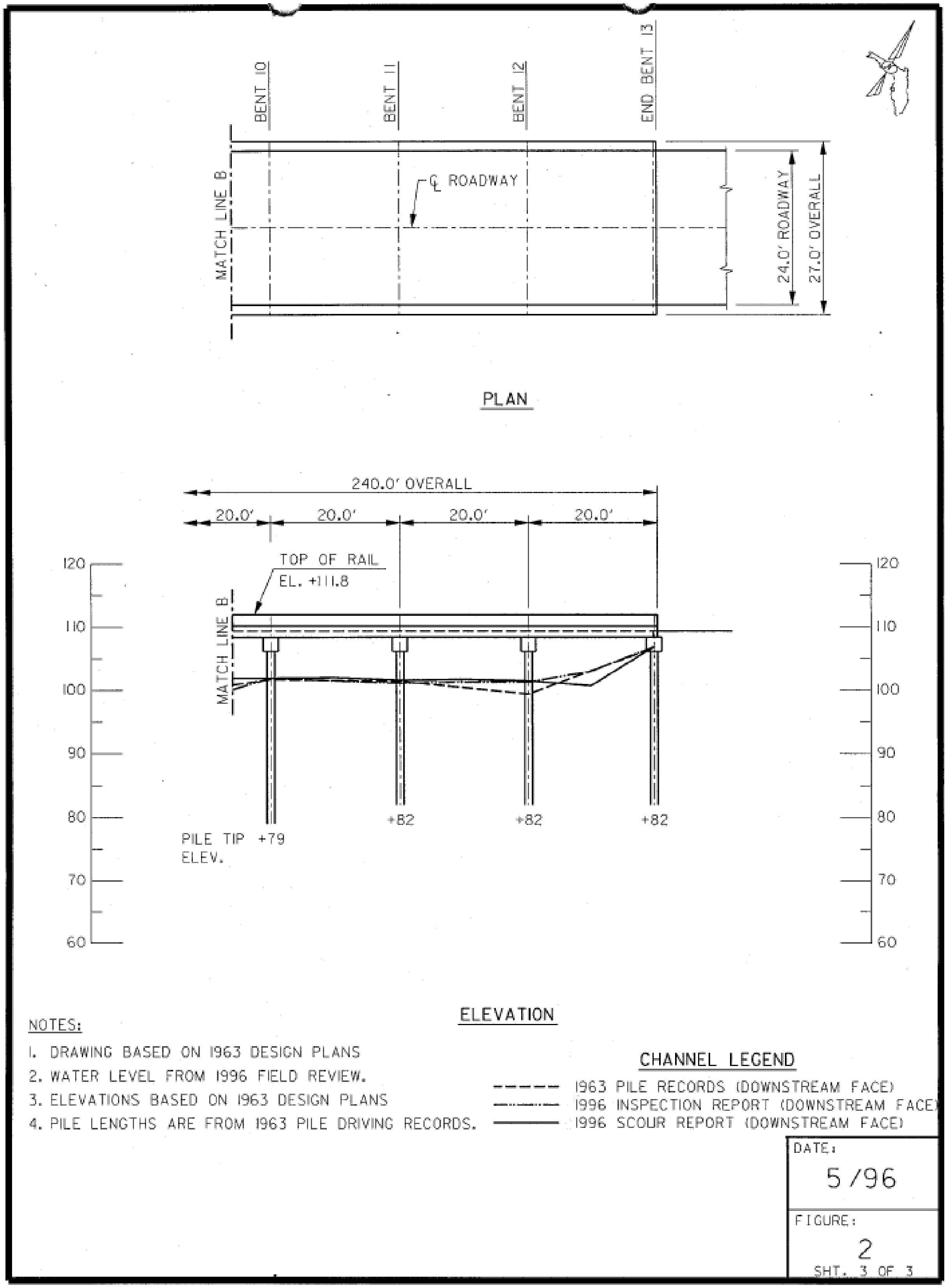


Figure 3-2 (cont.) Excerpt from Scour Evaluation Report

3.1.3.4 Previous Studies

If the project replaces or widens an existing bridge, the BHR or other hydraulic calculations for the existing bridge should be obtained, if possible. Other BHRs for bridges over the same water body may also provide useful information.

If a detailed study was performed by FEMA, then the Flood Insurance Study, the NFIP Maps, and the original model should be obtained (refer to Section 3.1.5).

Additional sources of existing studies can include the water management districts, the Florida Department of Environmental Regulation, counties, and the U.S. Army Corps of Engineers.

3.1.3.5 Maintenance Records

Contact the local district or local agency maintenance staff for bridge inspection reports, historical overtopping, and/or maintenance issues at the bridge site.

3.1.4 Drainage Basin Information

Drainage basin information is needed for the hydrologic analysis. The type of information collected depends upon the hydrologic method used in the analysis. Refer to Section 3.2 below, and the FDOT Hydrology Handbook, for guidance on the hydrologic analysis and data requirements.

The drainage basin boundaries should be delineated on the Bridge Hydraulics Recommendation Sheet. Federal, state, and local agencies, including the water management districts, often publish basin studies and delineate basin areas. Many of these are available online. Verify the boundaries found on older maps.

Information should also be gathered on other structures on the river upstream and downstream of the proposed bridge site. The information gathered should include the size and type of structure for comparison with the proposed structure.

3.1.5 FEMA Maps

The FEMA Flood Insurance Rate Map and the Flood Insurance Study for the site should be obtained. These maps can be ordered or downloaded from the FEMA Map Service Center at the following link:

http://msc.fema.gov/

Backup and supporting data for a detailed study, if the area has a detailed study, can also be obtained from FEMA. At the time of this writing, this information cannot be ordered through the website. Call the FEMA Map Service Center for ordering information.

3.1.6 Upstream Controls

Upstream controls may influence the discharge at the crossing. Pump stations and dams are two common controls. Salinity intrusion structures are another example. The agency exercising control over these structures should be contacted to obtain information regarding geometrics, intended mode of operation, flow rate data, and history including structure failures. It is important to consider the likelihood of upstream structure failures when considering flow regimes. A dam break analysis may be appropriate.

3.1.7 Site Investigation

A field investigation is recommended for all new bridge construction. Data obtained during a field investigation can aid in hydraulic model construction, identify problem erosion areas, and characterize stream stability. A field investigation should be performed during the early stages of design. The following checklist (Neill [1973]) outlines some key items of basic data to be collected (not all may apply to a particular site):

- Look for channel changes and new tributaries compared to the latest aerial photographs or maps from the office data collection
- Look for evidence of scour in the area of the existing structure and check the adequacy of existing abutment protection
- Check for recent repairs to the existing abutment protection (as compared with the age of the bridge)
- Check for local evidence of overflow or breaching of the approaches
- Search the site for evidence of high flood levels, debris, or stains on the structure that may indicate flood levels
- Search for local evidence of wave induced erosion along the banks
- Note the velocity direction through the bridge and estimate the velocities (note the date and time of these observations)
- Photograph the channel and adjacent areas
- Seek evidence of the main overflow routes and flood relief channels
- Search for hydraulic control points upstream and downstream of the structure
- Assess the roughness or flow capacity of the floodplain areas
- Describe and photograph the channel and overbank material in situ
- Seek evidence on largest size of stone moved by flood or waves
- Seek local evidence of channel shifting, bank and shore erosion, etc. and their causes
- Seek local evidence of channel bed degradation or aggradation

- Seek evidence of unrecorded engineering works that would affect flows to the bridge such as dredging, straightening, flow diversions, etc.
- Observe the nearby land uses that might be affected by flood level changes

Consider visiting other structures across the stream or river upstream and downstream of the proposed bridge site.

3.2 Hydrology

In most riverine analyses, steady-state conditions will be assumed and the hydraulic analysis will be performed using the peak discharge for each frequency analyzed. The peak discharge may vary at different locations on the stream if there are tributaries within the reach, but each discharge will be assumed to remain constant with respect to time.

The criteria for selecting discharges used for riverine analysis are given in Section 4.7 of the FDOT Drainage Manual. Further guidance is given in the FDOT Hydrology Handbook.

Generally, the length of the structure does not control the hydrology. That is, in general, a longer structure will not significantly increase the discharge downstream. When considering the inaccuracies associated with the hydrology, the effect of the structure length and the resulting backwater (or reduction of backwater) will not usually significantly affect the amount of water going downstream. However, if the hydraulics engineer or regulatory agency is significantly concerned about this effect, then an analysis should be conducted to verify the concern. The pre and post water surface profiles can be calculated and routed with an unsteady flow model.

3.3 Model Selection

Before selecting a specific model to use at a given bridge site, two general decisions must be made to isolate groups of appropriate models. Two basic decisions are:

1. One-dimensional or two-dimensional
2. Steady flow conditions or unsteady flow conditions

3.3.1 One- verses Two-Dimensional

The accuracy of one dimensional model depends upon the ability of the modeler to visualize the flow patterns during the design events in order to properly locate the model cross sections. Complicated flow patterns caused by site factors such as skewed approach embankments, multiple openings, other nearby crossings, and the presence of bends, meanders, and confluences within the reach, may indicate that a two-dimensional model may be more appropriate.

3.3.2 Steady verses Unsteady Flow

An unsteady flow model should be used for the following conditions:

- Mild stream slopes less than two feet per mile. If the slope is greater than five feet per mile, steady flow can be used. For slopes between these values, consider the cost and complexity of an unsteady model verses the cost importance of the bridge.
- Situations with rapid changes in flow and stage. Models of dam breaks are the primary example of this situation.
- Bifurcated streams (streams where the flow divides into one or more channels and recombines downstream).

More information on these situations can be found in USACE Manual EM 1110-2-1416, River Hydraulics.

3.3.3 Commonly Used Programs

The most commonly used one-dimensional models are HEC-RAS and WSPRO. HEC-RAS was developed by the U.S. Army Corps of Engineers Hydrologic Engineering Center for a number of river hydraulic modeling applications, including the hydraulic design of waterway bridges. "WSPRO" (Water Surface PROfile) is the acronym for the computer program developed by FHWA specifically for the hydraulic design of waterway bridges. The drainage engineer should always ensure the latest version is being used and document the version in the Bridge Hydraulics Report.

HEC-RAS and WSPRO are both suitable to analyze one-dimensional, gradually varied, steady flow in open channels and can also be used to analyze flow through bridges and culverts, embankment overflow, and multiple-opening stream crossings. HEC-RAS has the additional capability of analyzing unsteady flow.

The WSPRO program analyzes unconstricted valley sections using the standard step method, and incorporates research for losses across a bridge constriction. HEC-RAS allows the user to select the method used to analyze the bridge losses, including energy (standard step), momentum, Yarnell and WSPRO methods. Both programs allow the drainage engineer to readily analyze alternate bridge openings. The output provides water surface elevations, bridge losses, and velocities for both the constricted (with bridge) and the unconstricted (with no bridge) condition. This information can be used to estimate the backwater effects of the structure and provides input information for scour analysis.

The most commonly used two-dimensional models are FESWMS and RMA 2.

The Finite Element Surface Water Modeling System (FESWMS) was originally developed for the Federal Highway Administration (FHWA) and the United States Geological Survey (USGS). The FHWA has continued to maintain and sponsor development of subsequent versions, which continue to incorporate features specifically designed for modeling highway structures in complex hydraulic environments. As such, it includes many features that other available two-dimensional models do not have, such as pressure flow under bridge decks, flow resistance from bridge piers, local scour at bridge piers, live-bed and clear-water contraction scour at bridges, bridge pier riprap sizing, flow over roadway embankments, flow through culverts, flow through gate structures, and flow through drop-inlet spillways. FESWMS can perform either steady-state or unsteady flow modeling.

RMA2 is a two-dimensional, unsteady, depth-averaged, finite-element, hydrodynamic model. It computes water surface elevations and depth-averaged horizontal velocity for subcritical, free-surface flow in two-dimensional flow fields. The program contains the capability of solving both steady- and unsteady-state (dynamic) problems. Model capabilities include: wetting and drying of mesh elements; including Coriolis effects; applying wind stress; simulating five different types of flow control structures; and applying a wide variety of boundary conditions. Applications of the model include calculating water levels and flow distribution around islands; flow at bridges having one or more relief openings; in contracting and expanding reaches; into and out of off-channel hydropower plants; at river junctions; and into and out of pumping plant channels; circulation and transport in water bodies with wetlands; and general water levels and flow patterns in rivers, reservoirs, and estuaries.

3.4 Model Setup

The following data will be required to perform the hydraulic and scour analysis for a bridge crossing:

- Geometric Data
- Flow Data (upstream boundary)
- Loss Coefficients
- Starting Water Surface Elevations (downstream boundary)
- Geotechnical Data (D_{50} soils information)

3.4.1 Defining the Model Domain

The upstream, downstream, and lateral study boundaries are required to define the limits of data collection. The model must begin far enough downstream to assure accurate results at the bridge, and far enough upstream to determine the impact of the bridge crossing on upstream water surface elevations. The lateral extent should ensure that the model includes the area of inundation for the greatest flood analyzed. Underestimating the domain can cause the water surface calculations to be less accurate than desired or require additional survey at a higher cost than the inclusion in the initial survey. Overestimation can result in greater survey, data processing, and analysis cost.

3.4.1.1 Upstream

At a minimum, the upstream boundary should be set far enough upstream of the bridge to encompass the point of maximum backwater caused by the bridge. If a point of concern where the water surface elevation must be known is further upstream, then the model must be extended to that point. An example would be upstream houses or buildings because the 100-year water surface elevation must be kept below their floor elevation. Check with permitting agencies, including cities and counties, as some have limits on the amount of backwater allowed at a given distance upstream.

Equation 3-1 can be used to determine how far upstream data collection and analysis needs to be performed.

$$Lu = 10000 * HD^{0.6} * HL^{0.5} / S \qquad \text{(Eq. 3-1)}$$

where:

- Lu = Upstream study length (along main channel) in feet for normal depth starting conditions
- HD = Average reach hydraulic depth (1% chance flow area divided by cross section top width) in feet
- S = Average reach slope in feet per mile
- HL = Headloss ranging between 0.5 and 5.0 feet at the channel crossing structure for the 1% chance flow

The values of HD and HL may not be known precisely since the model has not yet been run to determine these values. They can be estimated from FEMA maps, USGS Quadrangle Maps (or other topographic information).

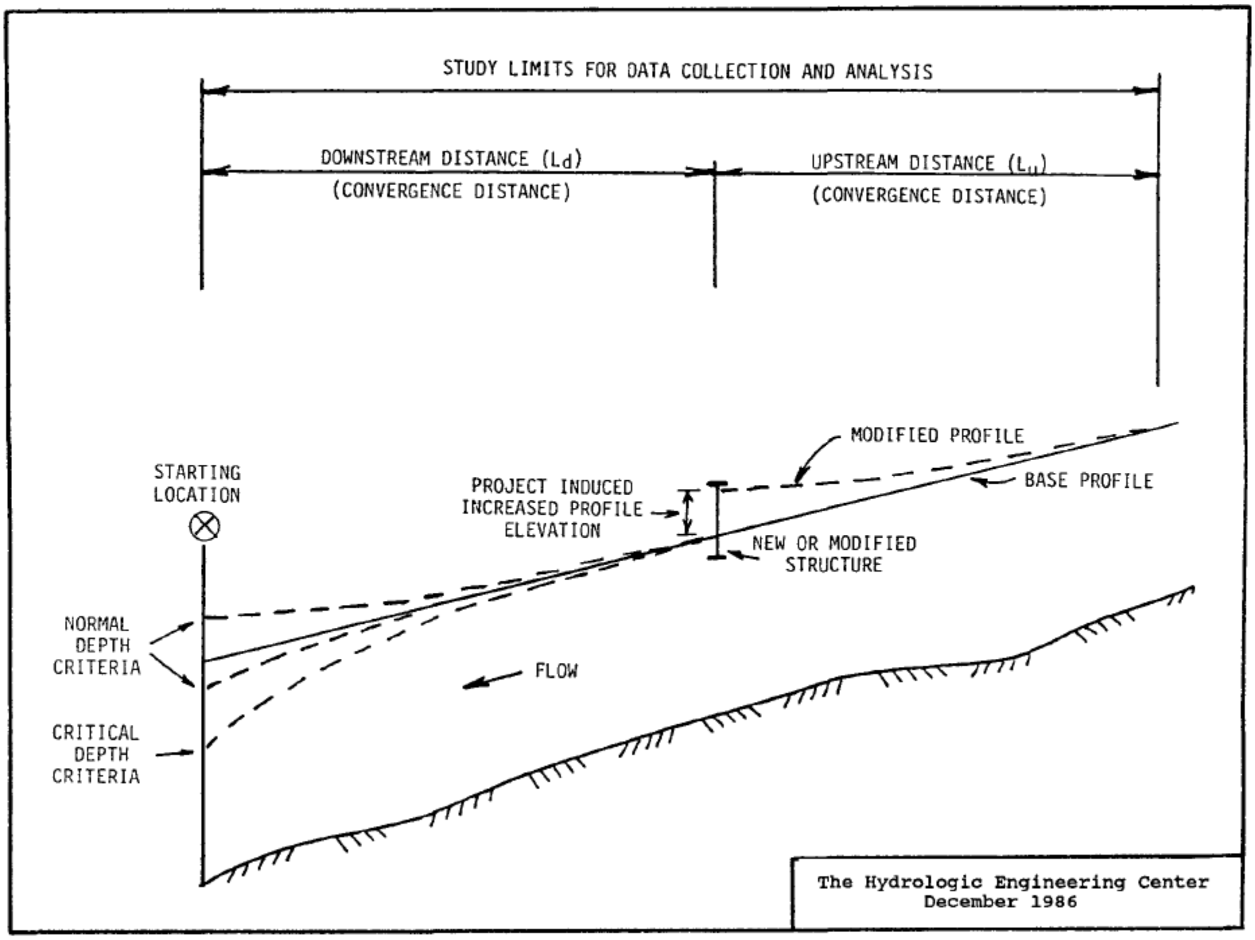


Figure 3-3 Open Channel Depth Profiles

3.4.1.2 Downstream

Open channel hydraulics programs must have a starting water surface elevation specified by the user at the downstream boundary of the model. The programs allow for one or more of the following methods of specifying the starting water surface elevation:

- Enter a water surface elevation at the downstream boundary.
- Enter a slope at the downstream boundary which is used to calculate the normal depth from Manning's Equation.
- Assume critical depth at the downstream boundary.

The modeler must decide which method will be used, and the decision will affect the distance to the downstream boundary of the model.

For the storm frequency being modeled, if a point of known water surface elevation is within a reasonable distance downstream, the model should be extended to that point.

Refer to the section below on convergence for guidance on determining if the point is within a reasonable distance.

Gages are points with a known relationship between the discharge and the water surface elevation. Lakes and sea level can also be points of known elevation. Other locations where the water surface elevation can be calculated from the discharge can include weirs, dams, and culverts if these locations are not significantly influenced by their tailwater.

The normal depth assumption to determine the starting water surface elevation can be used when the downstream channel and overbank is nearly uniform, both in cross section and slope, for a long reach downstream. The length of uniform channel that will be adequate will vary with the slope and properties of the channel, and can be estimated using Equation 3-2. This reach should not be subject to significant backwater from further downstream.

Equation 3-2 can be used to determine how far downstream data collection and analysis needs to be performed.

$$Ldn = 8000 * HD^{0.8} / S \qquad \text{(Eq. 3-2)}$$

where:

- Ldn = Downstream study length (along main channel) in feet for normal depth starting conditions
- HD = Average reach hydraulic depth (1% chance flow area divided by cross section top width) in feet
- S = Average reach slope in feet per mile

Some engineering judgment must be made by the drainage engineer when determining the variables HD, S, and HL. Guidelines are presented below:

a. Average reach hydraulic depth (HD) – If limited existing data is available, an estimate can be made using FEMA maps and Quadrangle Maps. Using the FEMA map, outline on the Quadrangle Map the boundary of the 1 percent chance flow. Select a representative location and plot a cross section using the Quadrangle Map. Plotting several cross sections may improve the estimate. The area (A), top width (TW), and thus the hydraulic depth (A / TW) for these cross sections are now determined. Average these hydraulic depths to determine an average reach hydraulic depth. Survey data or other existing geometric data that is more accurate than the Quadrangle Maps should be used if available.

b. Average reach slope (S) - Using the Quadrangle Maps, determine and average the slope of the main channel, left overbank, and right overbank.

c. Head loss (HL) - This term is also known as the "backwater”. Backwater is defined as the difference in the water surface elevation between the constricted (bridge) flow condition and the unconstricted (no bridge) flow condition at a point of interest upstream of the structure crossing. The drainage engineer must make an educated guess at the anticipated head loss. For a new bridge, the allowable head loss would be a reasonable estimate. In most cases, a maximum head loss of one foot would be expected for Florida.

3.4.1.3 Lateral Extents

The model should extend laterally on both sides of the floodplain to an elevation that is above the highest water surface elevation that will be modeled. Often this water surface elevation will not be known until the model is complete. But data must be collected in order to complete the model. Therefore, the water surface elevation and lateral extent must be estimated for the data gathering effort. The elevation or the lateral extent can be estimated from FEMA maps and other historical studies of the site. In some cases, it may be appropriate to set up a preliminary model based on limited data to estimate the water surface elevations. Whichever method is used to estimate the lateral extent of the model, consider making a conservative estimate to avoid additional data gathering at a later time, especially survey data.

3.4.2 Roughness Coefficient Selection

There are a number of references which can be used to select Manning's Roughness Coefficient within the main channel and overbank areas of riverine waterways. Two recommended references are:

1. "Guide for Selecting Manning’s Roughness Coefficients for Natural Channels and Flood Plains", Report Number FHWA-TS-84-204.
2. "Guide for Selecting Manning’s Roughness Coefficients for Natural Channels and Flood Plains", USGS Water-Supply Paper 2339 which can be accessed at the following link:

   http://www.fhwa.dot.gov/bridge/wsp2339.pdf

Roughness values from previous models or studies can be useful. However, these roughness values should be verified because conditions may have changed.

Roughness values can be varied within reasonable limits representative of the physical conditions of the site to calibrate the hydraulic model.

3.4.3 Model Geometry

Model selection was discussed in Section 3.3. This section discusses the creation of one- and two- dimensional models.

3.4.3.1 One-Dimensional Models

One-dimensional models use cross sections to define the geometry of the channel and floodplain. There are several good references which the drainage engineer can use as guidelines to locate and subdivide the cross sections. One good source is Computation of Water-Surface Profiles in Open Channels, by Jacob Davidian: USGS—Techniques of Water-Resources Investigations Reports Book 3, Chapter A15, 1984. This publication can be downloaded from:

http://pubs.usgs.gov/twri/

Some of the guidelines presented below are from this reference.

a. Cross sections should be taken where there is an appreciable change in slope.
b. Cross sections should be taken where there is an appreciable change in cross sectional area (i.e., minimum and maximum flow areas).
c. Cross sections should be spaced around abrupt changes in roughness to properly average the friction loss between the sections. One method is to evenly space cross sections on either side of the abrupt change. Refer to the spacing between XSEC1 and XSEC2 and between XSEC3 and XSEC4 in Figure 3-4 as an example. Another method is to locate a section at the abrupt change. Include the cross section twice, separated by a short flow length (maybe 0.1 foot), and using the two different roughness values as appropriate.

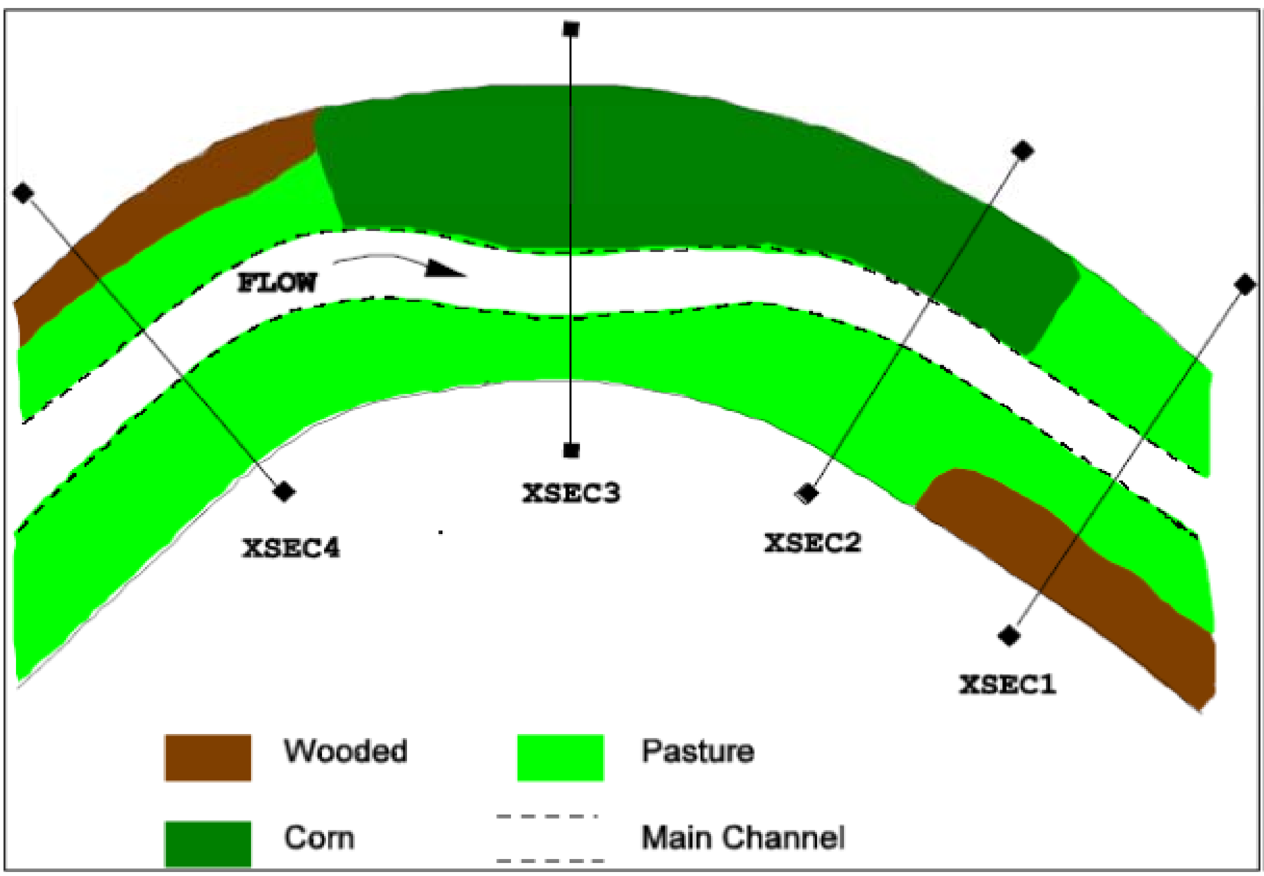


Figure 3-4 Example Cross Section Spacing

d. Cross sections should be taken normal to the flood flow lines. In some cases,”dog legging” cross sections may be necessary. Figure 3-5 illustrates this procedure.

e. Cross Sections should be placed at closer intervals in reaches where the conveyance changes greatly as a result of changes in width, depth, or roughness. The relation between upstream conveyance, K1, and the downstream conveyance, K2, should satisfy the criterion: 0.7<(K1/K2)<1.4.

f. Avoid areas with dead flow, eddies, or flow reversals.

g. Cross section ends must be extended higher than the expected water surface elevation of the largest flood that is to be considered in the sub-reach.

h. Cross sections should be placed between sections that change radically in shape, even if the two areas and the two conveyances are nearly the same.

i. Cross sections should be placed at shorter intervals in reaches where the lateral distribution of conveyance in a cross section changes radically from one end of the reach to the other, even though the total areas, total conveyance, and cross sectional shape do not change drastically. Increasing the number of subdivisions will generally increase the value of alpha, and therefore increase the velocity head. Spacing the cross sections closer together will help prevent drastic changes in the velocity head.

j. Cross sections should be located at or near control sections.

k. Cross sections should be located at tributaries that contribute significantly to the main stem. The cross sections should be placed such that the tributary enters the main stem in the middle of the sub-reach.

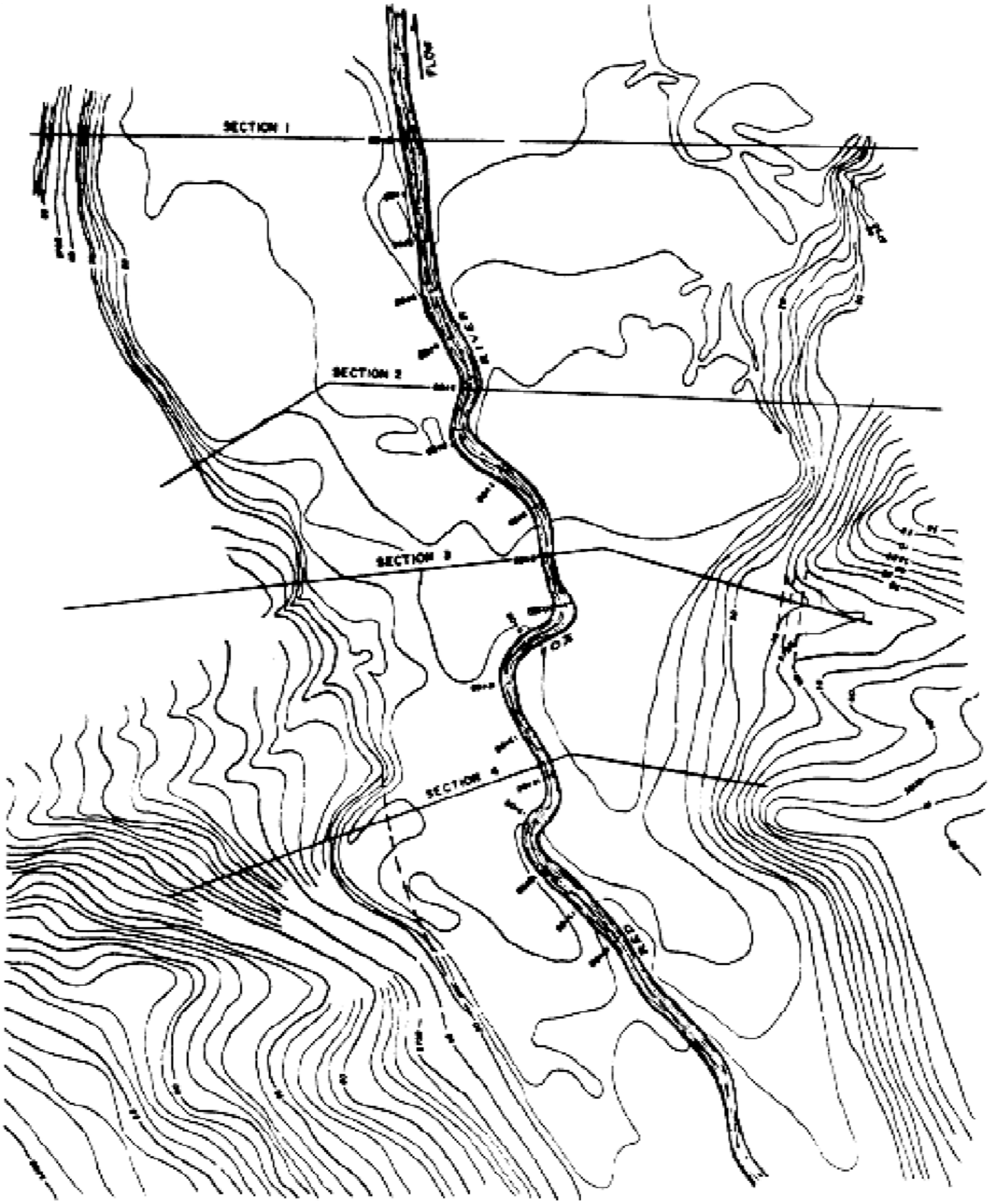


Figure 3-5 “Dog Legging” Cross Section

Subdivisions of cross sections should be done primarily for major breaks in cross-sectional geometry. Major changes in the roughness coefficient may also call for additional subdivisions.

The importance of proper subdivision, as well as the effects of improper subdivision, is illustrated in Examples 3-1 and 3-2 found in Appendix E.

Figures 3-6 and 3-7 show guidelines on when to subdivide.

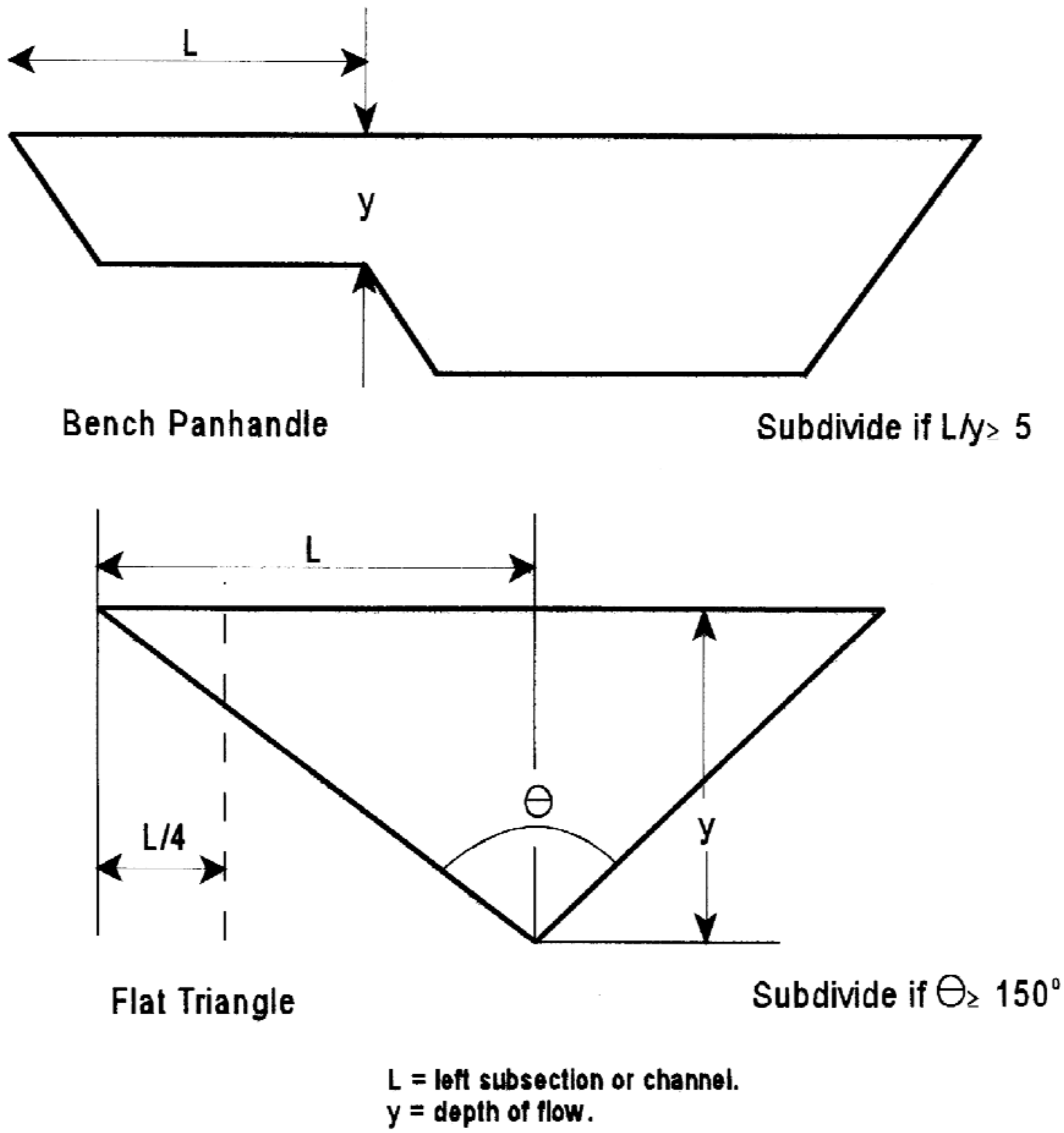


Figure 3-6 Subdivision criteria of Tice (written communication, 1973)

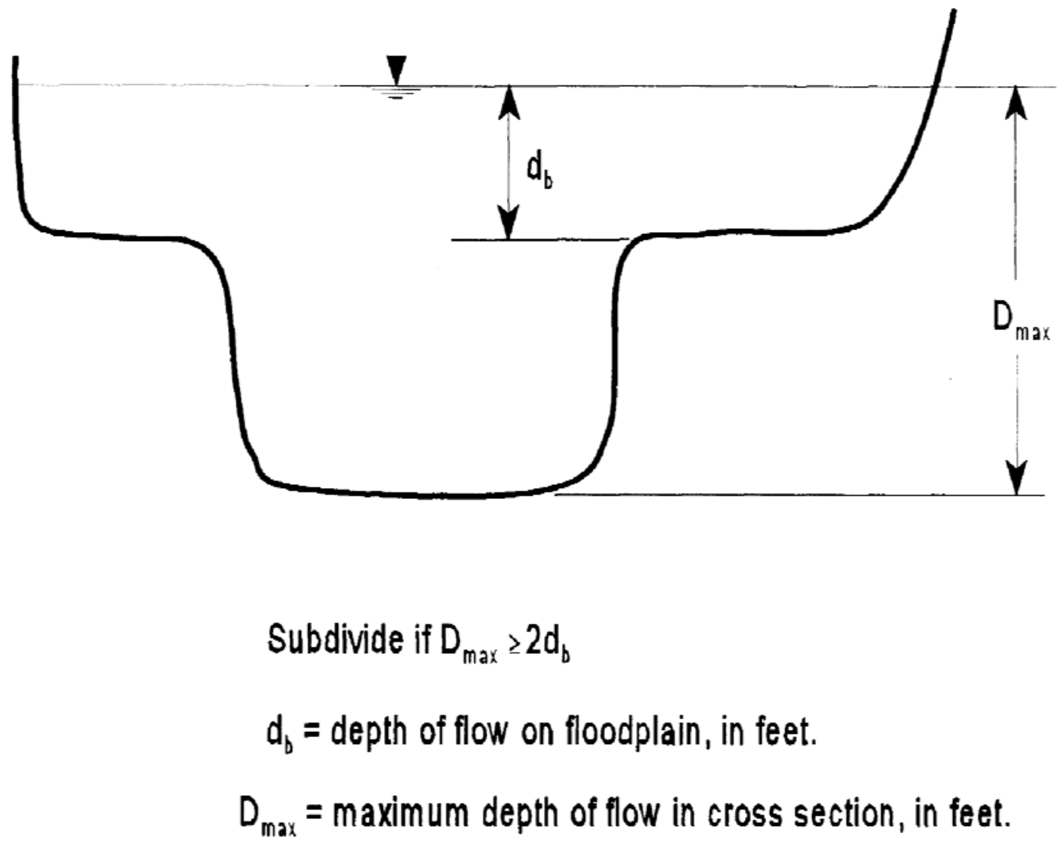


Figure 3-7 Subdivision Criteria of Tice (written communication, 1973)

The energy equation (Equation 2-2 in the FDOT Open Channel Handbook) includes a term for the kinetic energy or velocity head, $V^2/2g$. The average velocity, V, for the entire cross section is used in the equation. In reality the velocity is not a constant value. It is highest in the middle of the channel near the water surface and lowest at the edges of the channel near the channel bottom. Using the average velocity in the equation means that the sum of the differing velocities in the cross section is being squared, $(v_1 + v_2 + \ldots + v_n)^2$. However, to correctly determine the kinetic energy the differing velocities should first be squared and then summed, $v_1^2 + v_2^2 + \ldots + v_n^2$. Since the sum of the squares is greater than the square of the sum, the kinetic energy correction factor is needed. This factor is usually represented by the Greek letter alpha in the energy equation, and is therefore referred to as alpha for short.

Alpha values are calculated and reported for each cross section in both HEC-RAS and WSPRO. However, neither program provides warnings when alpha values are out of range. Incorrect alpha values can cause significant errors. The hydraulics engineer should check the alpha values to be sure they are appropriate.

Alpha values should typically stay in the ranges shown in Appendix A, Bridge Hydraulics Terminology. In general, the more subdivisions in a cross section, the larger alpha will become. Alpha values greater than 3 should be checked. If adjacent cross sections have comparable values, or if the changes are not sudden between cross sections, such values can be accepted. But if the change is sudden, some attempt should be made to obtain uniformity. The drainage engineer may want to consider the following:

a. Resubdivide the cross section(s).
b. Place additional cross sections to provide a smoother transition of the alpha values from one cross section to the next. Note that if the bridge routine in WSPRO is used, additional cross sections cannot be placed between the EXIT and APPROACH sections.

Additional guidance is provided in the Techniques of Water-Resource Investigations (TWRI) Reports.

3.4.3.2 Two-Dimensional Models

Recommendations for developing model geometry for two-dimensional models will depend upon the model employed. Two-dimensional models employ either finite element or finite difference computation schemes. Finite difference models represent the model domain with a regular grid of ground elevations. Figure 3-8 displays examples of the different types of grids employed in finite difference modeling. Finite element methods represent the model domain with a network of triangular and quadrilateral elements that can vary widely in both size and orientation. Figure 3-9 and Figure 3-10 display examples of finite difference and finite element model meshes.

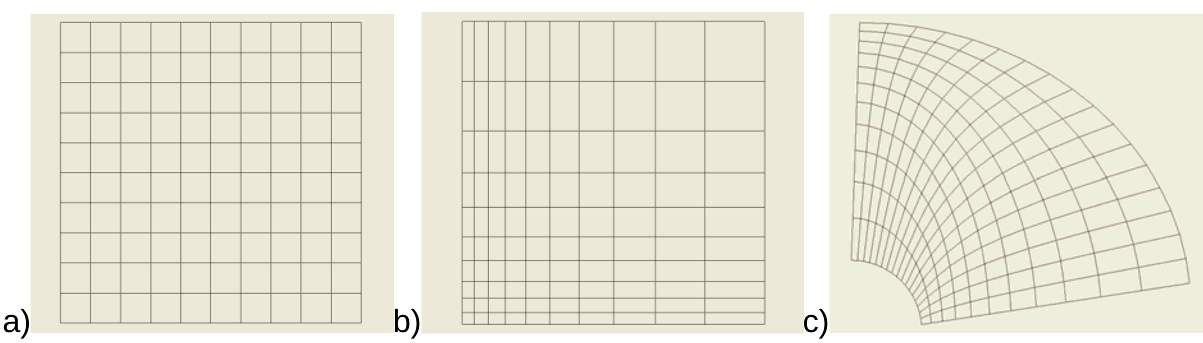


Figure 3-8 Example of (a) Cartesian, (b) Rectilinear, and (c) Curvilinear Grids

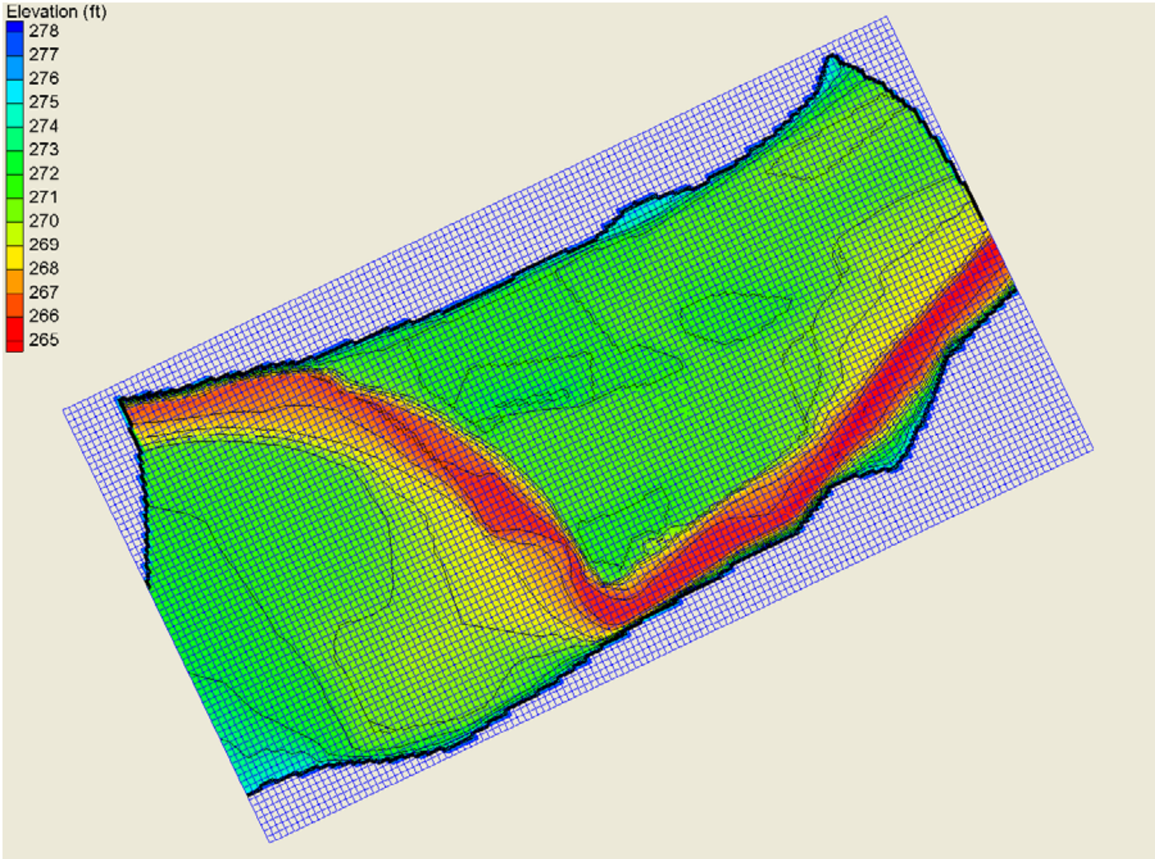


Figure 3-9 Example of a Finite Difference Model Mesh

After defining the model domain, the next step in model geometry development is specification of the element locations, sizes, and orientation. In other words, one must specify the resolution of the model. Finite element models will typically incorporate increased resolution at the project location, along bathymetric features that influence flow through the waterway (shoals, point bars, etc.), and around physical structures in the flow field (causeways, embankments, weirs, etc.) and less resolution with increased distance from the location of interest. Additionally, higher resolution is often incorporated in areas of rapidly changing bathymetry or topography. Examples include at channel banks, head cuts, drop structures, seawalls, and bridge abutments. This varying resolution allows for optimization of computation speed. An example of varying resolution is illustrated in Figure 3-10 with the increased resolution at the inlet and along the navigation channel and decreased resolution in the deeper areas offshore. Mesh generation typically takes place via a Graphical User Interface (GUI). One example is SMS (Surface water Modeling System) available through Aquaveo which provides a number of mesh generation and editing tools as well as pre- and post-processors for a wide variety of hydraulic and wave models. Model resolution is oftentimes one of the model parameters that is modified to achieve both model stability and model calibration.

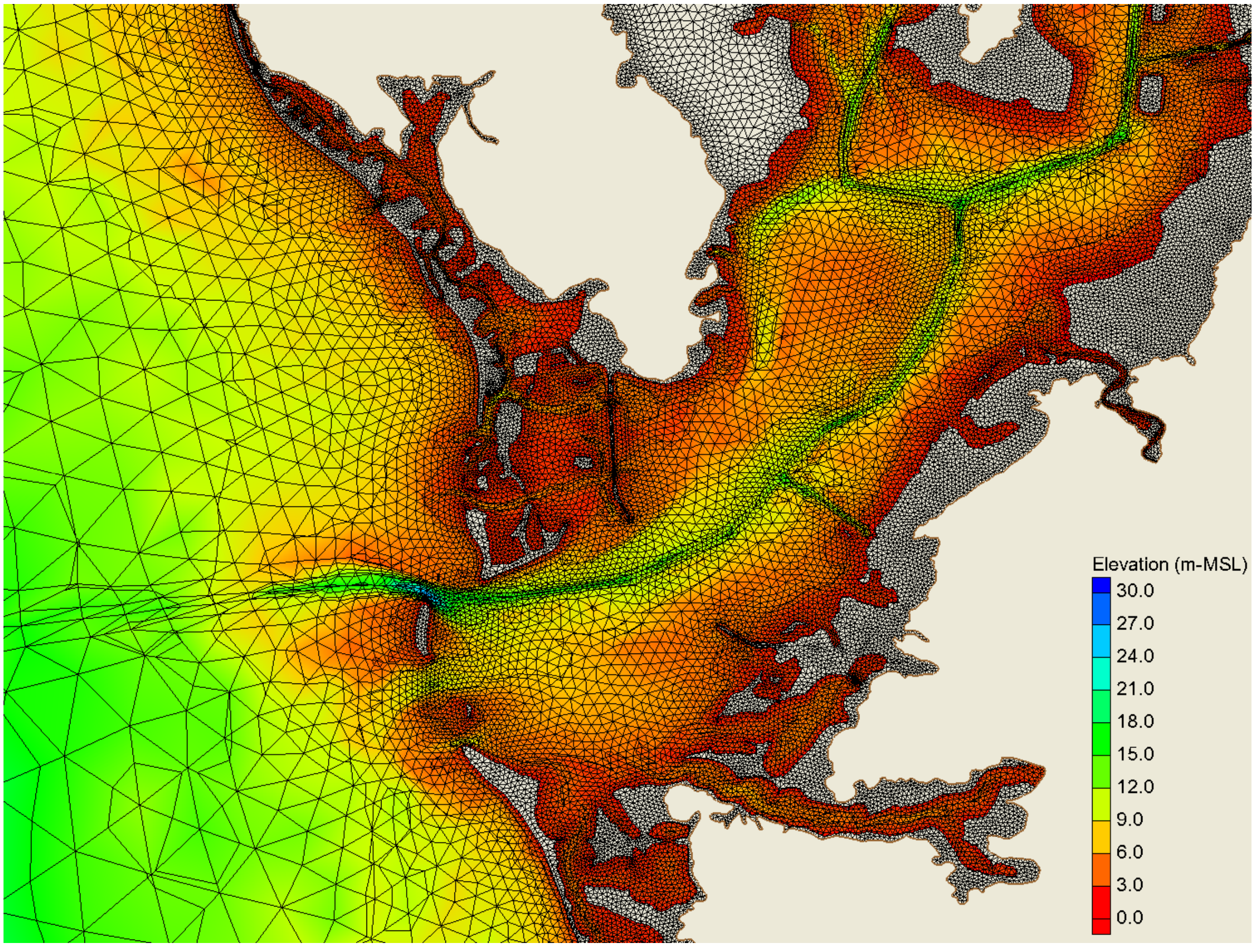


Figure 3-10 Example of a Finite Element Model Mesh

Resolution specification for finite difference models is more challenging than with finite element models. For models that can employ curvilinear or rectilinear grids, resolution can be increased in a few select locations. By nature of the grids, however, this resolution propagates in both ordinal directions from the area of interest through the remainder of the grid. For Cartesian grids, the resolution of a grid is uniform throughout the domain. Thus, the resolution at the bridge location will dictate the resolution for the remaining domain. For large domains requiring fine resolution at the bridge location, a common technique is to employ a nested grid scheme.

After specifying the model resolution, the final step in preparing the model geometry involves specifying the elevations at the model element nodes. Again, this is typically performed with automated mesh generation programs that interpolate a survey data set onto the prepared grid or mesh. This step can sometimes lead to interpolation errors depending upon the relative resolution of the survey data and the model grid/mesh as well as the quality of the TIN (triangular irregular network) representing the survey data. Careful examination of how well the grid/mesh represents the elevations of the model domain is an important part of the model calibration process.

3.4.4 Boundary Conditions

3.4.4.1 Upstream Flow

The flow at the upstream boundary must be given for a riverine analysis. For a steady-state analysis, the peak discharge for each frequency will be specified at the upstream boundary. For an unsteady flow analysis, a flow hydrograph will be specified at the upstream boundary.

3.4.4.2 Downstream Stage

The stage at the downstream cross section must be specified. Known water surface elevations are the first choice. These can be lake levels, sea levels, or control sections such as a gage, studies (e.g., FEMA) or critical depth sections.

Normal depth can be used in many cases when the stream channel is nearly uniform for a fairly long reach. HEC-RAS or WSPRO can be used to compute the normal depth by providing an energy slope equal to the channel slope. This method is also known as "slope conveyance". The channel slope can be determined using a USGS Quadrangle Map. The slope should be determined below the last downstream cross section where contour lines cross the stream channel. Other estimates of energy slope can be used; however, the resulting water surface elevation would not be "normal depth".

When there is no gage information available and when normal depth flow (slope conveyance) cannot be assumed at the bridge site, "convergence" should be used.

3.4.4.3 Convergence

Water surface profiles will converge to a single profile if given enough distance to converge. The distance depends on the channel and overbank properties and the slope of the river. The distance can be estimated using Equation 3-2.

Convergence can be determined as follows:

a. Trial and error calculations are made assuming a range of water surface elevations. This assumed range of water surface elevations should bracket the drainage engineer’s best guess of the water surface elevation at the furthest downstream cross section. Typically this is done using an estimate of the friction slope and calculating normal depth.

b. Using the estimate of water surface elevation at the furthest downstream cross section, develop four water surface profiles for the design discharge based on a range of potential water surface elevations. Two of the bracketed elevations should represent the range that the water surface should be between, and the other two should represent the range the water surface is unlikely to be outside. Refer to Figure 3-11.

c. The computed profiles will converge toward the true profile. The profiles should converge within an acceptable tolerance by the <u>first section of interest in the</u>

reach (see Figure 3-11). If the profiles do not adequately converge, then additional geometric data should be obtained downstream.

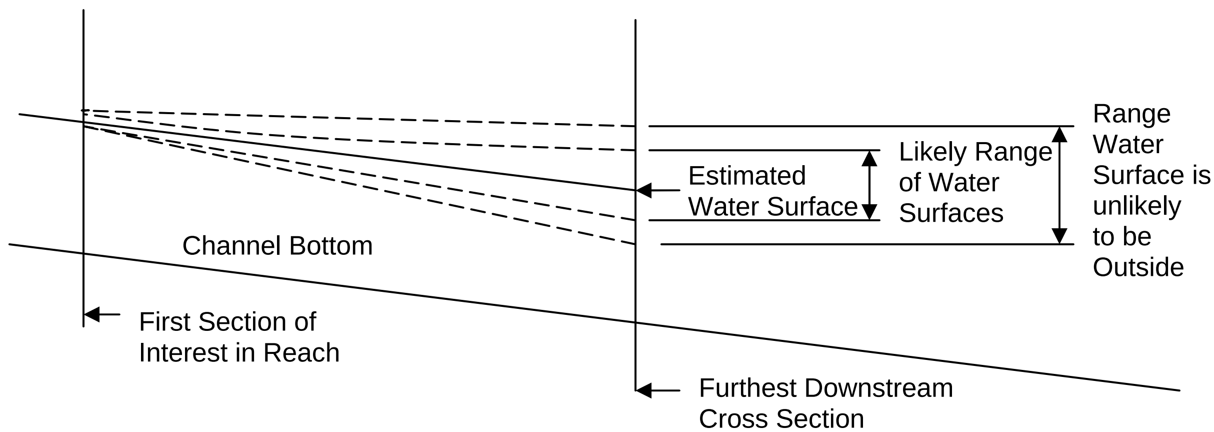


Figure 3-11 Convergence Profiles

3.4.5 Bridge Model

3.4.5.1 Roughness

The roughness around and under the bridge can be significantly different than the roughness upstream and downstream due to rubble riprap protection and clearing of trees and underbrush. The main channel roughness is often the same through the bridge from upstream to downstream. A significant extent of rubble riprap protecting the piers or channel banks would be the most common reason that the roughness will change.

Many Florida floodplains are heavily vegetated. Many riverine bridges span a significant length across the floodplain. The area beneath the bridge is often cleared of the trees and underbrush, and is maintained that way. This will reduce the roughness. However, rubble protection of the abutment will increase the roughness. The guidelines for subdivision (refer to Section 3.4.3.1) would usually recommend against subdividing at the toe of the abutment, so a weighted roughness should be determined.

Abrupt changes in roughness should be modeled appropriately to properly account for the friction loss between the cross sections. The Standard Step Method uses an average of the conveyance for each cross section to calculate the friction loss between the cross sections, which essentially averages the roughness values of the two sections. A desirable method of modeling abrupt roughness changes is to include two cross sections closely spaced at the change location. However, some of the bridge routines of the various models will not allow the extra cross section.

Nodes and elements in two-dimensional models can be placed such that abrupt roughness changes do not bisect elements.

3.4.5.2 Bridge Routine

Refer to HEC-RAS documention for cross section location information. Note that it should not be followed if the WSPRO bridge routine is used when modeling in HEC-RAS. Use the following recommendations for WSPRO.

The bridge routine in WSPRO uses the Standard Step Backwater Method, only with more complexity. The bridge hydraulics is based on the reach from the EXIT to the APPROACH section as defined in the WSPRO Manual. Although the manual specifies "one bridge length", this does not mean the EXIT section must be exactly one bridge length downstream from the Full-Valley section or that the APPROACH section must be exactly one bridge length (plus roadway width) upstream from the Full-Valley section. The locations of these sections can vary as follows.

EXIT Section:
The Exit Section can be located no less than, but as much as 10 percent greater than one bridge length from the Full-Valley section. See Figure 3-12.

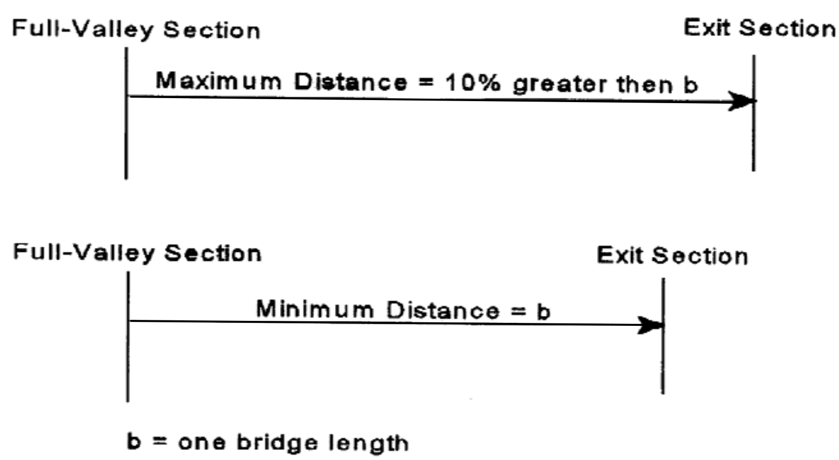


Figure 3-12 Location of Exit Section

APPROACH Section:
The approach section can be located as much as 15 percent less than or greater than one bridge length plus the roadway width from the upstream face of the bridge. See Figure 3-13.

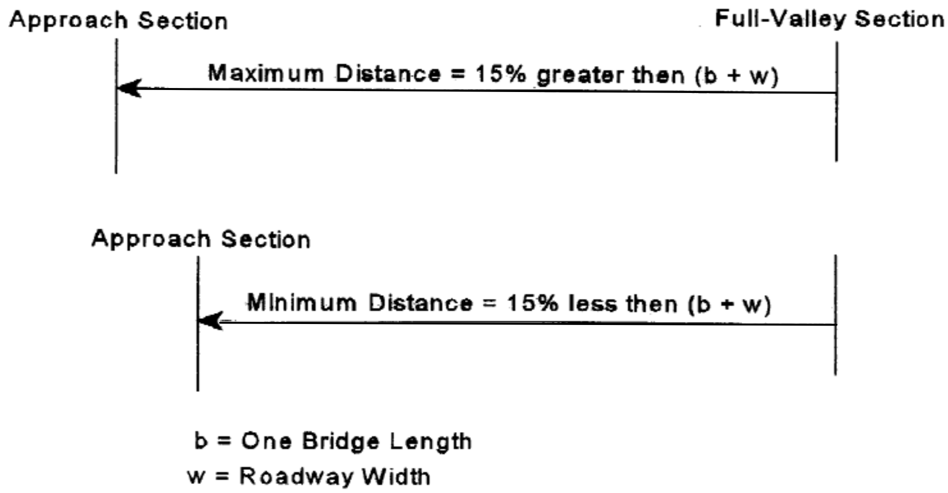


Figure 3-13 Location of Approach Section

If for some reason it is impossible to follow the cross section requirements, it may be necessary for the drainage engineer to analyze the site without using the bridge routine.

3.4.5.3 Piers

Single row pile bent bridges can often be modeled without modeling the piles and the hydraulic results will be the same as if they were included. However, regulatory agencies may want to see the piles included in the model. As the blockage becomes greater for more complex piers, the hydraulic results will change.

3.5 Simulations

3.5.1 Calibration

Calibration involves changing the value of coefficients until the model results match observed field conditions for one or more known events. Once the model has been calibrated to known events, then an unknown event such as the design frequency event can be modeled with more confidence.

Observed field data for a flood event can include:

- Water surface elevations
- Discharge measurements
- Velocity measurements

Data from multiple flood events should be obtained if available. The closer the magnitudes of the observed events are to the magnitude of the design events, the more certain the results will be.

Generally, gage data will be the most reliable source of information. Most gages used in riverine situations measure the water surface elevation. Figure 3-14 shows a simple staff gage which must be observed and recorded manually. More complex gaging stations will record stages automatically and either store the records for later download, or transmit the data using telemetry.

Figure 3-14 Staff Gage on the Suwannee River

Discharges are determined indirectly from the water surface elevations. Traditionally, a velocity meter is used to take measurements at intervals across the stream and the discharge is determined as shown in Figure 3-15. When the discharge has been determined at enough different water surface elevations, a stage verses discharge relationship can be established for the gage.

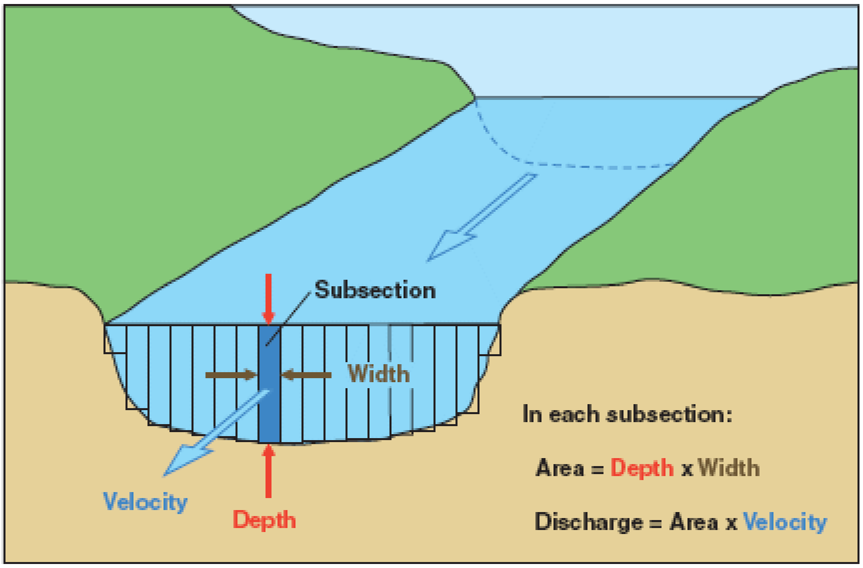


Figure 3-15 Discharge Determination with a Velocity Meter (from USGS Streamgaging Fact Sheet 2005-3131, March 2007)

More recently, discharges have been measured on some larger rivers with an Acoustic Doppler Current Profiler mounted on a boat (see Figure 3-16).

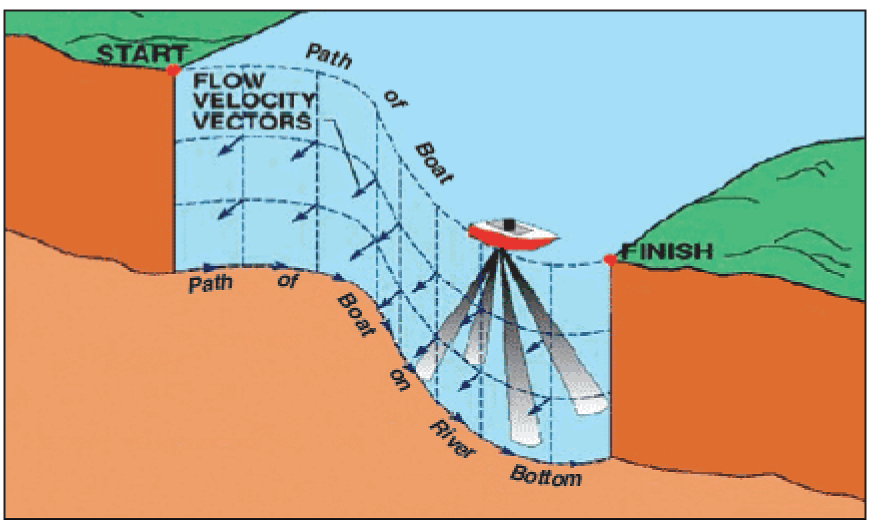


Figure 3-16 Discharge Determination with an Acoustic Doppler Current Profiler (from USGS Streamgaging Fact Sheet 2005-3131, March 2007)

The primary benefit of a gage will be to establish the discharge for an observed flood. If a gage is located within the model reach, then the gage can also supply stage and velocity information at one point in the model.

If gage data is unavailable, consider sending survey to measure:

- High water marks associated with known floods (Figure 3-17)
- Local resident or official high water permanent markers/signs (Figure 3-18)
- Ordinary high water marks (stain lines on existing bridge pilings or vegetative indicators)

Occasionally the Department and agencies such as USGS, FEMA, DEM or the water management districts may have surveyed or collected high water marks following a flood. Contacting them is an avenue to pursue.

Figure 3-17 Examples of High Water Marks after a Flood

If a gage is not available to determine the discharge of the known event, then estimating the discharge associated with the various high water marks will be difficult or impossible. Obtaining rain gage information for the flood and estimating the runoff from the rainfall is an option, assuming data from a suitable rain gage is available. Otherwise, the high water marks can only be compared to the computed design frequency profiles from the model to check the magnitudes for reasonableness.

Figure 3-18 Local Resident indicating Flood Level on the Caloosahatchee River near LaBelle in 1913

After available gage data and/or high water mark elevations are obtained, the next step is to develop the hydraulic model for the existing site conditions. In some situations this might entail multiple existing condition models if the site conditions have changed since some of the calibration floods. The model should be developed using standard guidance for the coefficients used in the model. The initial model results are then compared to the high water marks, and the coefficients are adjusted. The common coefficients that can be adjusted are:

- Manning's Roughness Coefficient
- Bridge loss coefficients (depending on the bridge routine used)
- Expansion and Contraction Coefficients

Manning's roughness is the basic adjustment tool for unobstructed reaches. Considerable uncertainty exists when estimating roughness values. Estimates by experienced hydraulics engineers often vary by plus or minus 20% (from EM 1110-2-1416). It is recommended to hold the channel roughness constant, and vary the overbank roughness. Also remember that Manning's roughness varies with depth which can affect calibration:

- As the depth over the roughness elements increase, n decreases.

- If the flow encounters a new roughness element as the flow depth increases, n will increase. For example, if tree branches are higher than a certain depth in the floodplain, the roughness will increase when the flow reaches the tree branches.

The calibration coefficients should not be adjusted outside of their normal ranges. If the calibration attempts are not acceptable reexamine the model. Common model parameters to review if calibration is a problem include:

- Ineffective flow areas
- Starting conditions downstream
- Cross section locations
- Cross section subdivisions
- Verify the accuracy of survey data or other geometric data
- Double check datums of geometric data
- Flow lengths
- Check warning messages

Note that calibration problems can be caused by different issues. The modeler will need to use their judgment in the calibration process. There is no universally accepted procedure or criteria for calibration.

Calibrating unsteady flow models is more difficult than calibrating steady flow models. Adjust to steady flow conditions first, if possible. Unsteady flow models need to be calibrated over a wider range of flows than steady-state models. Storage in the system is an important parameter for unsteady flow, and can essentially be used as an adjustment parameter. For more detail on techniques for unsteady flow calibration, refer to USACE Manual EM 1110-2-1416, River Hydraulics.

Two-dimensional models have eddy viscosity, or turbulent loss coefficient that becomes another calibration parameter. This term in essence replaces expansion and contraction losses in a one-dimensional model. However, there is not an established correlation between the two losses. The best way to calibrate eddy viscosity is with measured velocities. Remember that the two-dimensional velocity is depth-averaged, so the measured velocity must be converted to a depth-averaged velocity for comparison. Set the value high first, and then lower it until the desired velocity distribution is obtained. The general order of calibration for two-dimensional models would be to calibrate roughness values to observed water surface elevations, and then adjust eddy viscosity to observed velocities.

When using both velocities and stages for calibration, check for internal consistency of the observed data. The velocity times the area for the stage should be approximately equal to the discharge.

3.5.2 Existing Conditions

The existing conditions should be modeled to compare with the results from the proposed structure and to calibrate the model to observed flood data. If the existing condition is a bridge at the site, then consider also modeling the natural conditions at the site prior to construction of the existing bridge.

3.5.3 Design Considerations

Review Project Development and Environment (PD&E) Documents for commitments made during the NEPA process. During PD&E, a Location Hydraulics Study should look at alternate locations for the plan view of the roadway crossing of the stream or river. Adverse hydraulic conditions should be identified in the Location Hydraulics Study for consideration when planning the roadway crossing. The final location will not depend solely on hydraulic aspects, but they should be considered during the initial planning of the roadway. The location and alignment of the highway can either magnify or eliminate hydraulic problems at the crossing. By the time the Bridge Hydraulics Report is prepared, the location and alignment of the road should be set. However, minor changes to the alignment may still be possible.

The length of the bridge and the location of the abutments are usually investigated and selected in the Bridge Hydraulics Report. Traditionally, at least three lengths are analyzed. One is the minimum hydraulic structure, the bridge that creates no more than one foot of backwater and does not violate other allowable water surface conditions. Another bridge length examined is the bridge that spans all wetlands. Other potential bridge lengths to investigate include:

- The length of the existing bridge.
- For dual bridges, the length of the existing dual bridge left in place.
- Breaks in fill height if bridging is less expensive than roadway fill.
- Minimum bridge length based on setbacks from the channel banks.

Other considerations when designing and modeling the proposed conditions are:

- Place the bridge in a crest vertical curve, if possible. Allowing the approach roadways to overtop more frequently than the bridge will provide relief for the bridge, and reduce the possibility of damage to the structure. If a portion of the roadway is damaged, it can usually be repaired more easily than the bridge.
- Try to center the bridge over the main channel of the flow. At a minimum, set the toe of the abutments ten feet back from the top of the channel bank.
- Consider skewing the abutments and intermediate bents to align with the flood flow direction to reduce scour potential.

Chapter 4

Tidal Analysis

Hydraulic and scour analyses of tidal and tidally influenced bridges should be performed by a qualified coastal engineer. Section 2.1 defines the requirements and credentials of coastal engineers qualified to perform tidal analyses for the FDOT.

4.1 Data Requirements

Evaluation and design of tidally influenced bridges requires a preliminary, systematic data collection effort to determine the hydraulic conditions at the structure, calculate the scour, and develop the wave climate at the structure. This information includes details of the bridge geometry, the bed composition and elevations, and historical measurements and studies.

4.1.1 Survey Data

Survey data is required for several aspects of a bridge hydraulics and scour analysis. It not only provides the elevation data to construct hydraulic and wave models, but also provides needed sediment characteristics for scour calculations. The requirements for a tidal analysis are the same as those for riverine analyses with one exception: typically, the size of the modeling domain for tidal studies is substantially larger than those for riverine studies. Since new survey acquisition of the required data over the entire domain is rarely cost-effective, survey data acquired around the bridge should be supplemented with publically available data. Several sources exist for supplemental data including the following examples:

- Bathymetric and topographic data from the National Geophysical Data Center (http://www.ngdc.noaa.gov/mgg/bathymetry/relief.html, Example: Figure 4-1)
- Digital Elevation Models from the FDEP Land Boundary Information System website (http://www.labins.org/mapping_data/dem/dem.cfm)
- Coastal LiDAR data from NOAA's Coastal Services Center (http://coast.noaa.gov/digitalcoast/data/coastallidar)

Caution should be exercised when combining data from several sources. There can be wide ranges in accuracy due to differing measurement techniques and survey dates. Careful attention should also be paid to conversion between different horizontal and vertical coordinate systems. Boundaries between survey data sets should be examined for inconsistencies and corrected.

The accuracy and density of survey data becomes more important near the site of interest. This is especially true of bathymetry for wave modeling when depth limitation is expected to govern wave conditions.

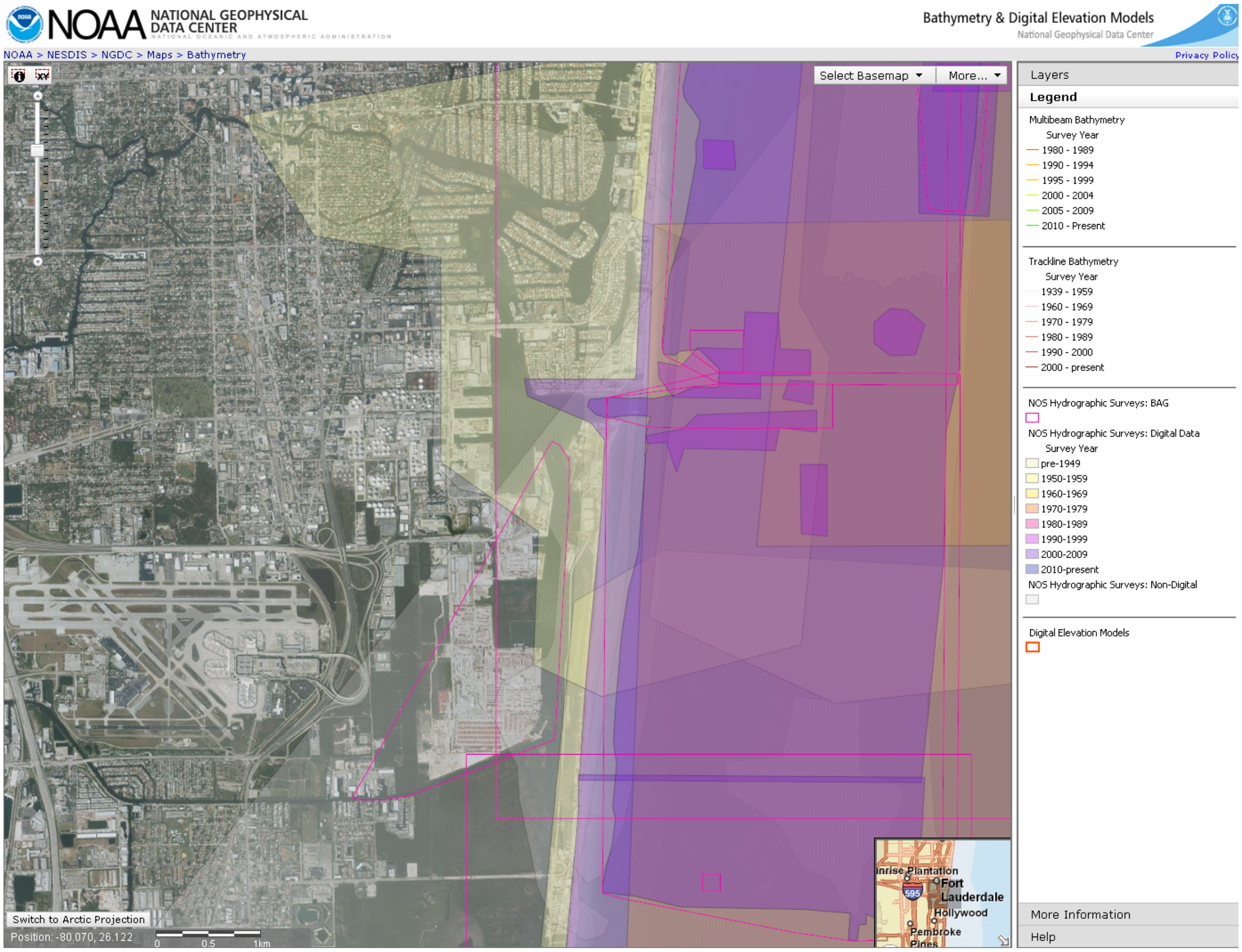


Figure 4-1 NOAA National Geophysical Data Center Website

4.1.2 Geotechnical Data

In order to calculate scour at bridge foundations, geotechnical information is required to establish the bed composition and its resistance to scour. Data requirements for tidal bridges are the same as those for riverine bridges. Refer to section 3.1.2 for discussion of geotechnical data requirements.

4.1.3 Historical Information

Historical information provides data for calibration through gage measurements and historical high water marks, data for calculation of long-term scour processes through historical aerial photography and Bridge Inspection Reports, and characterization of the hurricane vulnerability through the hurricane history.

4.1.3.1 Gage Measurements

Gage measurements provide information both for model calibration as well as for model boundary conditions. Several sources of gage data are publically available. The types of gage measurements typically employed in tidal analyses include:

- Streamflow and river stage gages – For establishing inland boundary conditions and calibration.
- Tide gages – For oceanward boundary conditions and calibration of tidal circulation.
- Wave gages – For calibration of wave models.

Data sources of streamflow and river stage records are the same as those discussed for riverine analyses.

Tide gage data can also be employed for development of model boundary conditions as well as for model calibration. Tide gages record stage at a fixed location in tidally influenced areas. NOAA maintains gages throughout the state. Recent and historic data is available online at http://www.co-ops.nos.noaa.gov/. In Florida, the site provides data at 29 active stations (Figure 4-2) and historic data at 722 locations.

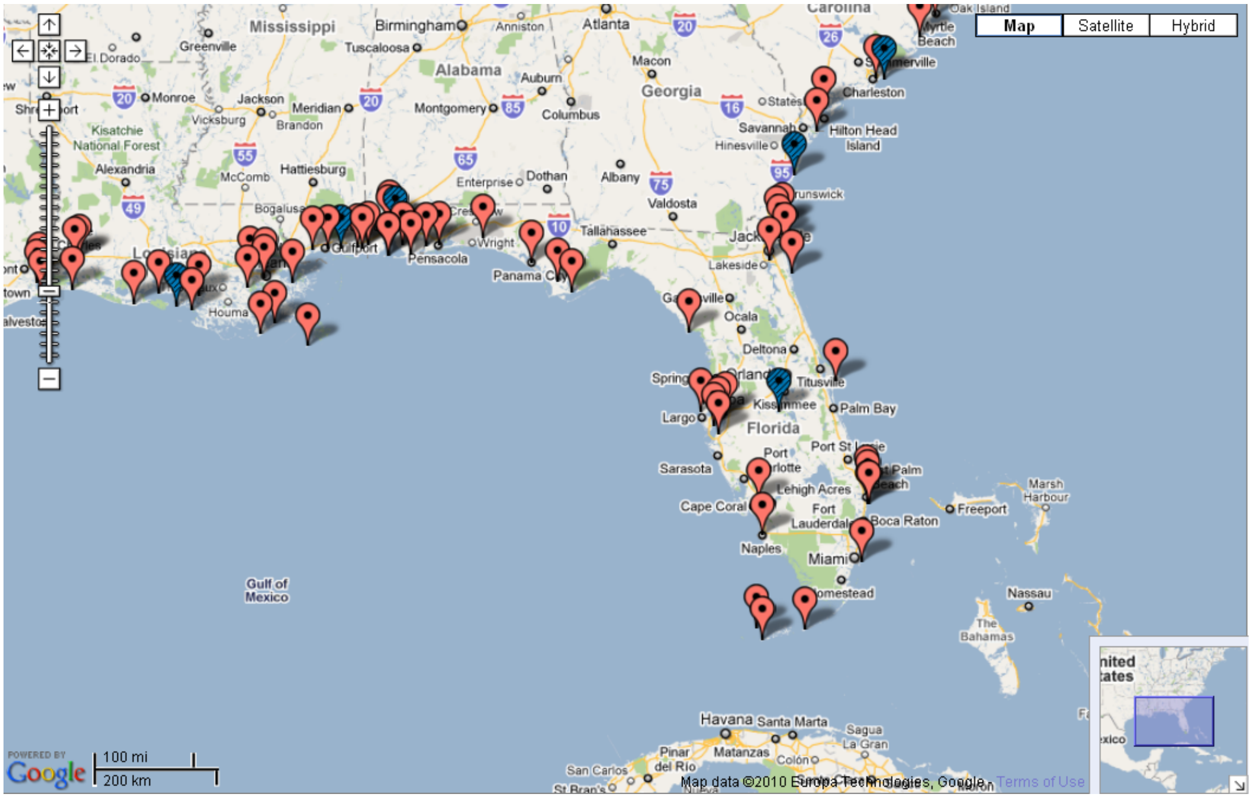


Figure 4-2 Location of Florida's Active Tide Stations Maintained by NOAA (Source: http://www.co-ops.nos.noaa.gov/gmap3/)

Wave gage data, used to calibrate data for wave models, is typically much rarer than either streamflow or stage records. The National Data Buoy Center (NDBC), a part of the National Weather Service (NWS), designs, develops, operates, and maintains a network of data collecting buoys and coastal stations. Several of these stations include measurement of wave parameters including significant wave height, swell height, swell period, wind wave height, wind wave period, swell wave direction, wind wave direction, wave steepness, and average wave period. The NDBC website (http://www.ndbc.noaa.gov/) provides both recent and historical observations at several locations around Florida (Figure 4-3). Figure 4-4 provides an example of this type of data as a time series of significant wave height. Sources of wave gage data for interior waters (such as bays, estuaries, intracoastal waterways, etc.) are much harder to locate. Possible sources may include previous studies and academic institutions.

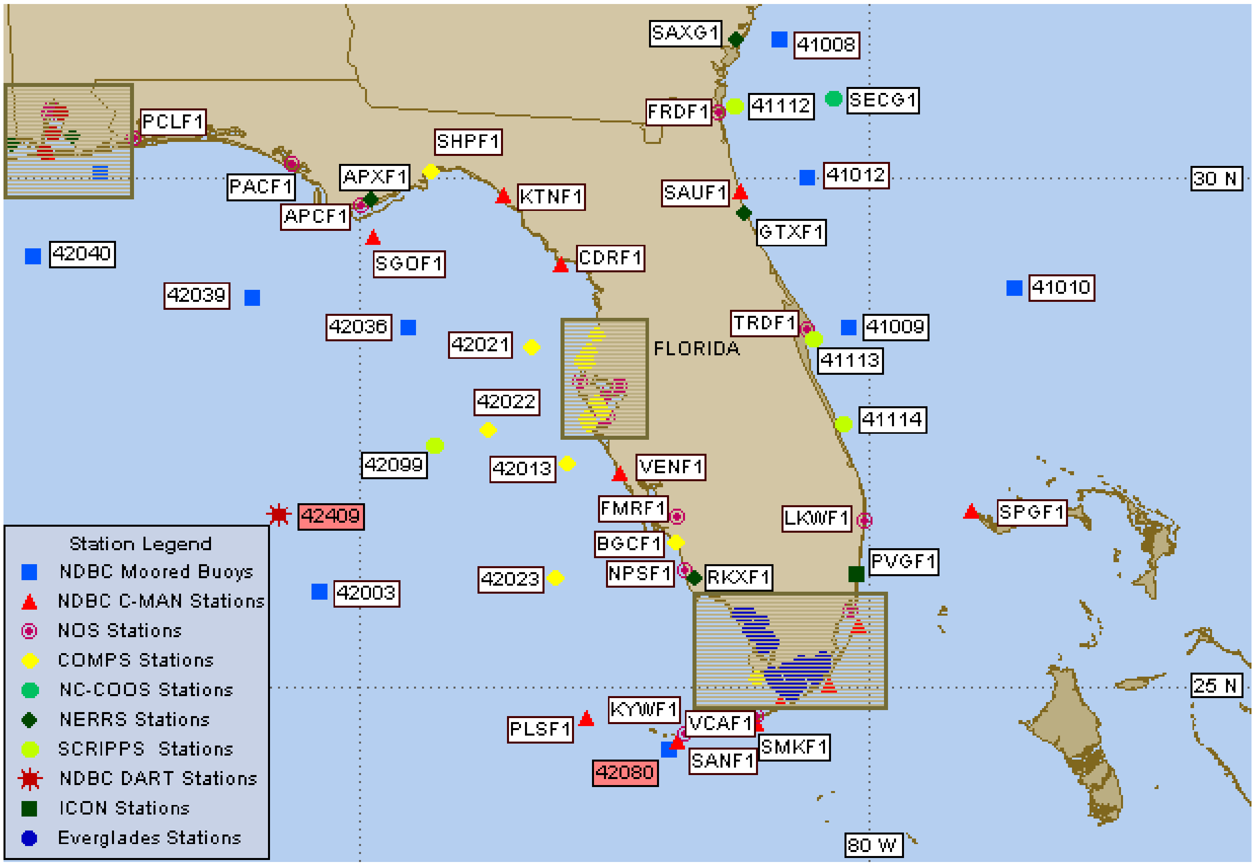
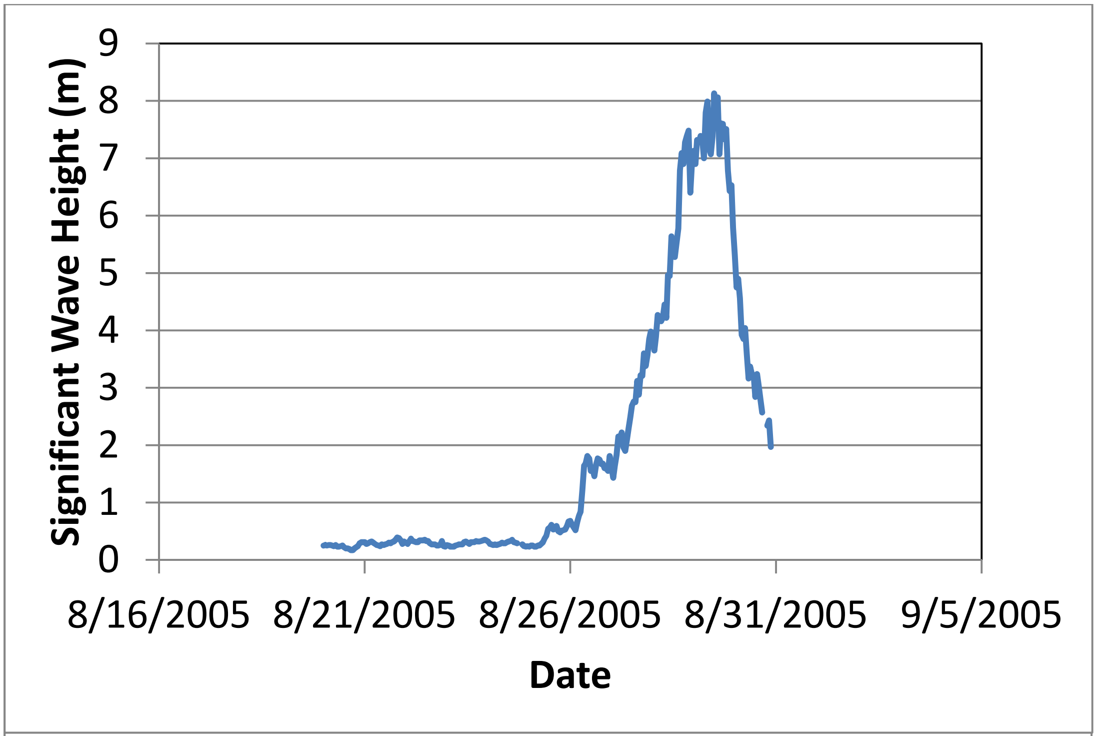


Figure 4-3 Locations of NDBC Stations around Florida (Source: http://www.ndbc.noaa.gov/)



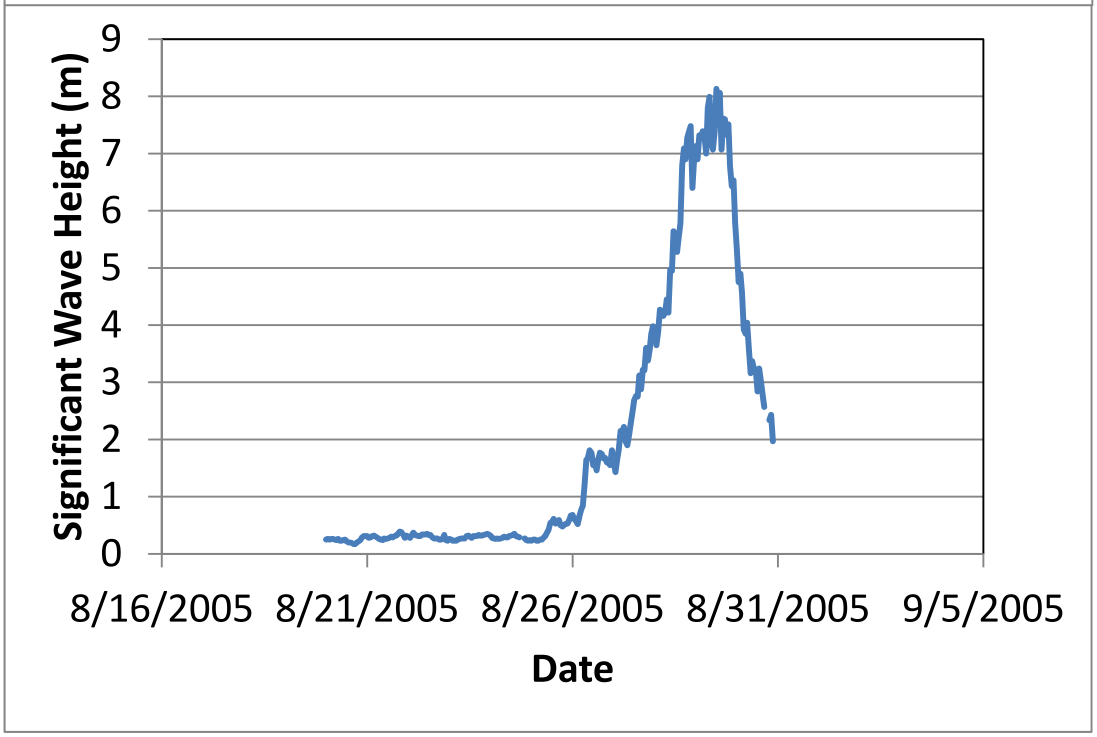


Figure 4-4 Example of Wave Gage Data at NDBC Station 42039 during Passage of Hurricane Katrina

4.1.3.2 Historical High Water Marks

The historical hurricane high water marks provide additional calibration data sets for the storm surge numerical model during specific hurricane events. Post-storm damage assessments are typically performed for or by FEMA. Although the survey accuracy has significantly increased over recent years, caution should be exercised when employing these data. Coastal high water marks are typically designated as one of three basic types:

- Surge - represents the rise in the normal water level
- Wave height - represents the coastal high water mark elevation due to more direct wave action
- Wave runup - represents the height of water rise above the stillwater level due to water rush up from a breaking wave

High water marks are often found near each other and can vary widely in elevation. Surge only high water marks occur only where the structure is at a location sheltered from waves. As waves propagate inland during a surge, the high water conditions on structures and land can vary widely. Coastal wave height flooding is created by the crest of the wave riding on the surge. Thus, differences will occur between high water marks measured on the interior and exterior walls of a structure. Finally, wave runup high water marks include the effects of waves breaking on sloping surfaces. After a wave breaks on a beach or sloping surface, a portion of the remaining energy will propel a bore that will run up the face of the slope. The vertical distance the bore travels above the still water level is termed the wave runup. Wave runup often pushes debris to its maximum limit where it is left as a wrack line (a line of debris illustrating the extent of the wave runup).

4.1.3.3 Hurricane History

The hurricane history of the project location characterizes the hurricane frequency at the project as well as the historical impacts to the site location. Including this information in the Bridge Hydraulics Report elevates the importance of examining hurricane surge and wave impacts, providing an qualitative examination of the frequency of hurricane influences at the bridge site. Additionally, it can provide a tool for comparing the selected calibration hurricane to the overall activity for the area. Information to be included in the BHRshould include the historical hurricane paths, historical storm year and category as well as discussion of significant storms to impact the area. An example of the hurricane paths and listing of the historical hurricanes is displayed in Figure 4-5 and Table 4-1 (from http://csc.noaa.gov/hurricanes/#).

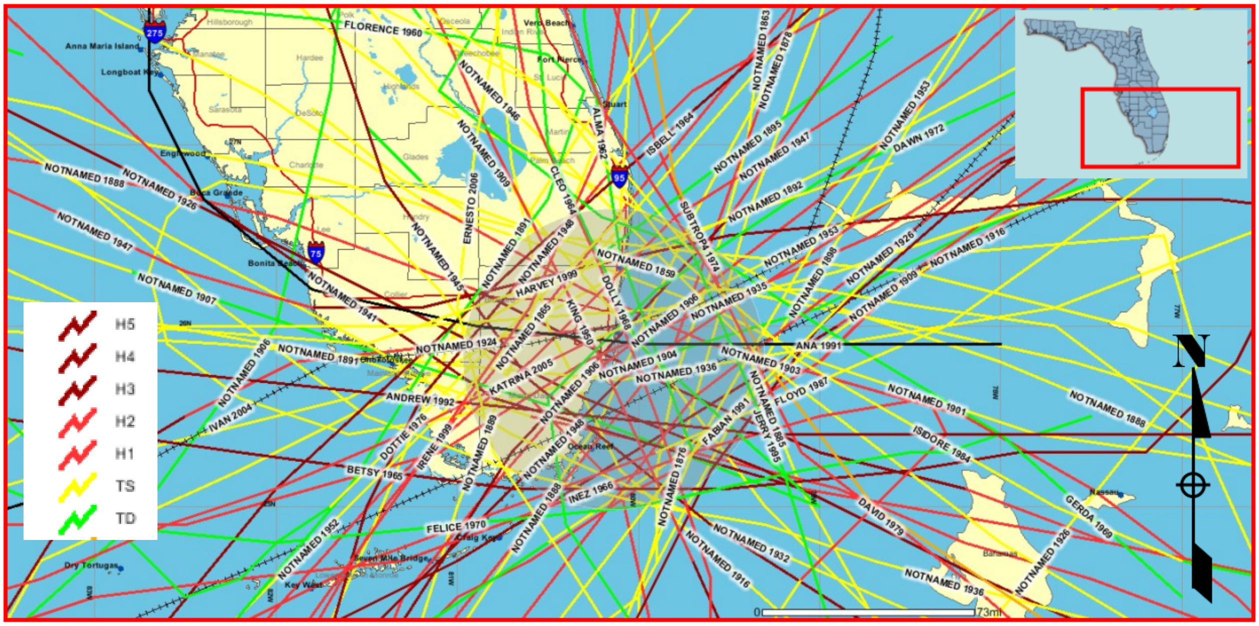


Figure 4-5 Hurricane and Tropical Storm Tracks Passing within 50 Nautical Miles (nmi) of Miami (Source: NHC)

Table 4-1 Hurricanes Passing within 50 nmi of Miami

Year	Month	Day	Storm Name	Wind Speed (kts)	Pressure (mb)	Category
1865	10	23	NOTNAMED	90	0	H2
1870	10	10	NOTNAMED	90	0	H2
1878	10	21	NOTNAMED	70	0	H1
1885	8	24	NOTNAMED	70	0	H1
1888	8	16	NOTNAMED	110	0	H3
1891	8	24	NOTNAMED	75	0	H1
1903	9	11	NOTNAMED	75	976	H1
1904	10	17	NOTNAMED	70	0	H1
1906	10	18	NOTNAMED	105	953	H3
1909	10	11	NOTNAMED	100	957	H3
1924	10	21	NOTNAMED	70	0	H1
1926	9	18	NOTNAMED	120	0	H4
1926	10	21	NOTNAMED	95	0	H2
1935	9	28	NOTNAMED	100	0	H3
1935	11	4	NOTNAMED	65	973	H1
1941	10	6	NOTNAMED	105	0	H3
1945	9	15	NOTNAMED	120	0	H4
1947	9	17	NOTNAMED	135	947	H4
1947	10	12	NOTNAMED	75	0	H1
1948	9	22	NOTNAMED	100	0	H3
1948	10	5	NOTNAMED	110	975	H3
1950	10	18	KING	95	0	H2
1964	8	27	CLEO	90	968	H2
1964	10	14	ISBELL	110	968	H3
1965	9	8	BETSY	110	952	H3
1966	10	4	INEZ	75	984	H1
1979	9	3	DAVID	85	973	H2
1987	10	12	FLOYD	65	993	H1
1992	8	24	ANDREW	130	937	H4
1999	10	16	IRENE	65	986	H1
2005	10	24	WILMA	110	953	H3

4.1.3.4 Historical Aerial Photographs

Historical aerial photographs aid in evaluating the channel stability at a bridge crossing. Comparison of photographs over a number of years can reveal long-term erosion or accretion trends of the shorelines and channel near the bridge crossing. An example of this is provided in Figure 4-6 and Figure 4-7. From the figures, changes in shoreline location occur south of the east abutment as well as to the spit south of the inlet. Calculation of long-term trends will be further discussed in Section 6.1.1. Sources of historical aerial photography are the same as those discussed in the previous chapter in Section 3.1.3.2.

Figure 4-6 Heckscher Drive (SR-A1A) near Ft. George Inlet in 1969

Figure 4-7 Heckscher Drive (SR-A1A) near Ft. George Inlet in 2000

4.1.3.5 Existing Bridge Inspection Reports

Existing Bridge Inspection Reports often provide sources of recent and historical cross section measurements as well as identify areas of hydraulic/scour related damage or repairs. Refer to Section 3.1.3.3 for additional discussion on obtaining and utilizing these reports in hydraulic analyses.

4.1.3.6 Wave Information Studies

Another source of coastal wave hindcast data is the Wave Information Studies (WIS), developed and maintained by the U.S. Army Corps of Engineers (USACE) Coastal and Hydraulic Laboratory. The WIS project produced an online database of hindcast, nearshore wave conditions along the U.S. coasts. The hindcast data provides a source of decades-long wave data that can provide boundary conditions or calibration data for nearshore wave modeling. The data includes hourly wave parameters of significant wave height, peak period, mean period, mean wave direction, and wind speed and direction (Figure 4-8). The database includes both nearshore and offshore gages along both Florida's Atlantic Ocean and Gulf of Mexico shorelines. The data is available via the following link: http://wis.usace.army.mil/

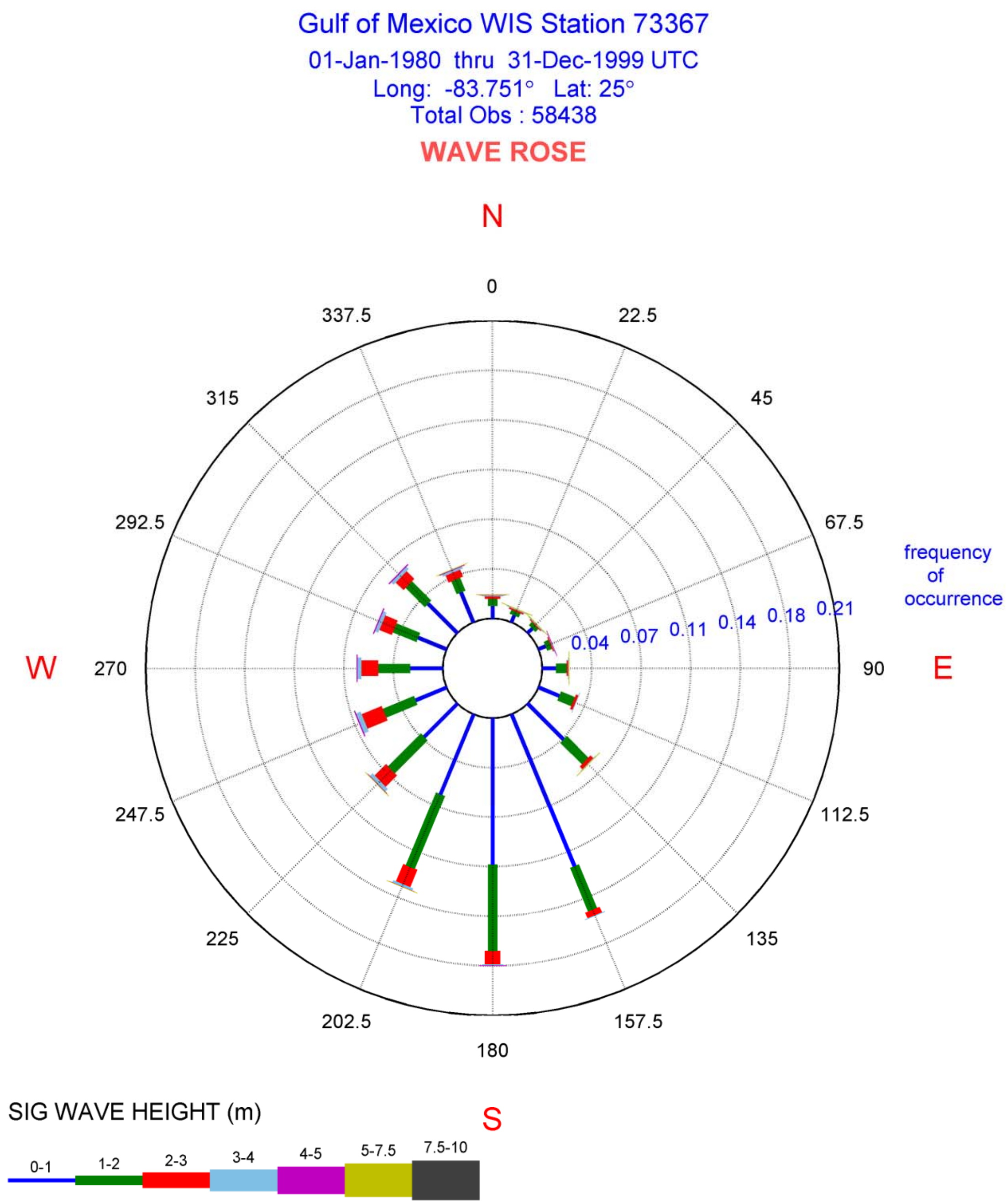


Figure 4-8 Example of Available WIS Data from: **http://chl.erdc.usace.army.mil/datacollection**

4.1.3.7 Previous Studies

Previously performed studies of a waterway can provide additional sources of data for the hydraulic/coastal engineer. Refer to Section 3.1.3.4 for sources and discussion of previous studies.

4.1.4 FEMA Maps

FEMA Flood Insurance Rate Maps (FIRM) are the official map of a community that displays the floodplains — specifically special hazard areas and risk premium zones — as delineated by the Federal Emergency Management Agency (FEMA): located at https://msc.fema.gov/portal These maps display areas that fall within the 100-year flood boundary. Information pertinent to bridge hydraulics analysis includes whether the bridge resides in a FEMA floodway (see Section 2.2). Additionally, the map's 100-year elevations can provide a check for modeling results for the area. It is not unusual for the FEMA listed elevations to differ significantly from hurricane storm surge modeling results developed at an individual site. Many of FEMA's older coastal studies were performed via application of either the TTSURGE or FEMA SURGE two-dimensional models, models driven with atmospheric (wind and pressure) boundary conditions. A Joint Probability Method analysis of the models results determined the return periods of surge elevations. The last time the FEMA SURGE model was used in a new or updated flood insurance study to revise the FIRMs occurred in the late 1980's. Thus, deviation in 100-year flood elevations from the published FEMA values can be attributed to differences in the numerical models, boundary conditions, inclusion of wave setup, as well as in the post-simulation analysis. More recently, FEMA has initiated coastal restudies of locations throughout Florida employing more up to date modeling and statistical analyses. As the new maps become available, they will replace older currently available maps.

4.1.5 Inland Controls

Data collection for inland controls follows the same recommendations as for the upstream controls of riverine analysis (Section 3.1.6).

4.1.6 Site Investigation

A field investigation is recommended for all new bridge construction. Refer to Section 3.1.7 for a detailed list outlining key items to be collected during site investigations. In addition to this list, data collection at tidal bridges should also include the following:

- Look for evidence of wave scarping in bridge approaches.
- Note directions of largest fetches.
- Look for evidence of wave overtopping of seawalls and bulkheads.
- Note scattering of rubble riprap at toes of revetments, seawalls, and bulkheads by waves.

4.2 Hydrology (Hurricane Rainfall)

There can be significant surface runoff from land during hurricane events associated with heavy rainfall. For coastal areas, even though the storm surge is the larger concern, surface runoff may increase or decrease the surge effects depending on the phasing between the two (Douglass and Krolak, 2008).

The reference USACE (1986) *Engineering and Design Storm Surge Analysis EM 1110-2-1412* provides a methodology for estimating rainfall associated with landfalling hurricanes. The methodology is applicable for the area within 25 miles of the coast. It provides graphs of point rainfall depth for a given frequency and a given distance from the left or right of the storm track. The rainfall is considered to vary uniformly along the coast for any given storm. Also, the rainfall depths are considered uniform along any line parallel to the storm track extending across the 25 mile wide zone. Point rainfall graphs (Figure 4-9) are provided for selected frequency levels at either 6 or 12 hour intervals before landfall and after landfall. The reference provides techniques for estimating rainfall associated with hurricanes traveling at high, moderate, and slow speeds by multiplying the rainfall from the graphs by a ratio coefficient that is a function of area.

Alternatively, as a rule of thumb, a steady 10-year discharge may be assumed over the duration of the surge. This is likely to be conservative inlight of a recent examination of hurricane rainfall in North Carolina that suggests that a 2 year rainfall well represented historical storms in that state (OEA, 2011). Bridges over streams with short times of concentration (< 4 hours) are more likely to have coincidence between the storm surge passage and high runoff values. Historical review of the timing and magnitude of runoff at gaged locations near the project site can provide additional insight into the appropriate return period flow rates for boundary conditions. At a minimum, a sensitivity study should be performed to characterize the influence of the runoff magnitude on the flow properties at a subject bridge during a surge event.

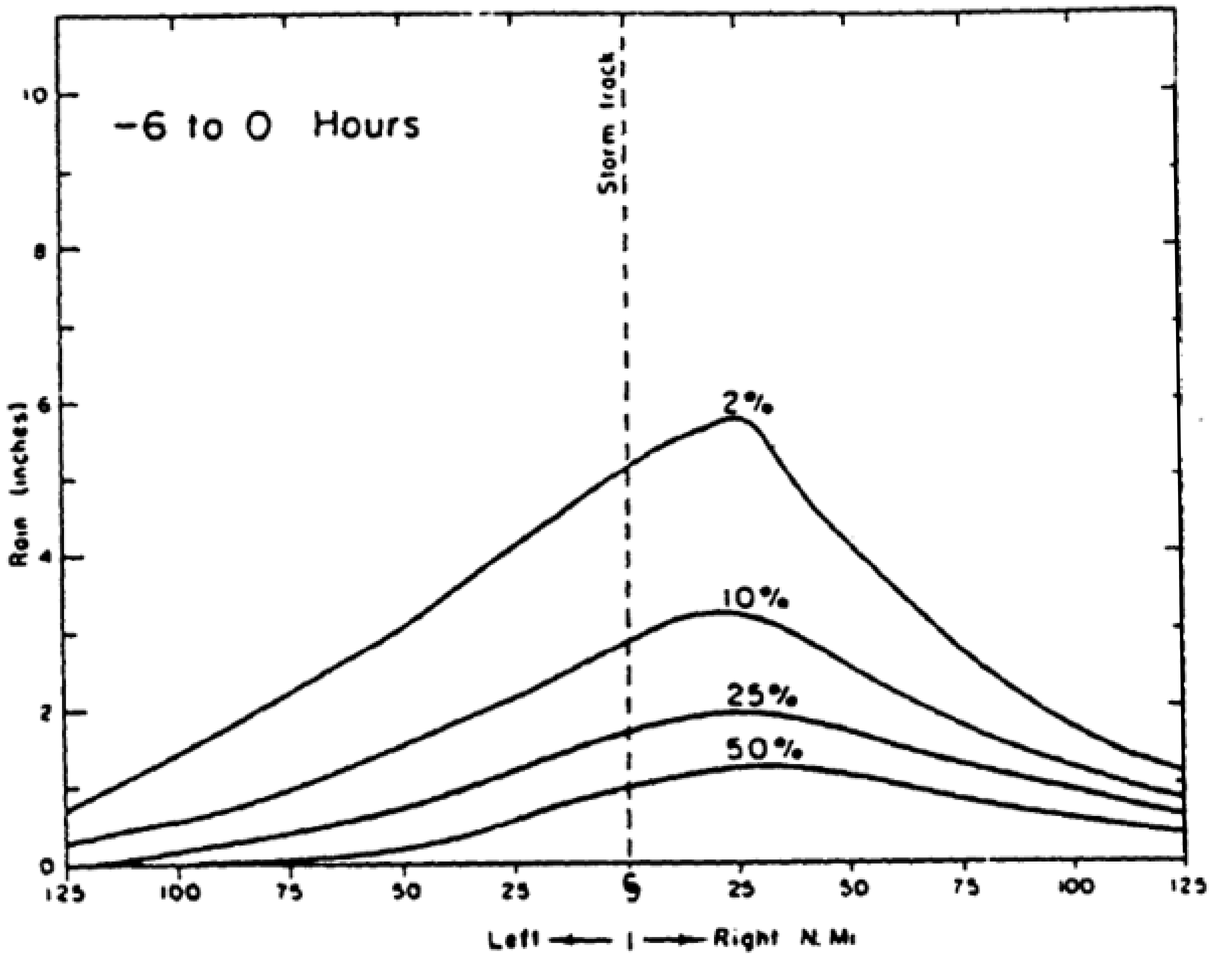


Figure 4-9 Rainfall for Selected Frequency Levels for Six Hours before Landfall (Source: USACE 1986)

4.3 Model Selection

Engineers performing hydraulic studies must weigh several factors when selecting a modeling approach including:

- Types of models (e.g., one-dimensional vs. two-, or three-dimensional models; finite-element vs. finite-difference models);
- Site conditions (e.g., embankment skew, multiple openings, etc.);
- Data availability (e.g., survey data, design flows/stages, etc.);
- Familiarity with the model;
- Schedule; and
- Budget.

The engineer should weigh all the factors mentioned above and select the appropriate model for the application. NCHRP Web-Only Document 106: Criteria for Selecting Hydraulic Models (Gosselin et al., 2006) provides a decision analysis tool and guidelines for selecting the most appropriate numerical model for analyzing bridge openings in riverine and tidal systems. The decision tool takes the form of a decision matrix that incorporates all the factors that influence model selection. These include site conditions, design elements, available resources, and project constraints. The utility of the decision tool is that it presents a formal procedure for the selection of the appropriate model to apply rather than rely on an intuitive process. Figure 4-10 presents an example where the engineer is selecting between one- and two-dimensional models. The figure shows the scoring and weighting of different aspects of the project with the final selection of the one-dimensional model based largely on advantages in scheduling. The selection procedure provides an easy-to-understand and defensible method for presentation to non-technical readers or policy makers. Also, through its application, it clearly identifies which features of the project are most important in the model selection for a specific application.

Design Criteria	Weight	One Dimensional Model		Two Dimensional Model	
		Score 1=low 3=medium 5=high	Weight x Score	Score 1=low 3=medium 5=high	Weight x Score
Site Conditions	**(1-10)**				
Bridges over Meandering Rivers	2	3	***6***	5	***10***
Bridges with Asymmetric Floodplains	7	3	***21***	5	***35***
Design Requirements	**(1-10)**				
Riprap	9	3	***27***	3	***27***
Pier Scour Calculation	3	3	***9***	3	***9***
Other Considerations	**(1-10)**				
Modeler Experience	3	5	***15***	5	***15***
Scheduling	10	5	***50***	3	***30***
Data Availability	3	5	***15***	5	***15***
Totals (Sum of Weight x Score)			***143***		***141***

Figure 4-10 Example of Model Selection Worksheet from NCHRP Web-Only Document 106

For tidal analyses, in general, one-dimensional modeling is appropriate for waterways with well-defined channels in areas that are not subject to lateral overtopping. An example would include rivers or canals that discharge directly to the open coast (e.g., Suwannee River, Florida Barge Canal). More complex waterways and flow circulation will require two-dimensional modeling. Examples requiring two-dimensional flow modeling include:

- Multiple interconnected channels
- Influence of multiple inlets
- Overtopping of barrier islands
- Bridges over tidal inlets
- Bridges over causeway islands
- Bridges through island chains

For wave models, a similar selection procedure is not currently available. Selection of the appropriate model is left to the engineer's experience and discretion after carefully weighing the required design criteria and model features. Model selection, once thoroughly considered, should be confirmed with the District Drainage Engineer.

4.3.1 Storm Surge Model

Development of design hydraulic parameters at a bridge location requires the model's capability to simulate storm surge propagation from an open coast to the bridge site. This necessitates application of an unsteady-state model. The following partial list includes several commonly employed one- and two-dimensional models for simulating hurricane storm surge:

- Advanced Circulation Model (ADCIRC) 2DDI
- TUFLOW
- DELFT3D
- FESWMS 2DH
- HEC-RAS 3.1.1 and up
- MIKE 11 HD v.2009 SP4
- MIKE 21 (HD/NHD)
- TABS RMA2
- UNET 4.0

4.3.2 Wave Model

Development of design wave climate parameters can employ either numerical models or deterministic methods. The references Coastal Engineering Manual (USACE 2002) and Shore Protection Manual (USACE 1984) both provide empirical equations and methodologies for calculating wave parameters over open water fetches. The following partial list includes several commonly employed tools and models for simulating hurricane generated waves:

- ACES
- MIKE 21 Flexible Mesh Spectral Wave Model
- MIKE 21 Nearshore Spectral Wave Model (NSW)
- RCPWAVE
- Simulating Waves Nearshore (SWAN)
- Steady-State Spectral Wave (STWAVE)

4.3.3 Model Coupling

Model coupling refers to the interaction between the wave and surge models when simulating hurricanes. With no coupling, the surge and wave models are performed independently. Since the wave model requires a water surface elevation for input, this can lead to under prediction if the surge is not taken into account. Figure 4-11, taken from Sheppard et al. (2006) *Design Hurricane Storm Surge Pilot Study, FDOT Contract No. BD 545 #42*, displays wave simulation modeling of Hurricane Katrina at a location offshore of Mississippi. In the figure, the “Without SS” curve is the wave height simulated without the storm surge as an input boundary condition. The “With SS” curve includes storm surge as an input into the wave model. Including storm surge produces a 4 meter increase in the predicted significant wave height.

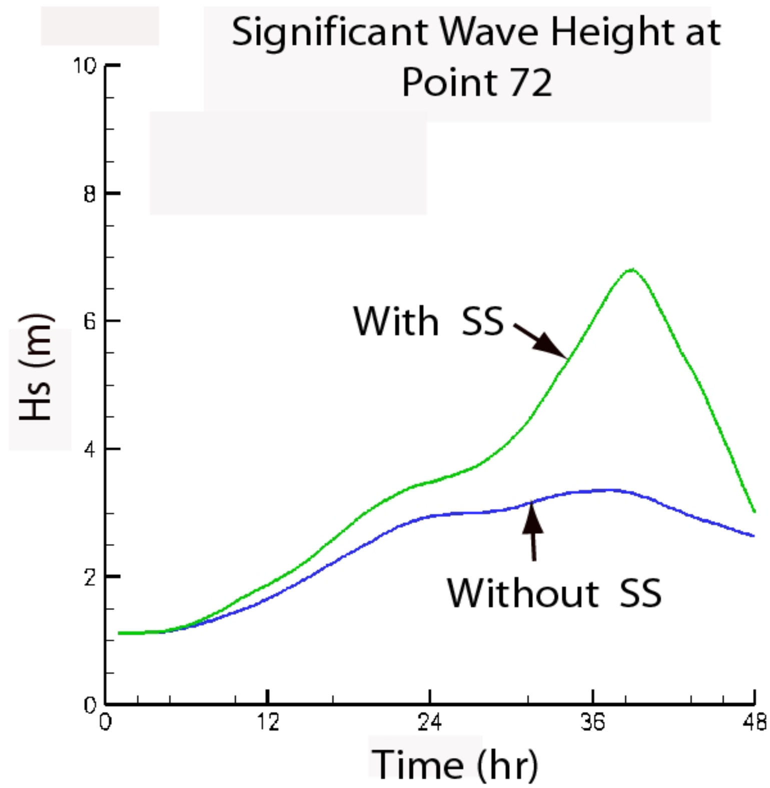


Figure 4-11 Wave Height Simulation during Hurricane Katrina with No Coupling (Without SS Curve) and with One Way Coupling (With SS Curve) (Source: Sheppard et al. (2006))

With one-way coupling, results (water elevations and currents) from the surge model are input into the wave model. This leads to more accurate prediction of the wave climate. With two-way coupling, results from each model are transmitted between the models at regular intervals. The wave model receives the simulated surge elevations and currents as an input, and the surge model receives the wave radiation stresses (a source term in the momentum equations that gives rise to wave setup) as an input. In general, -twoway coupling provides the most accurate predictions.

4.4 Model Setup

Model setup involves development of the model inputs for the hydraulic or wave model. It includes defining the model domain, assigning friction (roughness), creating the model geometry, and development of boundary conditions.

4.4.1 Defining the Model Domain

The model domain is the spatial coverage of the model upstream and oceanward of the bridge. The limits of the model extents are different for storm surge modeling than for riverine flood modeling. The model domain oceanward should extend to the point where boundary conditions can be well described. For storm surge studies, this is generally the open coast. Application of storm surge hydrograph boundary conditions, developed for the open coast, at upland locations (e.g., at river entrances on estuaries or bays) will result in overly conservative estimates of both surge elevation and flow rate at the bridge location. If the model involves wind and pressure boundary conditions rather than a hydrograph, the model should extend far enough offshore to accurately describe the coastal effects (wind and wave setup) that contribute to the storm surge.

At a bridge, the accuracy of the surge hydrograph will be a function of the model resolution between the open coast and the bridge location. Definition of the major tidal waterways between the ocean and the bridge is recommended. Often, this includes extending the model not only from the closest tidal inlet to the bridge, but also to nearby inlets as well. This is particularly true for bridges located on or near intracoastal waterways.

Flow through the bridge is a function of the storage upstream (inland) of the bridge. The model domain should extend far enough upstream and upland to accurately describe the flow prism during the surge event. Underestimating the storage area upstream of a bridge will result in underestimation of flow and scour at the site.

Definition of wave model extents will depend on the purpose of the wave model. If the modeling results will provide wave radiation stresses for the surge model, then the wave model should include similar offshore and lateral extents as the surge model as well as the interior waters. If the purpose of the wave model is only to provide local wave conditions at the site, then the model should extend from the bridge to the shoreline in all directions so that the fetch (distance that the wind blows over a water body) is adequately described in all directions.

4.4.2 Roughness Selection

Specification of the roughness parameters for tidal analyses follows the same procedures as for riverine conditions (Section 3.4.2). Some surge models can include different bottom stress parameterizations. For example, ADCIRC provides options for linear and quadratic bottom friction assignment in addition to a Manning's n formulation. Refer to the individual model documentation for roughness specification other than Manning's coefficient. Most wave models also include options for bottom friction. For example, the SWAN model includes frictional dissipation via the methodologies of JONSWAP, Collins, and Madsen. Again, refer to the software documentation for recommend values of friction parameters.

Roughness values through developed areas, inundated during the surge, are especially difficult to predict. The density of buildings is a key influence on roughness in these areas. Calibration data is helpful in targeting the proper n-value.

4.4.3 Model Geometry

Model geometry refers to the spatial resolution incorporated into the model to describe the waterway and overbank bathymetry and topography. For one-dimensional models this refers to not only the cross section locations, but also the number of points across the cross section. For two-dimensional models, this refers to the nodes and elements that comprise either the finite element mesh or the finite difference grid.

4.4.3.1 One-Dimensional Models

Specification of one-dimensional model geometry for tidal analyses follows the same recommendations as for riverine analyses (Section 3.4.3.1). In general, the only difference is the size of the model domain which is discussed in Section 4.4.1.

4.4.3.2 Two-Dimensional Models

Specification of two-dimensional model geometry for tidal analyses follows the same recommendations as for riverine analyses (Section 3.4.3.2). Again, the only difference is the size of the model domain, discussed in Section 4.4.1, which can extend into the offshore area. Adequate resolution should be incorporated into the model to resolve tidal inlet and offshore features (such as flood and ebb shoals, or coastal structures) that affect the flow properties of the inlets.

4.4.4 Boundary Conditions

Boundary conditions for tidal analyses depend upon the types of simulations, the models employed, and site specific properties. One-dimensional modeling of coastal bridges during surge events typically involves specification of an upstream flow boundary condition and an oceanward stage boundary condition where the stage is an open coast hurricane hydrograph. Two-dimensional surge modeling has more options for boundary conditions. These can include:

- Specifying the stage and flow similar to the one-dimensional model.
- Same conditions as above with an additional wind boundary condition specified over the entire model domain.
- Tidal constituent boundary conditions on the offshore, upstream flow, and meteorological forcing (wind and pressure) at each node.

This section describes several of the possible model boundary conditions for coastal bridge hydraulics analyses.

4.4.4.1 Upstream Flow Boundary Conditions

Specification of upstream flow boundary conditions follows the same recommendations as those with riverine flow boundary conditions (Section 3.4.4.1) with some exceptions. In tidal analyses in Florida, inland boundaries are typically located far from the bridge locations. This is done to accurately describe the storage inland of the bridge which is a significant factor in determining flow through the bridge. For example, in general, bridges with low elevation, wide floodplains inland will experience more flow during a surge than bridges with high elevation, narrow floodplains inland. This is because the greater, lower, inland storage will be less responsive to inflow, resulting in a lower inland stage during the flodd flow of the hurricane surge.

The hydrology for the boundary condition should be developed for the bridge location rather than at the location where the boundary condition is applied. Hurricane hydrology is discussed in Section 4.2.

4.4.4.2 Storm Surge Hydrographs

A frequent type of coastal bridge hydraulics analysis involves application of an open coast storm surge hydrograph as the oceanward boundary condition. Fortunately, in Florida, several agencies have developed coastal surge elevations associated with several return period intervals. In a study for the FDOT, Sheppard and Miller (2003) reviewed the literature to determine what information was available regarding 50-, 100-, and 500-year return interval open coast storm surge peak elevations and time history hydrographs. Based on information from the literature review, the study developed recommendations for selecting ocean boundary conditions for modeling inland storm surge propagation in Florida’s coastal waters. From their findings, Sheppard and Miller recommended that the FDOT employ the storm surge heights for 50-, 100- and 500-year return interval hurricane storm surges developed by the FDEP. This recommendation was made on the basis that FDEP had included all of the major surge generation mechanisms (astronomical tides, wind setup, wave setup, etc.) in their analyses and that they had compared their results with near coast water marks in buildings where possible. One failing of the FDEP values was that only the counties with sandy beaches (25 of the 34 coastal counties) in Florida were analyzed by FDEP. To address this problem, Sheppard and Miller developed surge elevations by interpolating values from the surrounding counties using FEMA and NOAA results as guides. Figure 4-12 presents the locations of the FDEP developed elevations as well as the locations of the interpolated elevations (in italics).

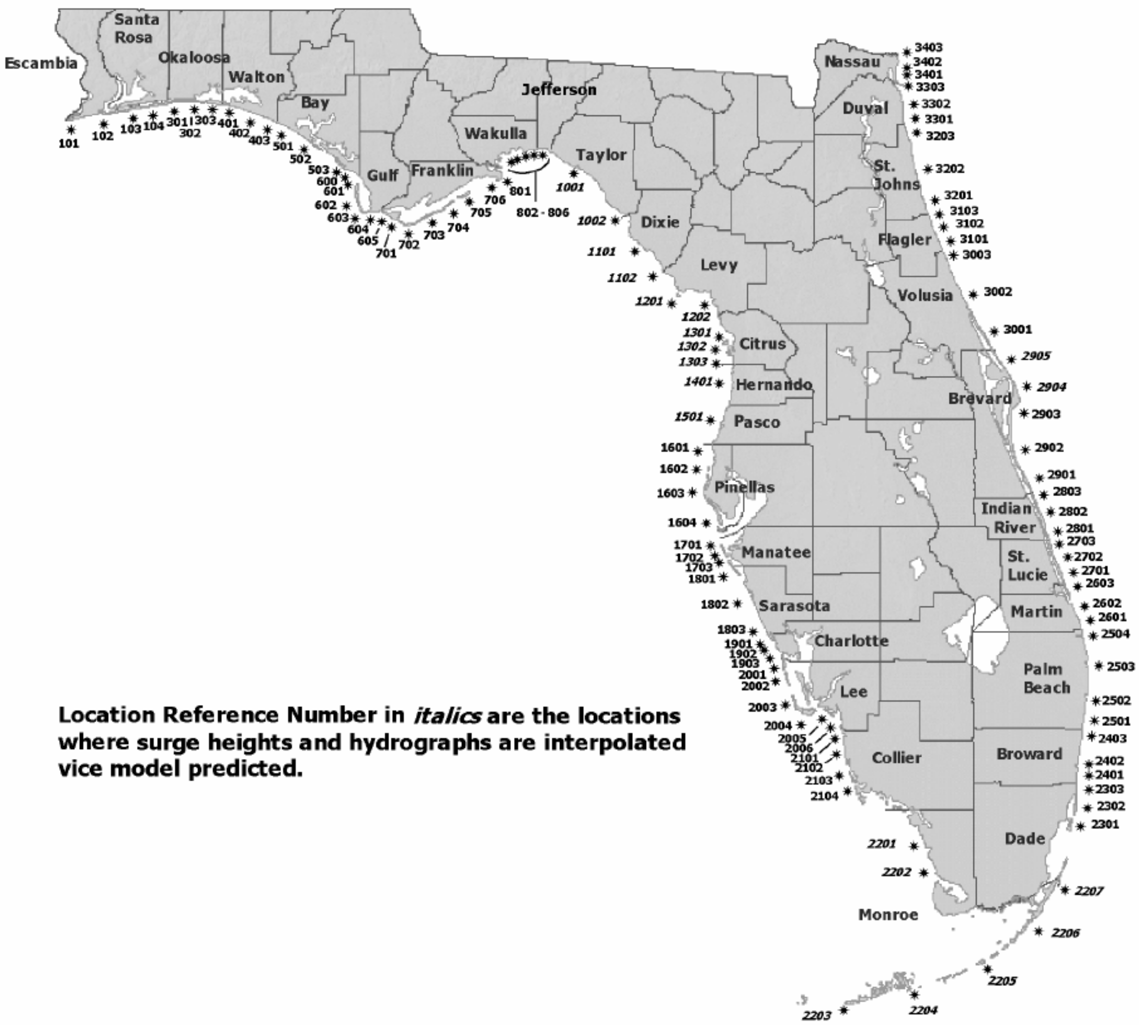


Figure 4-12 Storm Surge Peak Elevation and Hydrograph Locations

The above guidance and supporting report are available at the following URL:

http://www.dot.state.fl.us/rddesign/Drainage/FCHC.shtm

4.4.4.3 Hurricane Generated Winds

For bridges located near the ends of bays and estuaries, wind setup can be a major contributor to the surge elevation. Figure 4-13 illustrates the effects that local wind setup can have on surge elevations. It displays results of a hindcast of the 1852 Unnamed Hurricane in Tampa, FL at the Courtney Campbell Bridge near the northern end of Old Tampa Bay. Hindcasts were performed with meteorological (spatially and temporally varying wind and pressure fields) boundary conditions and tidal constituent forcing on the offshore boundary. The line labeled Surge and Wind includes the "real" hindcast. For the simulation represented by line labeled Surge Only, the wind speeds in the boundary condition file were set to zero only at inland locations. Thus, this line represents the case where surge at the bridge is only created from propagation of the surge hydrograph inland.

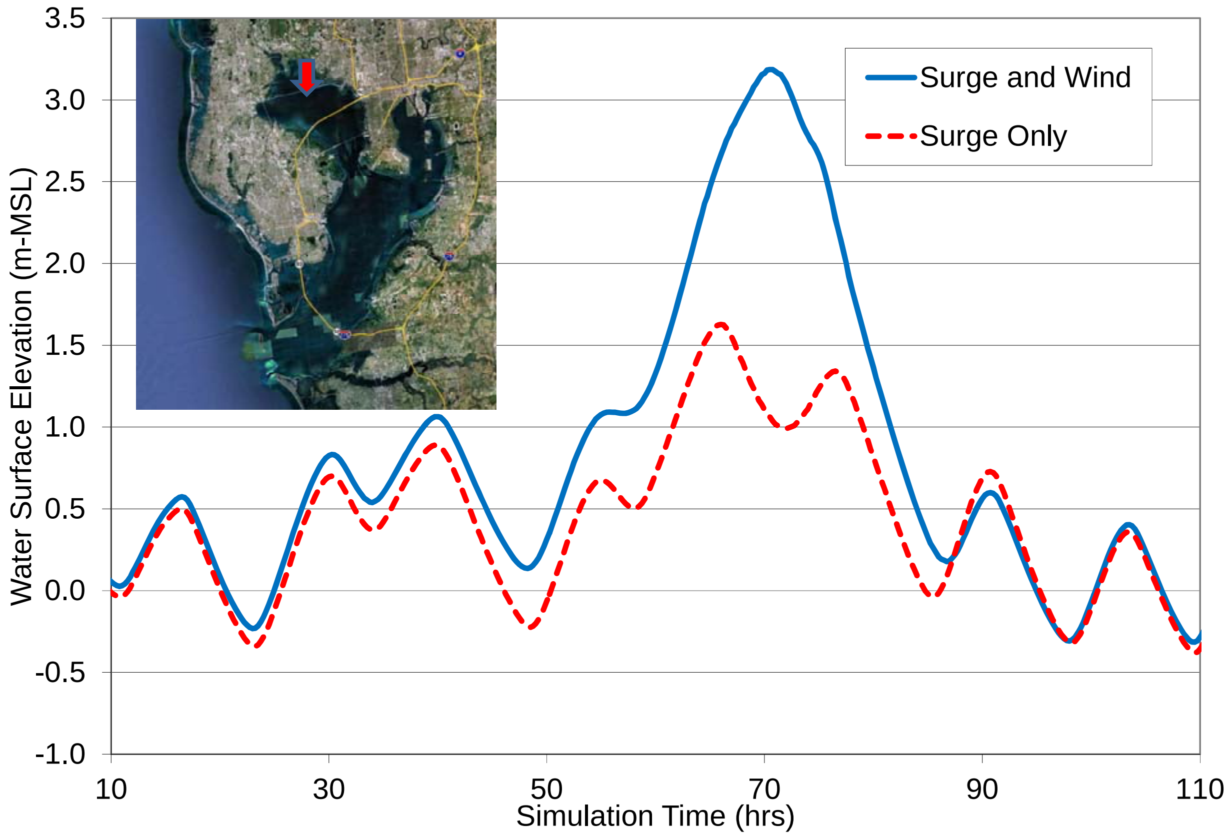


Figure 4-13 Surge Elevations at the Courtney Campbell Bridge Location during the 1852 Unnamed Hurricane both with and without Local Wind Effects

Another example of how bridge location affects the importance of wind setup is seen in the hindcast of Hurricane Ivan in 2004 that made landfall near Pensacola, FL. Figure 4-14 displays the calculated storm surge elevation time series at the I-10 Bridge over Escambia Bay (red line) and at the Pensacola Bay Bridge (blue line). Located near the back Escambia Bay, the I-10 Bridge experienced a significantly higher storm surge than did the Pensacola Bay Bridge even though the Pensacola Bay Bridge was located nearer to the inlet. This is directly attributable to the wind setup that occurred near the back of Escambia Bay.

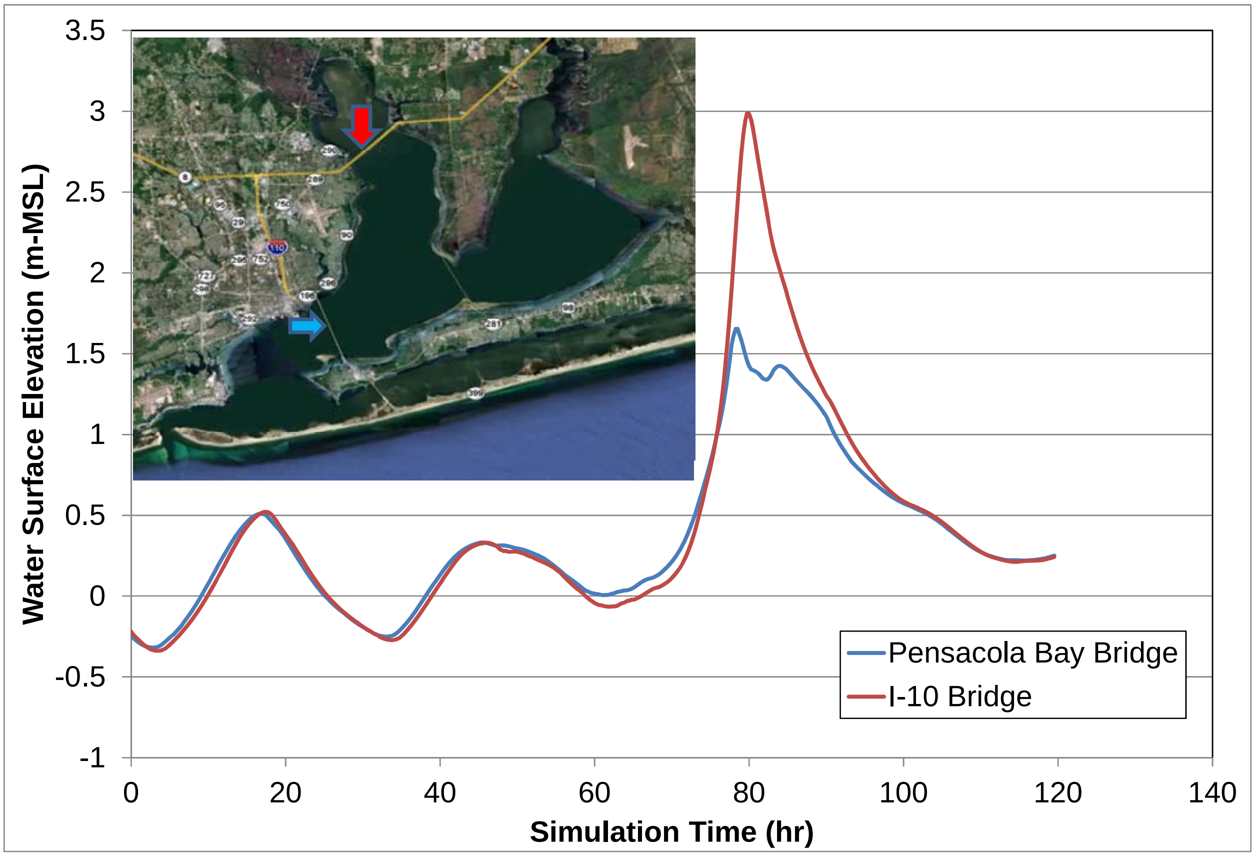


Figure 4-14 Hindcasted Surge Elevations at the I-10 over Escambia Bay Bridge and Pensacola Bay Bridge during Hurricane Ivan 2004.

Shown above, hurricane winds can play a major role in describing surge propagation. The reference *AASHTO Guide Specifications for Bridges Vulnerable to Coastal Storms* (AASHTO 2008) provides a methodology for determining peak design wind speeds for a number of mean recurrence intervals. It references ASCE Standard 7-05 as the source for determining design wind speeds throughout the country. The AASHTO Specification also states that if design coastal storm wind speeds exist at a site then these values should be used.

In Florida, Dr. Michel Ochi at the University of Florida (Ochi 2004) presents a methodology for predicting the hurricane landfall wind speeds along the Florida coast. He examined tropical cyclones (including hurricanes) which landed on or passed nearby the Florida coast from the NOAA hurricane database HURDAT. He divided the Florida coast into 15 districts (Figure 4-15), and developed expected extreme values for different return periods. Table 4-2 gives the expected maximum sustained (1-min average) wind speed for landfalling hurricanes calculated from Ochi's methodology.

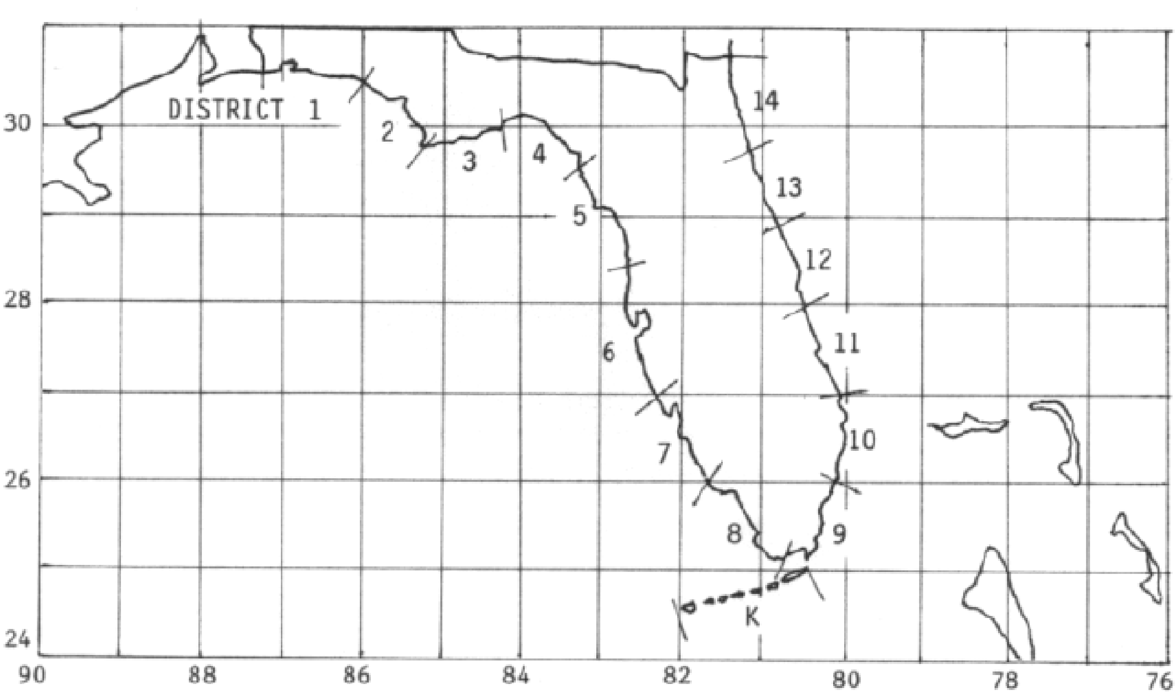


Figure 4-15 Locations of Coastline Division Employed in Wind Speed Analysis by Ochi (2004) (Source: Ochi 2004)

Table 4-2 Example of Extreme Landfall Wind Speeds for Florida via the Ochi Methodology

	Most Probable Maximum Sustained Wind Speed (mph)		
District*	50-year	100-year	500-year
K	130.9	141.4	162.3
1	110.5	120.5	140.5
2	107.0	116.6	135.7
3	97.5	107.5	127.5
4	82.9	88.8	100.3
5	104.0	115.3	138.4
6	89.7	101.3	125.1
7	96.8	112.4	144.9
8	127.1	137.9	159.4
9	136.5	148.0	171.2
10	140.2	147.7	162.8
11	104.0	112.0	127.6

* Districts 12-14 did not have enough storm impacts to generate a confident statistical analysis.

4.4.4.4 Hurricane Hindcasts
Hurricane hindcasts simulate the wave and surge climate associated with a unique historical hurricane (Section 4.5). These types of simulations are primarily performed with two-dimensional models. Boundary conditions typically take the form of temporally and spatially variable wind and pressure fields (meteorological boundary conditions) applied over the entire model domain. Additional boundary conditions include an offshore stage boundary condition equal to the daily tidal fluctuation at the condition locations. This can take the form of either specified tidal elevation time series (e.g. tidal hydrographs) or be a feature of the model as selected tidal constituents (e.g. ADCIRC). The best source for tidal hydrographs is NOAA's Center for Operational Oceanographic Products and Services (http://www.co-ops.nos.noaa.gov/) for real-time and measured tidal gage data as well as tidal prediction.

Hurricane wind and pressure fields can be developed in a number of ways. They range from simple analytic models (e.g. Holland 1980) to three-dimensional modeling. Several agencies have performed hindcasts of specific storms including FEMA, NOAA, and USACE. These hindcasts are sometimes available upon request. Additionally, several commercially available sources of hind cast data also exist.

4.4.5 Bridge
When constructing a model to simulate hurricane surge propagation and wave climate, accurate representation of the bridge and its influence on the hydrodynamic processes is necessary. In general, the same techniques employed for riverine analyses also apply to analysis of coastal bridges during storm surges.

4.4.5.1 Roughness
Roughness specification at bridge cross sections for tidal analyses follows the same recommendations as for riverine analysis (Section 3.4.5.1).

4.4.5.2 Bridge Routine
Selection of the appropriate bridge routines for tidal analyses follows the same recommendations as for riverine analysis (Section 3.4.5.2).

4.4.5.3 Piers
Incorporating the effects of bridge piers into the hydraulic model for analysis of coastal bridges follows the same procedure as for riverine bridges (Section 3.4.5.3). For two-dimensional modeling, typically, piers are not modeled directly because their planform areas are significantly smaller than the areas of elements that resolve the bridge openings. However, several options exist for including the effects of bridge piers. Several models incorporate the loss effects into the hydraulic computation routines. An example is FST2DH (part of FESWMS). FST2DH contains an automatic routine that accounts for the effect of piers or piles on flow by increasing the bed friction coefficient within elements that contain them (Froelich 2002). ADCIRC also contains routines for

incorporating the effects of bridge piers through a loss term in the momentum equations due to the pier drag (http://adcirc.org/home/documentation/special-features/).

Gosselin et al. (2006) examined the effects of resolving bridge piers through element elimination in cases where the pier width was a large percentage (5-35%) of the overall bridge cross section top width. The piers were incorporated by deleting elements within the mesh occupied by the piers. The authors compared results of the two-dimensional modeling with one-dimensional modeling results for the same geometry and flow conditions. The results compared well at the bridge cross section, but compared poorly downstream of the piers. The authors concluded that whereas the one-dimensional model incorporates the frictional losses from the piers through an increase in the wetted perimeter, by modeling the piers through element deletion, the two-dimensional model does not account for frictional losses if a slip boundary condition along the model edges is employed. Rather, losses from the piers are attributed to the momentum losses associated with the creation of the secondary flows around the piers and in the wake region.

Regarding wave models, most publically available software do not include effects of bridge piers on wave propagation.

4.5 Simulations

Following construction of the surge and wave model domains, development of the boundary conditions, and specification of the input model parameters, the model simulations can begin. This section describes the model simulations typically performed as part of the hydraulic analysis of a coastal bridge.

4.5.1 Model Calibration

Before performing design simulations, the surge and wave model should be properly calibrated. Model performance through calibration and verification are often evaluated through both qualitatively and quantitatively, involving both graphical comparisons and statistical tests. For surge models, calibration should include both tidal propagation simulations and historical storm events. For tides, calibration is achieved by comparing tidal simulations for a period of record to either measured data collected at specific locations or comparison to widely available NOAA predictions at several locations. FEMA (2007) recommends that tidal calibration should be achieved to better than 10% in both amplitude variation throughout the domain and phase variation. In general, flow rate or velocity calibration is typically not performed due to lack of reliable data. Flow calibration is typically more difficult to achieve than for water surface elevation data. However, if this data is available, acceptable limits for calibration should be more generous than those for tidal amplitude, yet still provide reasonable representation of the flow. FEMA also indicates that failure to achieve calibration may be indicative of inadequate grid resolution, especially at inlets and other critical points. Zevenbergen et al. (2005) provides a thorough description of model troubleshooting including suggestions for addressing model execution failures, numerical instability and calibration problems. These suggestions are contained in the table below:

Table 4-3 Suggestions for Model Calibration (Source: Zevenbergen et al. (2005))

If a model fails to execute check:	The causes of numerical instability are:	Model calibration will be affected by:
• Program output error messages • Missing input data • Incorrect input data • Missing input files • Inconsistent input data	• Computational time step too long • Lack of geometric refinement • Wetting and drying problems • Weir flow	• Appropriate model extents • Accuracy of model bathymetry • Correct datum conversions for bathymetry • Correct datum conversions for tide gages • Inclusion of wind effects • Inclusion of appropriate upstream inflow

Calibration to known storm events is significantly more complex than tidal calibration. Ideally, the calibration would include accurate measurements of both the model inputs (surge hydrograph or wind and pressure fields) as well as accurate surge measurements at locations throughout the model domain (gage measurements or high water marks). This is seldom the case. In fact, high water marks provide one of the more difficult data sources to calibrate to since they often contain effects of local wave climate and can vary significantly in close proximity to each other. If reliable information is available, calibration to a known storm event is recommended. Comparison of model results with gage data or high water marks helps identify problems with domain extents, model resolution, grid resolution, or friction assignment.

Calibration of wave models is also difficult because calibration data is rarely available. If the data is available or acquired, then the calibration process should involve qualitative and quantitative comparisons of measured and simulated wave height, period, and direction. However, if measurements are unavailable, then the coastal engineer should demonstrate that the wave model simulations provide reasonable results, were performed employing accepted standards for input parameters, and incorporate an appropriate level of conservatism.

4.5.2 Storm Surge Simulations

Storm surge simulations should include, at a minimum, the design and check events for scour and the design frequency event for the bridge as specified in Section 2.3 (e.g., the 50-year for mainline interstate, high use or essential bridges). Results from the simulations include time series of water surface elevation, velocity, and flow rate. Simulation results should be extracted not only at the bridge cross section, but at locations upstream of the bridge piers (for local pier scour calculation). The number of locations is related to the length of the bridge. For shorter bridges, extracting conditions at the location of the maximum velocity will be sufficient. For longer bridges, there will be greater variation in velocity magnitude and direction. Thus, results should be extracted at a greater number of locations to resolve the variation. Flow rates and water depths should be extracted upstream of the bridge constriction for contraction scour calculations.

Figure 4-16 displays an example of water surface elevation and velocity time series during the 100-year return period hurricane through Wiggins Pass near Naples, FL. The figure is typical of storm surge propagation through coastal waters. A peak in velocity magnitude precedes the peak in water surface elevation as the surge propagates inland. A second peak in velocity magnitude occurs as the surge recedes. The magnitude, phase and duration of the velocity magnitude peaks are a function of the shape of the surge hydrograph and the response of the interior waterways.

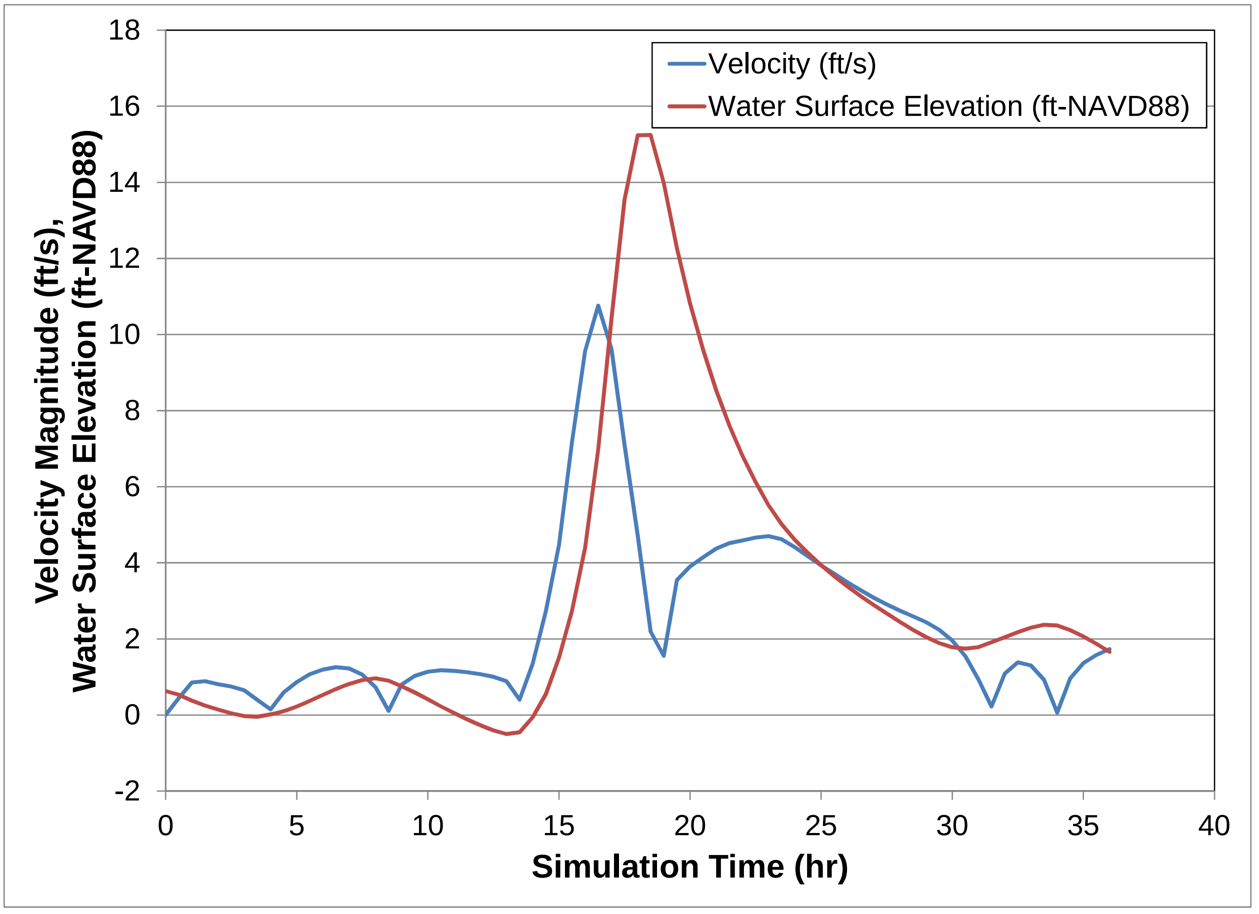


Figure 4-16 Example of Water Surface Elevation and Velocity Time Series during the 100-year Return Period Hurricane through Wiggins Pass near Naples, FL

4.5.3 Design Considerations

Coastal bridges are typically not located in FEMA floodways and typically not examined for their effects on backwater. The bridge location and profile are typically set for reasons related to right-of-way, environmental impacts, navigation, corrosion, etc. rather than for bridge hydraulics (backwater impacts). The engineer should review the recommendations contained in Section 3.5.3 for riverine studies to determine whether they apply for a particular coastal bridge location. Situations that do require comparison of existing and proposed conditions include: major modifications to the bridge profile or to the floodplain (e.g., causeway islands), bridge replacements that transition from spill-through to wing-wall abutments, etc.

An additional design consideration involves vessel collision. The LRFD specifications require using the “average current velocity across the waterway.” Determining this velocity for tidal flows requires a separate simulation of the spring tidal flows. The average current velocity should correspond to the peak velocity occurring over this simulation.

4.5.4 Wave Simulations

Wave parameters are necessary for both calculation of wave forces on bridge superstructures and for design of abutment protection. According to AASHTO (2008), wave forces (discussed in Section 4.6) are calculated from 100-year return period wave conditions only. Additionally, abutment protection is similarly designed to resist the 100-year wave conditions. The wave model should simulate, at a minimum, the 100-year return period hurricane generated wave conditions at the site.

Time dependent (unsteady) wave modeling gives more accurate design wave conditions at the bridge location. As an alternative, steady-state modeling of the wave conditions during the peak storm surge provides sufficient, though conservative, design conditions. Inputs to the wave modeling will include design wind speeds, water surface elevations, bathymetry/topography, and wind direction. If the wind direction is unknown, the wave modeling should include, at a minimum, steady-state simulations of the wind field along the direction of the longest fetches (Figure 4-17).

Wave models typically provide the significant wave height and the peak period. The significant wave height is a statistical parameter representing the average of the highest one-third of the waves in a wave spectrum. The peak period is the wave period corresponding to the maximum of the wave energy spectrum. For design of bridge superstructures, AASHTO recommends employing the maximum wave height rather than the significant wave height. The AASHTO equation for converting between the two is $H_{max} = 1.80H_{significant}$.

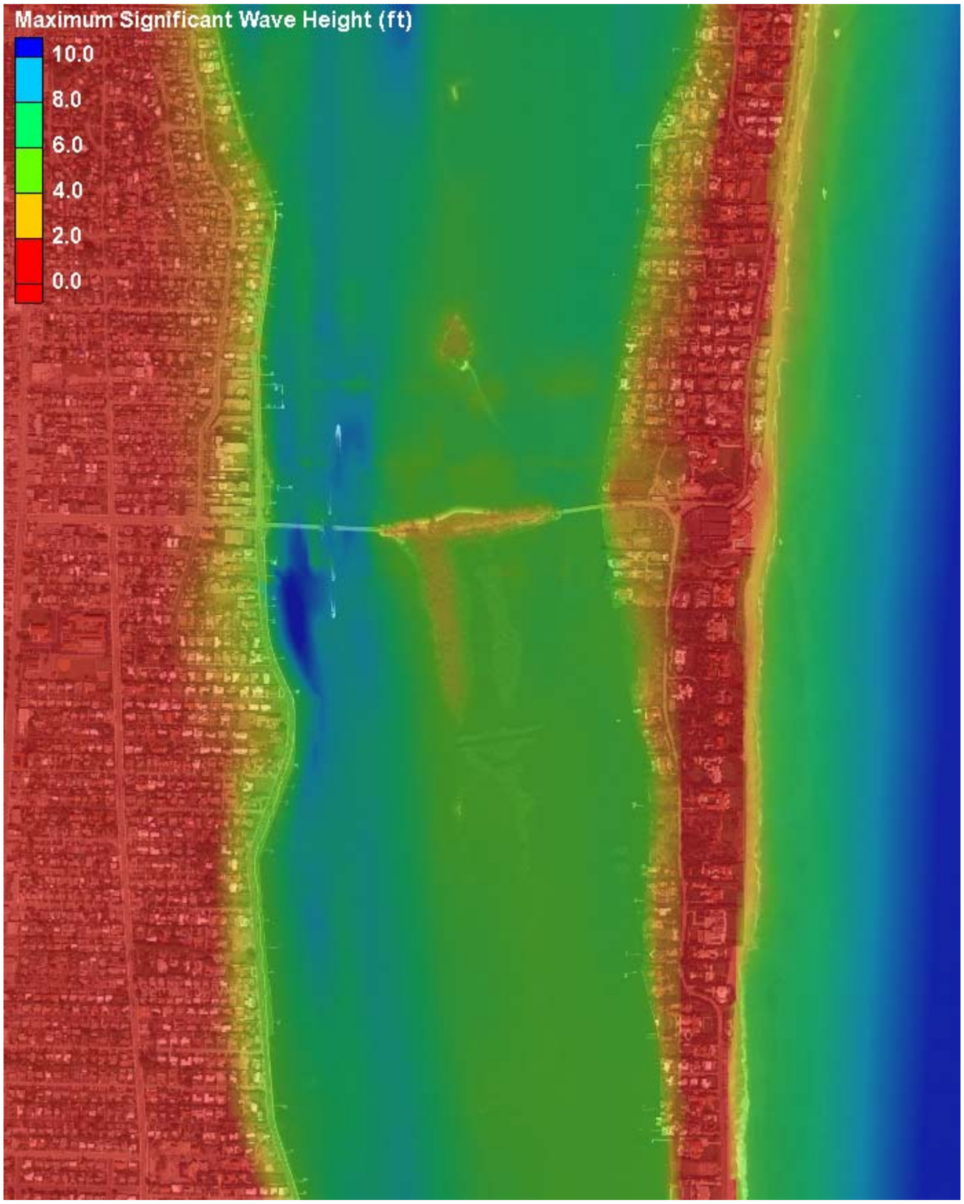


Figure 4-17 Example of Significant Wave Height Contours from Wave Modeling

4.6 Wave Forces on Bridge Superstructures

Bridge design must consider wave forces on bridge superstructures to prevent the type of damage experienced at the I-10 Bridge over Escambia Bay during Hurricane Ivan in 2004 (Figure 4-18). Wave forces on bridge superstructures are addressed in Section 9.5 of the Drainage Manual and Section 2.5 of the Structures Design Guidelines. The bulletin provides guidance on applying the specifications in the AASHTO Guide Specifications for Bridges Vulnerable to Coastal Storms to FDOT bridges. It states, for bridges spanning waters subject to coastal storms, the superstructure low chord must have a minimum one foot vertical clearance above the 100-year design wave crest elevation. If this clearance cannot be met, the bridge superstructure should be raised as high as feasible and the bridge superstructure designed to resist storm wave forces. For these bridges, the design strategy depends on the importance/criticality of the bridge when considering the consequences of bridge damage caused by wave forces. If a bridge is deemed to be extremely critical, it would generally be designed to resist wave forces. Bridges that are deemed “Non-Critical” do not require evaluation for wave forces.

Figure 4-18 Damage to the I-10 over Escambia Bay Bridge during Hurricane Ivan (2004)

Figure 4-19 defines the parameters involved in estimating wave forces and moments on bridge superstructures from the AASHTO Specification. The interaction between the wave and bridge superstructure produces vertical (uplift) forces, horizontal forces, and over-turning moments. Computing design surge/wave-induced forces and moments on bridge superstructures requires knowledge of the meteorological and oceanographic (met/ocean) design conditions and the proper force and moment equations. The AASHTO Guide Specifications for Bridges Vulnerable to Coastal Storms provides methods for determining both.

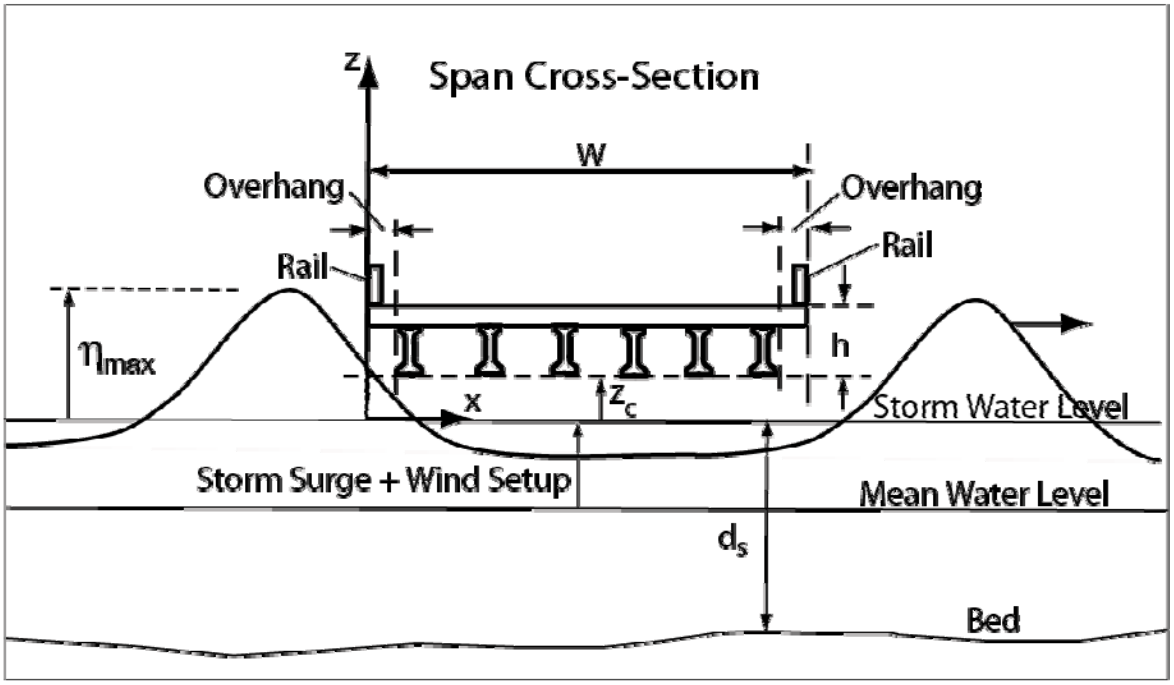


Figure 4-19 Definition Sketch for Wave Forces

The AASHTO Specifications provide a series of parametric equations for calculating the wave forces. Two sets of equations are provided — one corresponds to the time of the maximum vertical force and one corresponding to the time of the maximum horizontal force. For example, for the maximum vertical force, the vertical force is the maximum value experienced by the structure during passage of the design wave and the horizontal force and moment are the values at the time of maximum vertical force.

Chapter 5

Manmade Controlled Canals

Manmade controlled canals have the following typical characteristics:

- Will have some type of downstream control structure such as salt water intrusion barriers, flood control weir, and/or pumps that will regulate the discharge.
- Will not normally flood out of bank, even in a 100-year storm.
- Low design velocities, typically 1 – 3 fps and are often subject to aggradation and require periodic dredging to maintain the needed cross section.
- Abutments will not typically encroach into the cross section of the canal; therefore, there will be no contraction of flow and little backwater caused by the bridge.
- Even if piles are in the flow of the canal, the design discharge will not create substantial scour around the piles because the velocity is low and the pile size is typically small.
- The hydraulic design discharge and stage are usually available from the canal owner.

Given the typically innocuous hydraulic and scour conditions at controlled canal bridges, the prudent level of effort required for the bridge hydraulics analysis is considerably less than the typical bridge. Therefore, the hydraulics report should be abbreviated from the traditional Bridge Hydraulics Report; the following is an outline of the subjects that should be included for controlled canals.

Introduction

- Bridge Location Map
- Waterway owner (LWDD, SFWMD, CBDD, etc.)
- Description of waterway – Man-made, straight, controlled canal, etc.
- Use of canal – Navigation, recreation, flood protection, irrigation, etc.
- Other unusual details

Watershed Description & Flow

- Basin map from water management district or permitting agency
- Any available information on drainage area – Maps, acreage, control structures, etc.

Design discharge and stage information from owner - usually 10- or 25-year (Note: If design information is less than Drainage Manual hydraulic or scour design frequency, discuss with the District Drainage Engineer. Also, if the design discharge and stage are not available, then a full bridge hydraulics analysis is needed)

- Testimony from Bridge Inspection records – Aggradation/degradation, condition of revetment, debris problems, etc.

Channel Excavation, Clearance, and Other Owner Requirements

- Required canal typical section from owner
- Lateral limits of channel excavation – Usually 10 feet beyond bridge drip edge
- Any other pertinent information from owner – Sacrificial pile, bank overtopping, vertical and horizontal clearance requirements, etc.

Scour Estimation

- General scour – Usually none due to lack of natural meander and tendency toward aggradation.
- Contraction scour – None if no overbank flow, unless pile blockage is >10% of the waterway width.
- Typically pier scour on controlled canals is less than 5 feet. With no additional general or contraction scour, the CSU equations may be used.

Abutment Protection

- Refer to Minimum Abutment Protection in Section 4.9 of the Drainage Manual
- Use bedding stone if wave impacts from boat wakes are significant
- Owner may have specific requirements for abutment protection

Bridge Deck Drainage

Refer to Chapter 7, Deck Drainage

Appendix

- Correspondence with owner regarding canal design parameters and requirements
- Pictures
- Bridge Inspection Reports, if significant
- Evidence of field review

Chapter 6

Bridge Scour

The lowering of the streambed at bridge piers is referred to as bridge sediment scour or simply bridge scour. Bridge scour is one of the most frequent causes of bridge failure in the United States and a major factor that contributes to the total construction and maintenance costs of bridges in the United States. Under-prediction of design scour depths can result in costly bridge failures and possibly in the loss of lives; while over-prediction can result in significant cost increases. For these reasons, proper prediction of the amount of scour anticipated at a bridge crossing during design conditions is essential. Policy on scour estimates can be found in the FDOT Drainage Manual.

For new bridge design, bridge widenings, and evaluation of existing structures, scour elevation estimates for each pier/bent shall be developed for the following:

1. Worst case scour condition (long-term channel processes, contraction scour and local scour) up through the design flood event (Scour Design Flood Event).
2. Worst case scour condition (long-term channel processes, contraction scour and local scour) up through the check flood event (Scour Check Flood Event).
3. "Long-term scour" for structures required to meet the extreme event vessel collision load. “Long-term scour” refers to either everyday scour for live-bed conditions or the 100-year total scour for clear-water conditions. Refer to Section 6.2 for further discussion.

Scour estimates shall include the components discussed in the following sections.

6.1 Scour Components

For engineering purposes, sediment scour at bridge sites is divided into three categories:

- Long-term channel processes (channel migration and aggradation/degradation)
- Contraction scour
- Local scour

Scour associated with long-term channel processes is the change in bed elevation associated with naturally occurring or man-made influenced movement of the reach over which the bridge is located. These bed changes are characterized both as horizontal changes (channel migration) and as vertical changes (aggradation/degradation). Local scour is further divided into pier and abutment scour.

6.1.1 Long-Term Channel Processes

Changes upstream and downstream affect stability at the bridge crossing. Natural and man-made disturbances may result in changes in sediment load and flow dynamics resulting in adverse changes in the stream channel at the bridge crossing. These changes may include channel bank migration, aggradation, or degradation of the channel bed. During aggradation or degradation of a channel, the channel bed and thalweg tend to accrete or erode.

Channel stability, as characterized by channel migration and aggradation/degradation of the channel bed, is an important consideration in evaluating the potential scour at a bridge for two reasons. First, because aggradation and degradation influence the channel's hydraulic properties and secondly, bank migration, thalweg shifting, and degradation may cause foundation undermining regardless of whether the bridge experiences the design event.

6.1.1.1 Channel Migration

Lateral channel migration is an important factor to consider when deciding on a bridge's location Factors affecting lateral channel migration include stream geomorphology, bridge crossing location, flood characteristics, characteristics of the bed and bank material, and wash load (Richardson and Davis, 2001).

Techniques for addressing channel migration are found in the FHWA document HEC-20 (Legasse et al., 2001). These techniques generally include critical examination/comparison of historical measurements/records combined with field observations to forecast future trends. Sources of historical records include bridge inspection records, historical maps, historical aerial photography, and historical surveys. In general, at bridges where the waterway exhibits a history of meandering, the hydraulics engineer should consider assuming that the elevation of the thalweg could occur at any point within the bridge cross section, including along the floodplain. If this conservative approach is excessively costly, it may be more cost-effective to mitigate potential future meander by river training or armoring.

For comparison of historical aerial photographs, the FDOT Surveying and Mapping Office currently maintains an archive for historical aerial photography called the Florida Aerial Photography Archive Collection (APAC). It can be accessed via the Aerial Photography Look-Up System (APLUS) at: http://www.dot.state.fl.us/surveyingandmapping/aerial_main.shtm . Additionally, the University of Florida maintains an online archive of historical aerial photography at: http://ufdc.ufl.edu/aerials/map. HEC-20 (Legasse et al., 2001) provides procedures for predicting and evaluating lateral channel migration through aerial photograph analysis in Chapter 6 of the document.

A special case of migration found in coastal zones is inlet migration. Inlets either migrate along the coast or remain fixed in one location. This is due to a complex interaction between the tidal prism (volume of water transported through the inlet during tides), open coast wave energy, and sediment supply. Although many of Florida's inlets are improved through jetty construction and bank stabilization, several inlets, particularly along the southwest coast, are not. New bridge construction and evaluation of existing structures over unimproved inlets should include a thorough investigation of the historical behavior of the inlet (through examination of historical aerial photographs and charts) to discern the migration trends to incorporate into the foundation design/evaluation as well as design/evaluation of the abutment protection. Types of inlet behavior can include:

- Updrift migration
- Downdrift migration
- Fluctuations in inlet width and depth
- Spit growth and breaching (resulting in oscillation of inlet location)

The analysis of coastal hydraulics for the design and evaluation of bridges over tidal inlets should be performed by a coastal engineer. References and aids in design/evaluation include the USACE's EM 1110-2-1810 Engineering and Design - Coastal Geology (1995) and EM 1110-2-1100 Coastal Engineering Manual (2006).

6.1.1.2 Aggradation/Degradation

Aggradation and degradation are related to the overall vertical stability of the bed. Long-term aggradation and degradation refers to the change in the bed elevation over time over an entire reach of the water body. Aggradation refers to the deposition of sediments eroded from the channel or watershed upstream of the bridge resulting in a gradual rise in bed elevation. Degradation refers to the gradual lowering of the bed elevation due to a deficit in sediment supply from upstream.

Given the potential influence of changes to the watershed to stability at a bridge location, the hydraulic engineer must not only evaluate the current stability of the stream and watershed, but also evaluate potential future changes in the river system (within reason). Examples of this include incorporation of watershed management plans or known planned projects (bridge/culvert replacements, dams, planned dredging, etc.) into evaluation of the vertical stability at the bridge location. As such, it is important that the engineer perform the necessary data collection (including contacting local agencies) to become aware of such projects/plans and incorporate them appropriately into the analysis.

For information on aggradation/degradation in riverine environments, refer to FHWA's HEC-18 and HEC-20. For more information refer to the U.S. Army Corps of Engineers' Coastal Engineering Manual (2006).

For existing bridge locations, by far, the most common evaluation of a channel's vertical stability is through examination of Bridge Inspection Reports. The reports (available upon request from the individual Districts) typically contain recent and historical inspection survey information. These surveys (typically lead-line surveys at each pier location on both sides of the bridge) are an excellent source of data on long-term aggradation or degradation trends. Additionally, inspection reports from bridges crossing streams in the same area or region can also provide information on the behavior of the overall waterway if information at a new location is unavailable. For new alignments, a review of historical aerial photographs is another method of channel stability analysis

Estimates of long-term vertical stability trends should be made over the lifetime (for new projects) or remaining lifetime (for evaluations of existing bridge or widening projects) of the subject bridge. If the result is degradation, the estimate at the end of the project life should be added to the total scour. If the result is aggradation, then documentation of the estimate should be made in the BHR. However, this estimate should not be included in the estimate of total scour. Rather, the current existing ground elevation should serve as the starting elevation for contraction and local scour.

As with channel migration, inlet stability is a special case of vertical stability. Examination of long-term trends through available historical information provides indicators of the inlet behavior over time. Additionally, inlet stability analyses can provide information on the evolutionary trends at the subject project. These analyses should be performed by a qualified coastal engineer. The references USACE's EM 1110-2-1810 Engineering and Design - Coastal Geology (1995) and EM 1110-2-1100 Coastal Engineering Manual (2006) provide additional resources.

6.1.2 Contraction Scour

Contraction scour occurs when a channel's cross section is reduced by natural or man-made features. Possible constrictions include the construction of long causeways to reduce bridge lengths (and costs), the placement of large (relative to the channel cross section) piers in the channel, abutment encroachment, and the presence of headlands (examples in Figure 6-1 and Figure 6-2). For design flow conditions that have long durations, such as those created by stormwater runoff in rivers and streams in relatively flat country, contraction scour can reach near equilibrium depths. Equilibrium conditions exist when the sediment leaving and entering a section of a stream are equal. Laursen's contraction scour prediction equations were developed for these conditions. A summary of Laursen's equations is presented below. For more information and discussion the reader is referred to HEC-18.

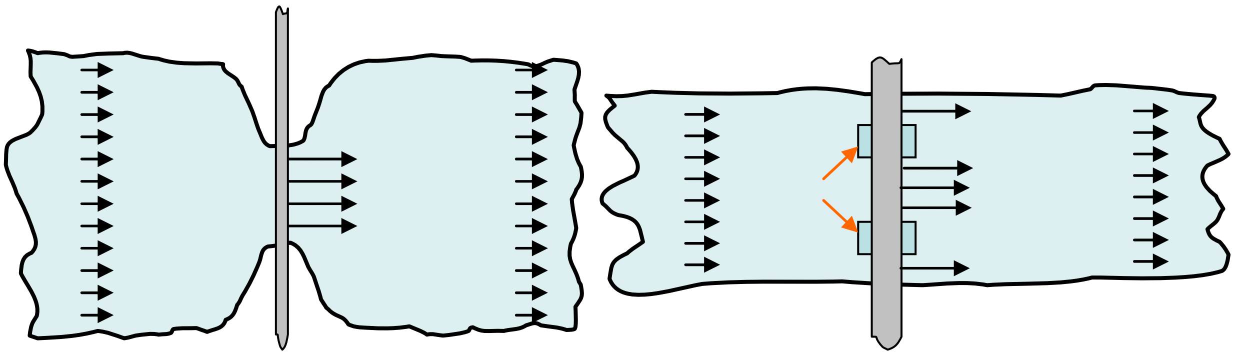


Figure 6-1 Examples of Contractions at Bridge Crossings

Figure 6-2 Example of Man-made Causeway Islands Creating a Channel Contraction

6.1.2.1 Steady, Uniform Flows

Laursen's contraction scour equations [Laursen (1960)], or rather a modified version of the equations recommended by HEC-18, were developed for steady uniform flow situations. This methodology provides the estimation of contraction scour for most bridge locations. However, predictions using these equations tend to be conservative, since the rate of erosion decreases significantly with increased contraction scour depth. Laursen developed different equations for clear-water and live-bed scour flow regimes. If the estimates of contraction scour via these equations are deemed too conservative (through application of engineering judgment), alternative analyses, including sediment transport modeling, may be pursued. In these situations, consult the District Drainage Engineer regarding the need to perform such an analysis.

A brief summary of the HEC-18 equations are presented herein. The reader is referred to HEC-18 for more information.

6.1.2.2 Live-Bed Contraction Scour Equation

The live-bed scour equation assumes that the upstream flow velocities are greater than the sediment critical velocity, V_c. The contraction scour in the section, y_s, is calculated from the equation below:

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{\frac{6}{7}} \left(\frac{W_1}{W_2}\right)^{K_1}$$

$$y_s = y_2 - y_o = \text{average contraction scour}$$

where:

- y_1 = Average depth in the upstream channel, ft (m)
- y_2 = Average depth in the contracted section after scour, ft (m)
- y_0 = Average depth in the contracted section before scour, ft (m)
- Q_1 = Discharge in the upstream channel transporting sediment, ft^3/s (m^3/s)
- Q_2 = Discharge in the contracted channel, ft^3/s (m^3/s)
- W_1 = Bottom width of the main upstream channel that is transporting bed material, ft (m)
- W_2 = Bottom width of the main channel in the contracted section less pier widths, ft (m)
- K_1 = Exponent listed in table below

Table 6-1 Determination of Exponent, K_1

V^*/ω	K_1	Mode of Bed material Transport
<0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
>2.0	0.69	Mostly suspended bed material discharge

where:

V_* = $(\tau_o/\rho)^{0.5}$, Shear velocity in the upstream section, ft/s (m/s)
ω = Fall velocity of bed material based on the D_{50}, ft/s (m/s) (Figure 6-3)
g = Acceleration of gravity, 32.17 ft/s^2 (9.81 m/s^2)
τ_o = Shear stress on the bed, lbf /ft^2 (Pa (N/m^2))
ρ = Density of water, 1.94 slugs/ft^3 (1000 kg/m^3)

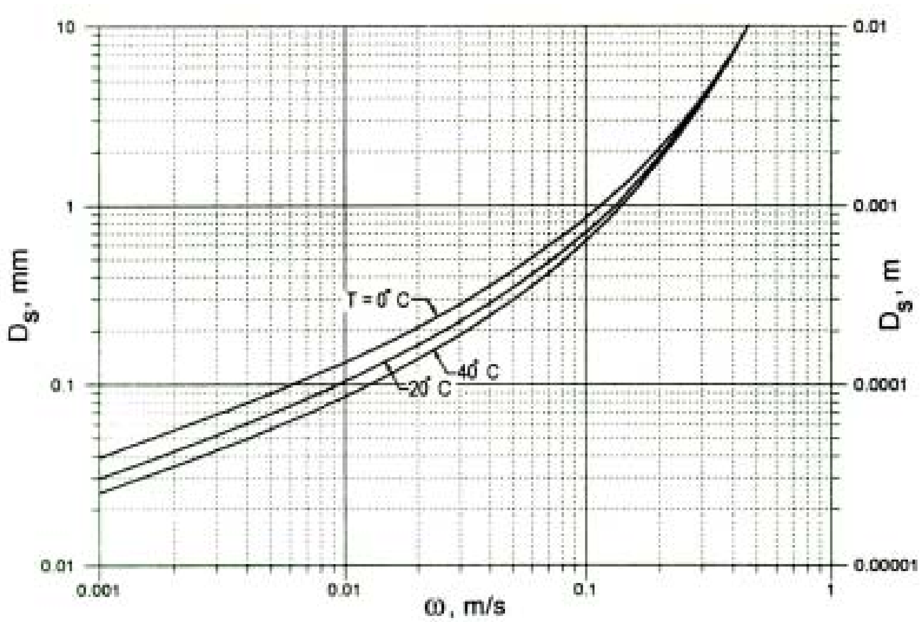


Figure 6-3 Fall Velocity of Sediment Particles with Diameter Ds and Specific Gravity of 2.65 (Source: HEC-18, 2001)

HEC-18 provides guidance for selecting upstream cross section location as well as the widths at the bridge and upstream cross sections. Notably, separate contraction scour calculations should be performed for the channel and left and right overbank areas (assuming they extend through the bridge). For cross sections that include multiple openings (including causeway bridges), upstream width selection involves delineating the flow patterns upstream of the bridge to properly identify the division of the flow from the upstream section to the bridges.

As stated previously, application of this methodology may result in overly conservative estimates. Section 6.1.2.4 discusses an alternative methodology for calculating contraction scour.

6.1.2.3 Clear-Water Contraction Scour Equation

The clear-water scour equation assumes that the upstream flow velocities are less than the sediment critical velocity. The contraction scour in the section, y_s, is calculated from the equation below:

$$y_2 = \left[\frac{K_u Q^2}{D_m^{\frac{2}{3}} W^2}\right]^{\frac{3}{7}}$$

$$y_s = y_2 - y_o = \text{average contraction scour}$$

where:

- y_2 = Average equilibrium depth in the contracted section after contraction scour, ft (m)
- Q = Discharge through the bridge or on the set-back overbank area at the bridge associated with the width W, ft^3/s (m^3/s)
- D_m = Diameter of the smallest non-transportable particle in the bed material (1.25 D_{50}) in the contracted section, ft (m)
- D_{50} = Median diameter of bed material, ft (m)
- W = Bottom width of the contracted section less pier widths, ft (m)
- y_o = Average existing depth in the contracted section, ft (m)
- K_u = 0.0077 (English units) or 0.025 (SI units)

For a more detailed discussion of these equations, the reader is referred to HEC-18.

As stated previously, application of this methodology may result in overly conservative estimates. Section 6.1.2.4 discusses an alternative methodology for calculating contraction scour.

6.1.2.4 Unsteady, Complex Flows

Application of Laursen's modified contraction scour equations at locations that experience design flows that are either unsteady or exhibit a complex flow field sometimes results in overly-conservative estimates of contraction scour. These situations include cases where: 1) the flow boundaries are complex, 2) the flows are unsteady (and/or reversing), and 3) the duration of the design flow event is short, etc. In these situations, an alternative to employing Laursen's modified equations is to perform two-dimensional flow and sediment transport modeling to estimate contraction scour depths (e.g., the USACE's RMA2 hydraulics model and SED2D sediment transport model). In these situations, consult the District Drainage Engineer regarding the need to perform sediment transport modeling.

6.1.3 Local (Pier and Abutment)

The main mechanisms of local scour are: (1) increased mean flow velocities and pressure gradients in the vicinity of the structure; (2) the creation of secondary flows in the form of vortices; and (3) the increased turbulence in the local flow field. Two kinds of vortices may occur: 1) wake vortices, downstream of the points of flow separation on the structure; and 2) horizontal vortices at the bed and free surface due to stagnation pressure variations along the face of the structure and flow separation at the edge of the scour hole.

Local scour is divided into two different scour regimes that depend on the flow and sediment conditions upstream of the structure. Clear-water scour refers to the local scour that takes place under the conditions where sediment is not in motion on a flat bed upstream of the structure. If sediment upstream of the structure is in motion, then the local scour is called live-bed scour.

For work in Florida, calculation of local pier scour must involve application of the Sheppard Pier Scour Equations detailed in the FDOT Bridge Scour Manual (Sheppard, 2010) rather than the CSU Pier Scour Equation when the total scour (long-term channel conditions, contraction scour, and pier scour) is greater than 5 feet. Additionally, the Florida Complex Pier Scour Procedure must be used in lieu of the complex pier scour procedure in HEC-18. The Florida Complex Pier Scour Procedure can be downloaded at: http://www.dot.state.fl.us/rddesign/Drainage/Bridge-Scour-Policy-Guidance.shtm

A brief overview of Sheppard's Pier Scour Equation and Florida Complex Pier Scour Procedure are presented below. Refer to the FDOT Bridge Scour Manual for detailed guidelines and examples.

Sheppard's Pier Scour Equations

Sheppard's Pier Scour Equations target three dimensionless hydraulic and sediment transport parameter groups to predict scour at simple piers. The equation is applicable to both riverine and tidal flows, applies to sediment sizes typical within the continental US, and gives good results for both narrow and wide piers. A detailed discussion is included in the FDOT Bridge Scour Manual: http://www.dot.state.fl.us/rddesign/Drainage/Bridgescour/FDOT-Scour-Manual-6-2-2005-Final.pdf). The pier scour equations are summarized below:

In the clear-water scour range ($0.4 \le \frac{V}{V_c} \le 1.0$)

$$\frac{y_s}{D^*} = 2.5 f_1 f_2 f_3$$

In the live-bed scour range ($1.0 < \frac{V}{V_c} \le \frac{V_{lp}}{V_c}$)

$$\frac{y_s}{D^*} = f_1\left[2.2\left(\frac{\frac{V}{V_c}-1}{\frac{V_{lp}}{V_c}-1}\right)+2.5\,f_3\left(\frac{\frac{V_{lp}}{V_c}-\frac{V}{V_c}}{\frac{V_{lp}}{V_c}-1}\right)\right],$$

and in the live-bed scour range above five ($\frac{V}{V_c} > \frac{V_{lp}}{V_c}$)

$$\frac{y_s}{D^*} = 2.2 f_1,$$

where:

$$f_1 \equiv tanh\left[\left(\frac{y_0}{D^*}\right)^{0.4}\right],$$

$$f_2 \equiv \left\{1-1.2\left[ln\left(\frac{V}{V_c}\right)\right]^2\right\},$$

$$f_3 \equiv \left[\frac{\left(\frac{D^*}{D_{50}}\right)}{0.4\left(\frac{D^*}{D_{50}}\right)^{1.2}+10.6\left(\frac{D^*}{D_{50}}\right)^{-0.13}}\right], \text{ and}$$

$$V_1 = 5V_c$$

$$V_2 = 0.6\sqrt{g\,y_0}$$

$$V_{lp} = \text{live bed peak velocity} = \begin{cases} V_1 \text{ for } V_1 > V_2 \\ V_2 \text{ for } V_2 > V_1 \end{cases}$$

where:

- y_s = Equilibrium scour depth, ft (m)
- D^* = Effective diameter of the pier, ft, (m)
- y_o = Water depth adjusted for general scour, aggradation/degradation, and contraction scour, ft (m)
- V = Mean depth-averaged velocity, ft/s (m/s)
- V_c = Critical depth-averaged velocity, ft/s (m/s)
- V_{lp} = Depth-averaged velocity at the live-bed peak scour depth, ft/s (m/s)
- D_{50} = Median sediment diameter, ft (m)

Methodology for determining depth-averaged critical velocity and depth-averaged live-bed peak velocity are found in the FDOT Bridge Scour Manual.

Florida Complex Pier Procedure

Most large bridge piers are complex in shape and consist of several clearly definable components. While these shapes are sensible and cost effective from a structural standpoint, they present a challenge for those responsible for estimating design sediment scour depths at these structures. The Complex Pier Methodology applies to any bridge piers different from a single circular pile. They can be composed of up to three components referred to here as the column, pile cap, and pile group as shown below in Figure 6-4.

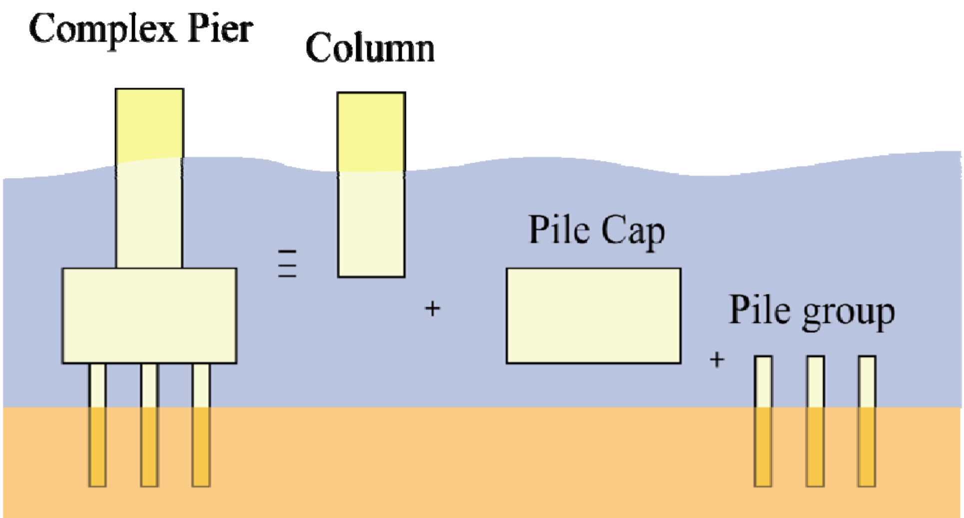


Figure 6-4 Complex Pier Components

The methodology is based on the assumption that a complex pier can be represented (for the purposes of scour depth estimation) by a single circular pile with an “effective diameter” denoted by D^*. The magnitude of the effective diameter is such that the scour depth at this circular pile is the same as that at the complex pier for the same sediment and flow conditions. The problem of computing equilibrium scour depth at the complex pier is therefore reduced to one of determining the value of D^* for that pier and applying Sheppard’s Pier Scour Equation to the circular pile for the sediment and flow conditions of interest. The methodology determines the total D^* for the complex structure can be approximated by the sum of the effective diameters of the components making up the structure, that is:

$$D^* = D^*_{col} + D^*_{pc} + D^*_{pg}$$

where:

D^* = Effective diameter of the complex structure
D^*_{col} = Effective diameter of the column
D^*_{pc} = Effective diameter of the pile cap
D^*_{pg} = Effective diameter of the pile group

The procedure for computing local scour depth for complex piers is further divided into three cases as illustrated in the Figure 6-5 below:

- Case 1 complex pier with pile cap above the sediment bed.
- Case 2 complex pier with pile cap partially buried.
- Case 3 complex pier with pile cap completely buried.

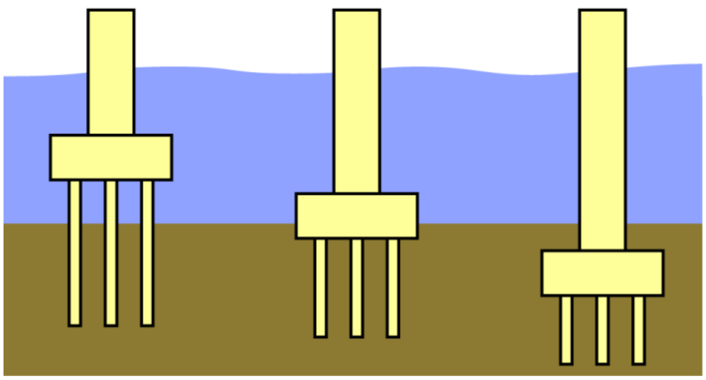


Figure 6-5 Three Cases of Local Scour Depth for Complex Pier Computations

Refer to the FDOT Bridge Scour Manual for a more detailed discussion on the procedure and the application of the equations. Additionally, an Excel spreadsheet for calculating scour at single piles and complex piers is available from the FDOT website via the following link: http://www.dot.state.fl.us/rddesign/Drainage/Florida-Scour-Manual-Training-Course.shtm

HEC-18 also provides equations for calculating local scour at abutments. However, as stated in the FDOT Drainage Manual, abutment scour estimates are not required when the minimum abutment protection is provided. Where you have significantly wide flood plains with high velocity flow around abutment consider analyzing abutment spatial requirements using HEC-23.

6.1.4 Scour Considerations for Waves

Waves are an important factor that must be addressed in the design of bridges exposed to long fetches. This is particularly true at bridge abutments and approach roadways. Figure 6-6 displays an example of the damage waves can cause during a hurricane event. The photograph shows the east approach to the I-10 Westbound Bridge over Escambia Bay following Hurricane Ivan. During the storm, waves breaking on the shoreline removed the undersized protection and eroded the fill at the approach slab, eventually undermining it. Proper design of abutment protection to withstand wave impact will be discussed in Section 6.4.

Many bridges in coastal environments incorporate seawalls into the design of abutment protection. Scour at vertical walls occurs when waves either break on or near the wall or reflect off the wall thus increasing the shear stress at the bottom of the wall. This is known as toe scour. Toe scour decreases the effective embedment of the wall and can threaten the stability of the structure. Current USACE guidance (CEM, 2001) indicates that, as a rule of thumb, the depth of scour experienced in front of a vertical wall structure is on the same order of magnitude as the incident maximum wave height. Methodologies for designing toe scour protection are presented in Section 6.4.

Figure 6-6 East Approach to the I-10 WB Bridge over Escambia Bay Following Hurricane Ivan (2004)

Regarding the impacts of waves on scour at bridge piers, laboratory modeling indicates that vertical piles subject to both waves and currents experience an increase in the effective shear stress at the bed. Additionally, there is an increase in the amount of suspended sediment and thus the sediment transport in the vicinity of the pile as compared with the transport associated with currents or waves alone. No current analytical methods are available for design purposes. However, some sediment transport models (e.g., SED2D) include methodologies for calculating the shear stress due to combined waves and currents.

6.2 Scour Considerations for Ship Impact

Piers designed to resist ship impact include in their load combinations estimates of "long-term scour." This long-term scour is different from the long-term channel conditions discussed in the previous section which referred to the lateral or vertical long-term processes that occur at a bridge crossing over the lifetime of the bridge. Rather the scour incorporated into design for ship impact is the scour that may be present at a pier when the impact occurs. For sites where everyday (normal daily) flows are in the clear-water regime – i.e., below the critical value for incipient motion of the bed sediments – this scour is the total 100-year scour for the structure. The reasoning is that if a design event occurs during the lifetime of the bridge, the daily flows are not sufficient to fill in the hole. For bridges where flows are in the live-bed regime, the "long-term scour" is the normal, everyday scour at the piers combined with the degradation and channel migration anticipated during the life of the structure. The reasoning here is that if the structure experiences a design event, the flows are sufficient to refill the scour hole following such an event.

For bridge replacements, parallel bridges, major widenings, etc., Bridge Inspection Reports and the design survey should be the primary basis for determining normal everyday scour. If the proposed piers are the same as the existing, the normal, everyday scour elevation should be that which is reflected in the inspection reports and the design survey (Figures 6-7 and 6-8). Slight differences in scour will likely exist between inspection reports and between the reports and the design survey. In these cases, an average scour elevation will be a reasonable estimate of normal, everyday scour. If there is a large difference, an extreme storm event may have occurred just before the inspection or survey. This should be investigated and addressed on a case-by-case basis.

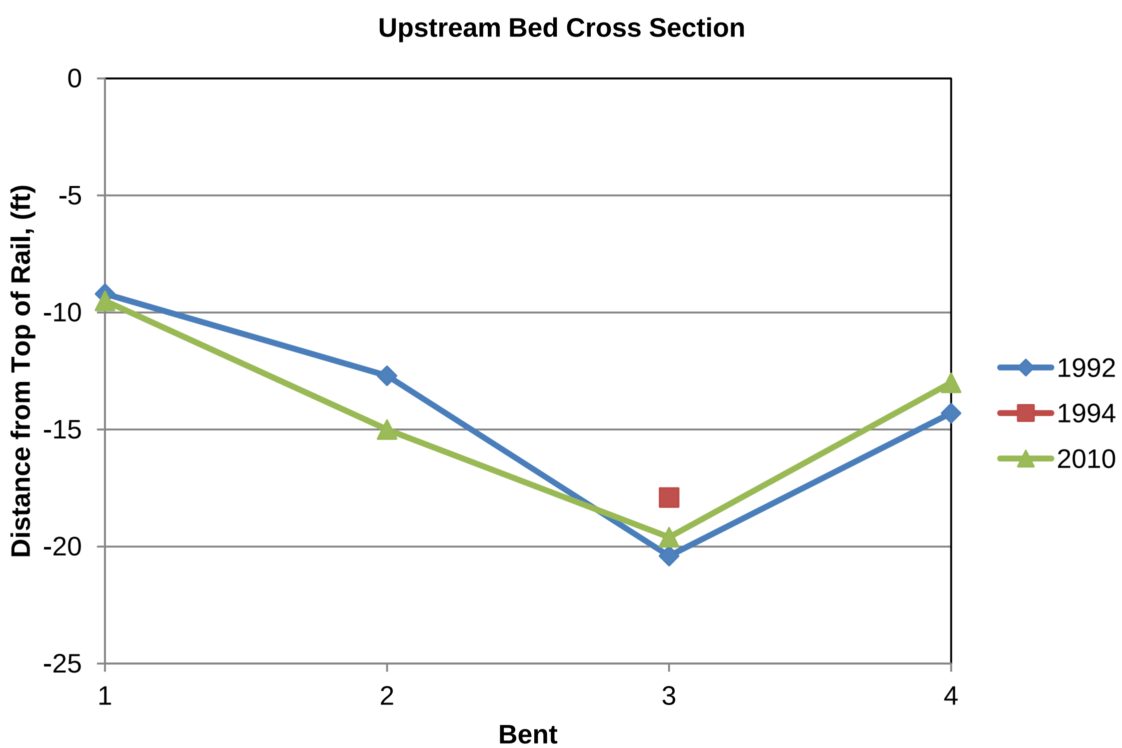


Figure 6-7 Example of Normal, Everyday Scour Holes from Bridge Inspection Data

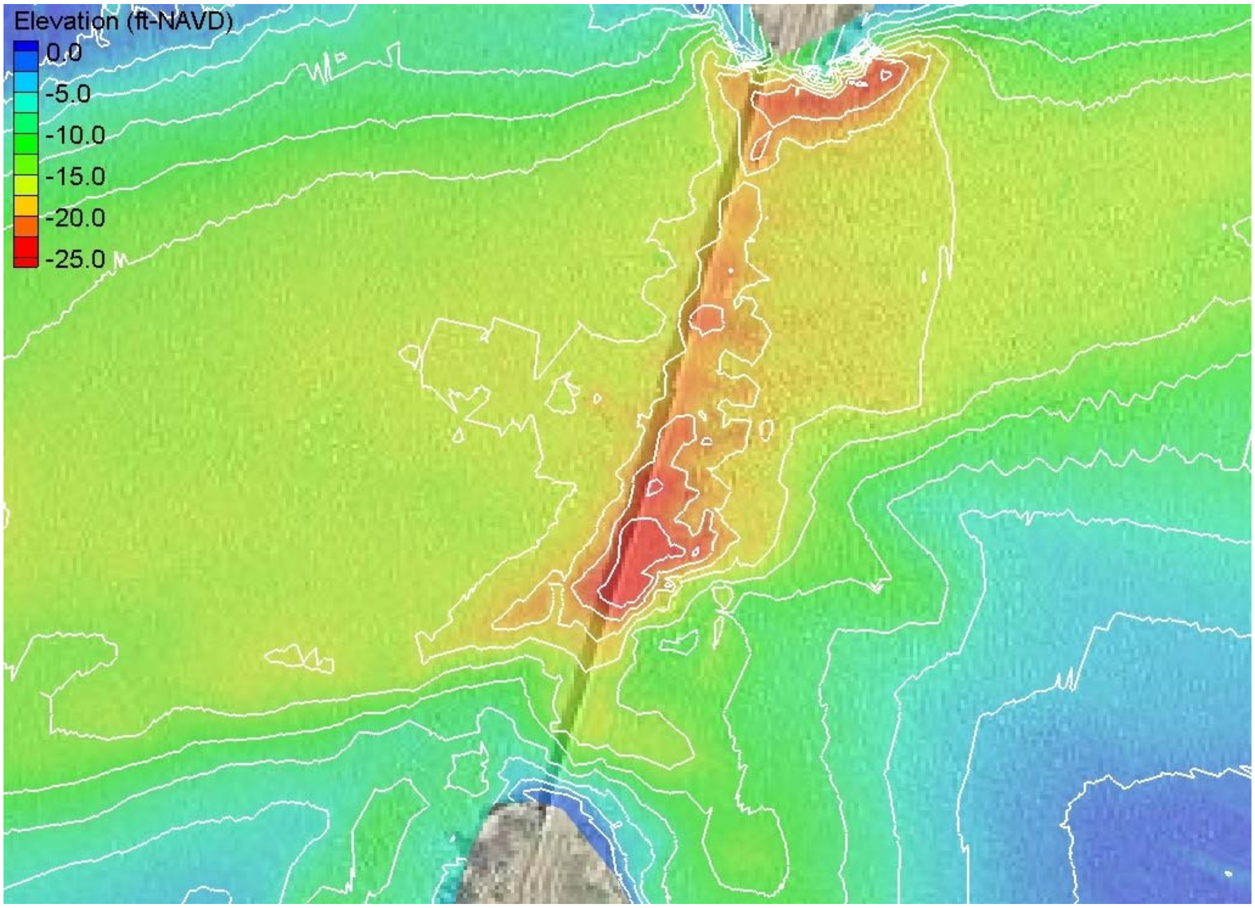


Figure 6-8 Example of Normal, Everyday Scour Holes from Survey Data

For structures in which the proposed piers will be a different size or shape than the existing or for new bridges/new alignments where there are no historical records available, estimates of the normal everyday scour should be based on hydraulic modeling results of expected daily flows. For riverine bridges, this should correspond to flows equal to the normal high water. For tidal flows, everyday flows correspond to the maximum flows experienced during spring tides.

6.3 Florida Rock/Clay Scour Procedure

The Florida Rock/Clay Scour Procedure was developed to address the scour resistance of cemented strata, rock, and clay. The procedure was originally developed to address scour in cohesive bed materials that are considered "scourable" according to the FHWA guidelines set forth by the FHWA in the memorandum located via the following link: http://www.fhwa.dot.gov/engineering/hydraulics/policymemo/rscour.cfm. Initiation of the Rock/Clay Scour Procedure should only occur following consultation with the District Drainage Engineer and the District Geotechnical Engineer.

The test methods establish the shear stress response of soils and the procedure integrates that response over the lifetime of expected flows at the bridge site. The procedure involves establishing the shear stress response of a site-specific sample using the RETA (Rotating Erosion Test Apparatus) and SERF (Sediment Erosion Recirculating Flume) devices, shown below in Figures 6-9 and 6-10 respectively, and then integrating that response over the flows expected in the life of the bridge to predict contraction or local scour at the bridge.

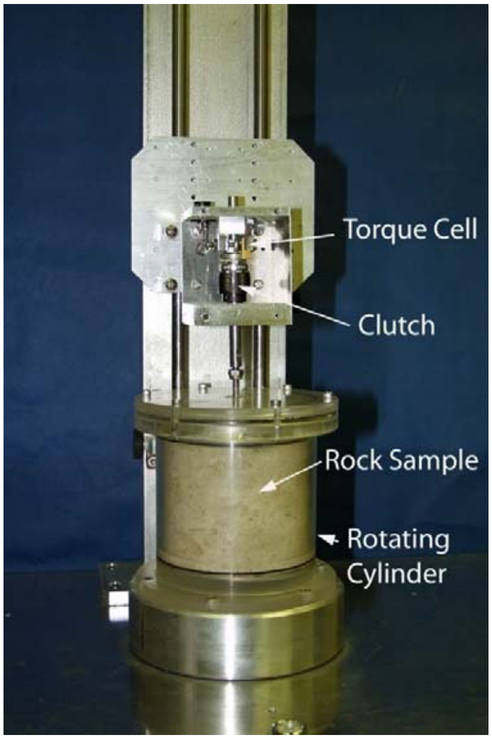


Figure 6-9 Rotating Erosion Test Apparatus (RETA, above)

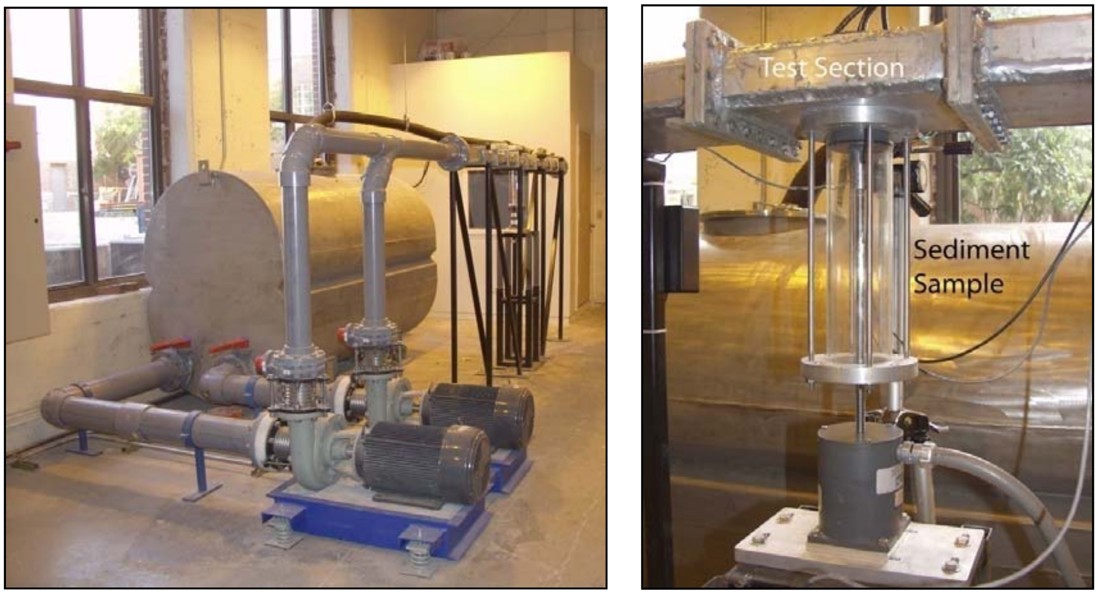


Figure 6-10 Sediment Erosion Recirculating Flume (SERF)

The procedure includes an appropriate amount of conservatism by incorporating the following assumptions: 1) the shear stress does not decrease within a local scour hole; 2) the bridge experiences an extremely aggressive bridge flow history over the bridge lifetime; 3) there is no refill of the predicted scour; and 4) only the more conservative of the RETA and SERF results of all cores tested for a particular bridge characterize the erosion properties of the bed. Districts should contact the State Drainage Engineer if scour-resistant soils are expected to be encountered in bridge design or the evaluation of existing bridge scour. The following link contains the FDOT Bridge Rock Scour Analysis Protocol and describes initiation of the process: http://www.dot.state.fl.us/rddesign/Drainage/Bridgescour/Bridge-Rock-Scour-Analysis-Protocol-Jan2008.pdf

6.3.1 Pressure Scour
See HEC-18

6.3.2 Debris Scour
See HEC-18

6.4 Scour Countermeasures
Scour countermeasures are defined as a measure intended to prevent, delay or reduce the severity of scour problems. For this discussion, they address the class of armoring countermeasures (as defined by HEC-23, Legasse, et al., 2009) to resist the erosive forces caused by a hydraulic condition. This section addresses scour countermeasures at both abutments and interior bents.

6.4.1 Abutment Protection

Proper bridge design includes abutment protection to resist the hydrodynamic forces experienced during design events. The FDOT Drainage Manual specifies the following minimum protection requirements:

Spill-through abutments – Where flow velocities do not exceed 9 fps, and/or wave heights do not exceed 3 feet, minimum protection shall consist of one of the following placed on a 1V:2H or gentler slope:

- Rubble riprap (Bank and Shore), bedding stone, and filter fabric — Rubble riprap (Bank and Shore) is defined in the FDOT Standard Specifications for Road and Bridge Construction Section 530.
- Articulated concrete block (cabled and anchored).
- Grout-filled mattress (articulating with cabling throughout the mattress).

Site specific designs and technical specifications are required when using articulated concrete block or grout-filled mattress abutment protection. The FDOT Structures Detailing Manual provides typical details for standard revetment protection of abutments and extent of coverage. The horizontal limits of protection shall be determined using HEC-23. A minimum distance of 10 feet shall be provided if HEC-23 calculations show less than 10 feet. Notably, neither grouted sand-cement bag abutment protection nor slope paving is considered adequate protection for bridges spanning waterways. Slope paving can develop cracks or upheaved slabs where loss of fill can occur. Grouted sand-cement bags often fail when cracks form around the individual bags and sediment is lost through cracks or displaced elements (Figure 6-11). Additionally, these systems are prone to failure due to undermining (erosion at the toe of the protection) or flanking (erosion at the edges of the protection) when the edges of the protection are not sufficiently buried.

Figure 6-11 Damage to Sand-Cement Grouted Riprap Abutment Protection

The horizontal and vertical extents, regardless of protection type, should be determined via the design guidelines contained in HEC-23. A minimum of 10 feet is recommended as a horizontal extent if the results from the HEC-23 calculations show that a horizontal extent less than 10 feet is acceptable. The drainage engineer should review the limits of right-of-way to ensure the minimum apron width at the toe of the abutment slope both beneath and around the bridge abutments along the entire length of the protection. If calculations from HEC-23 result in a horizontal extent outside the right-of-limits, the drainage engineer should do the following:

a. Recommend additional right-of-way.
b. Provide an apron at the toe of abutment slope which extends an equal distance out around the entire length of the abutment toe. In doing so, the drainage engineer should consider specifying a greater rubble riprap thickness to account for reduced horizontal extent (Figure 6-12).

Additional considerations regarding extents must be made in coastal areas subject to wave attack. Prolonged exposure to hurricane generated waves on unprotected approaches may lead to damage to the approach slabs (Figure 6-6) as well as the approach roadways. Consideration should be given to extending the limits of protection to include the approach spans in wave vulnerable areas.

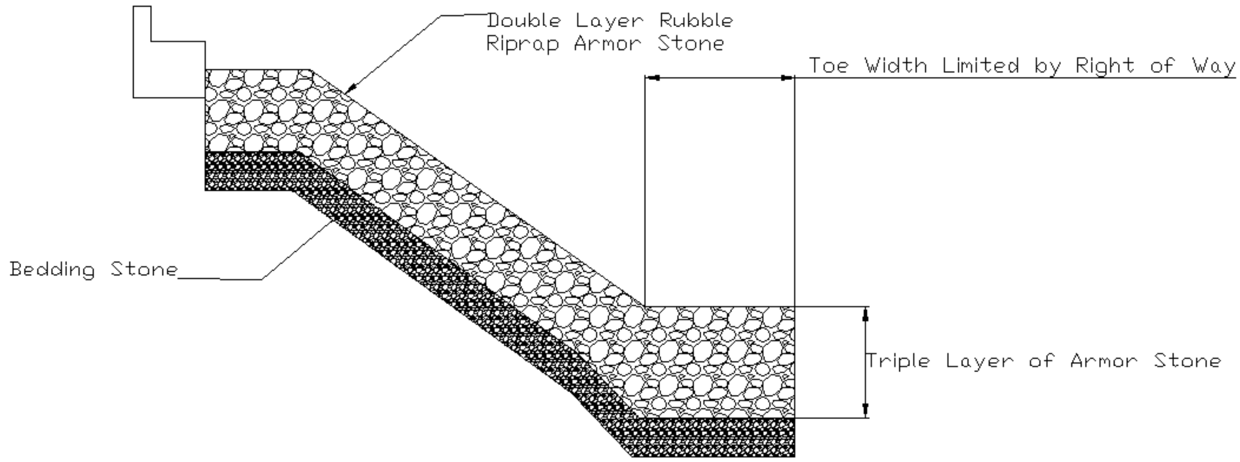


Figure 6-12 Example of Increased Toe Thickness to Offset Decrease in Toe Width

When bridges are to be widened, the drainage engineer may not be able to simply recommend the use of standard rubble riprap as defined in Section 4.9 of the Drainage Manual. Constructability issues may arise at existing bridges where the low chord elevations may prevent uniform riprap placement due to height constrictions. If this case arises, the drainage engineer can do the following:

a. Size the rubble according to the design average velocities determined at the abutment using HEC-23 rather than employ the minimum FDOT Bank and Shore Rubble Riprap. This may result in smaller armor stone sizes thus enabling easier placement.
b. Provide an alternate material in the plans. The material should be approved prior to installation.

<u>Bulkhead/vertical wall abutments</u>: Abutments must be protected by sheet piling with rubble toe protection below the bulkhead, and with revetment protection above the bulkhead when appropriate. The size and extent of the protection shall be designed for the individual site conditions.

Abutment protection should extend beyond the bridge along embankments that may be vulnerable during a hurricane surge. Wave attack above the peak design surge elevation and wave-induced toe scour at the foot of bulkheads must be considered. In such cases, a qualified coastal engineer should be consulted to determine the size and coverage of the toe scour protection. The choice of cabling material for interlocking block or concrete mattresses must consider the corrosiveness of the waterway. Steel cabling should not be used in salt or brackish waters (stainless steel is permissible).

Rubble riprap abutment protection is the preferred protection type for new bridges. Rubble riprap has several advantages (HEC-11) including:

- The riprap blanket is flexible and is not impaired or weakened by minor movement of the bank caused by settlement or other minor adjustments.
- Local damage or loss can be repaired by placement of more rock.
- Construction is not complicated.
- Vegetation will often grow through the rocks, adding esthetic and structural value to the bank material and restoring natural roughness.
- Riprap is recoverable and may be stockpiled for future use.

A drawback to rubble riprap is that it can be more sensitive than some other bank-protection schemes to local economic factors. For example, transport costs can significantly affect the construction costs.

A minimum rubble riprap abutment protection design (for an illustration of bridge abutment slope protection adjacent to streams refer to the FDOT Structures Detailing Manual at the following link: http://www.dot.state.fl.us/structures/structuresmanual/currentrelease/vol2_sdm/slopeprotectiondetails.pdf) where velocities do not exceed 9 fps and waves do not exceed 3 feet on a 1V:2H slope should consist of a 2.5-foot thick armor layer comprised of FDOT Standard Bank and Shore Rubble Riprap over a 1-foot thick layer of bedding stone over filter fabric. The filter fabric should be sized appropriately to prevent loss of the fill sediments. The purpose of the bedding stone is to ensure consistent contact between the filter fabric and the soil; and prevent the armor stone from damaging the filter fabric during construction or movement during design events. The riprap should have a well graded distribution to ensure inter-locking between the individual units which improves performance of the protection. For riverine applications, the drainage engineer should compare these minimums to the guidance presented in HEC-23 (Design Guideline No. 14) to ensure proper design. A notable feature of the slope protection cross-sections, illustrated in the FDOT Structures Detailing Manual's link above, is the sand cement bags located between the revetment and the abutment. This detail was added to the Standard following field inspection observations that the protection/abutment interface was often a point of failure. Shifting of the stones during a minor event would cause a gap to open at the top of the slope where fill would be eroded. This addition ensures that the filter fabric remains in contact with the abutment so that any settlement will not produce a gap between the structure and the stones.

For locations subject to wave impacts with wave heights greater than 3 feet, the engineer must also design the revetment to resist hurricane generated waves. Design of abutment protection should follow the same procedures and methodologies as design of rubble riprap protection that serves as shore protection. The U.S. Army Corps of Engineers provides guidance in the references USACE (2006) and USACE (1995). USACE Engineering Manual 1110-2-1614 (USACE 1995), in particular, provides multiple methodologies for properly sizing armor stone as well as designing the revetment extents, toe geometry, bedding stone, and armor layer distribution.

Often, this analysis will result in an armor stone size greater than that provided by the FDOT Standard Bank and Shore Rubble Riprap. When this occurs, the more conservative (larger stone size) design should be employed. For these designs, a modified special provision must be developed for the non-standard rubble riprap. The provision must specify the new riprap distribution developed employing the techniques located in USACE (1995) or a similar procedure. The engineer should develop a well graded distribution to the armor stone to ensure optimal performance. Additionally, for large armor stone, it may become necessary to include additional intermediate stone layers into the design to prevent loss of bedding stone between gaps in the armor stone. The reference USACE (1995) presents guidelines for design of granular filter layers as a function of the armor stone size.

For toe scour protection, the reference USACE (1995) provides guidance on sizing stones and designing the apron width. Toe apron width will depend on both geotechnical and hydraulic factors. The passive earth pressure zone must be protected for a sheet-pile wall. The minimum width from a hydraulic perspective should be at least twice the incident wave height for sheet-pile walls and equal to the incident wave height for gravity walls. Additionally, the apron should be at least 40% of the depth at the structure. This apron width should be compared to that required by geotechnical factors and adjusted appropriately. Regarding sizing of the armor stone, the reference provides a method developed by Brebner and Donnelly. USACE (2006) also provides guidance for toe scour protection in front of vertical wall structures in Section VI-5-6 of the Coastal Engineering Manual.

For revetment installations where wave attack is not expected to be significant, include all options (e.g., fabric-formed concrete, standard rubble, or cabled interlocking block, etc.), which are appropriate based on site conditions (examples in Figure 6-13 through Figure 6-15). HEC-23 provides guidance for design of these protection systems in the following design guidelines:

- Design Guideline 8 – Articulating Concrete Block Systems
- Design Guideline 9 – Grout-Filled Mattresses
- Design Guideline 14 – Rock Riprap at Bridge Abutments

Figure 6-13 Example of Rubble Riprap Abutment Protection

Figure 6-14 Example of Articulating Concrete Block Abutment Protection

Figure 6-15 Example of Grout Filled Mattress Abutment Protection

Selected options shown to be appropriate for the site should be documented in the BHR. A Technical Specification may be written based on the use of the most desirable revetment material, with the option to substitute the other allowable materials at no additional expense to the Department. This recommendation would help in eliminating revetment CSIP's (Cost Savings Initiative Proposals) during construction. No matter what options are allowed, the bedding (filter fabric and bedding stone) should be matched to the abutment material. Some of the options are not self-healing (i.e. not rubble riprap), and a major failure can occur if loss of the embankment material beneath the protection takes place.

As a final note, coastal bridges often incorporate seawalls into the abutment protection design. The caps of these structures often have a low elevation (below the design surge elevation) to tie into neighboring structures. Design of these structures should be addressed as containing elements of both spill-through and vertical wall abutments. The area in front of the seawall should include a toe scour apron designed in the same manner as for vertical wall abutments. Areas between the seawall and the abutment should be designed employing the same procedures as spill-through abutments. These designs should ensure encapsulation of the fill behind the seawall (Figure 6-16) to prevent loss of fill and potential failure of the anchoring system (Figure 6-17).

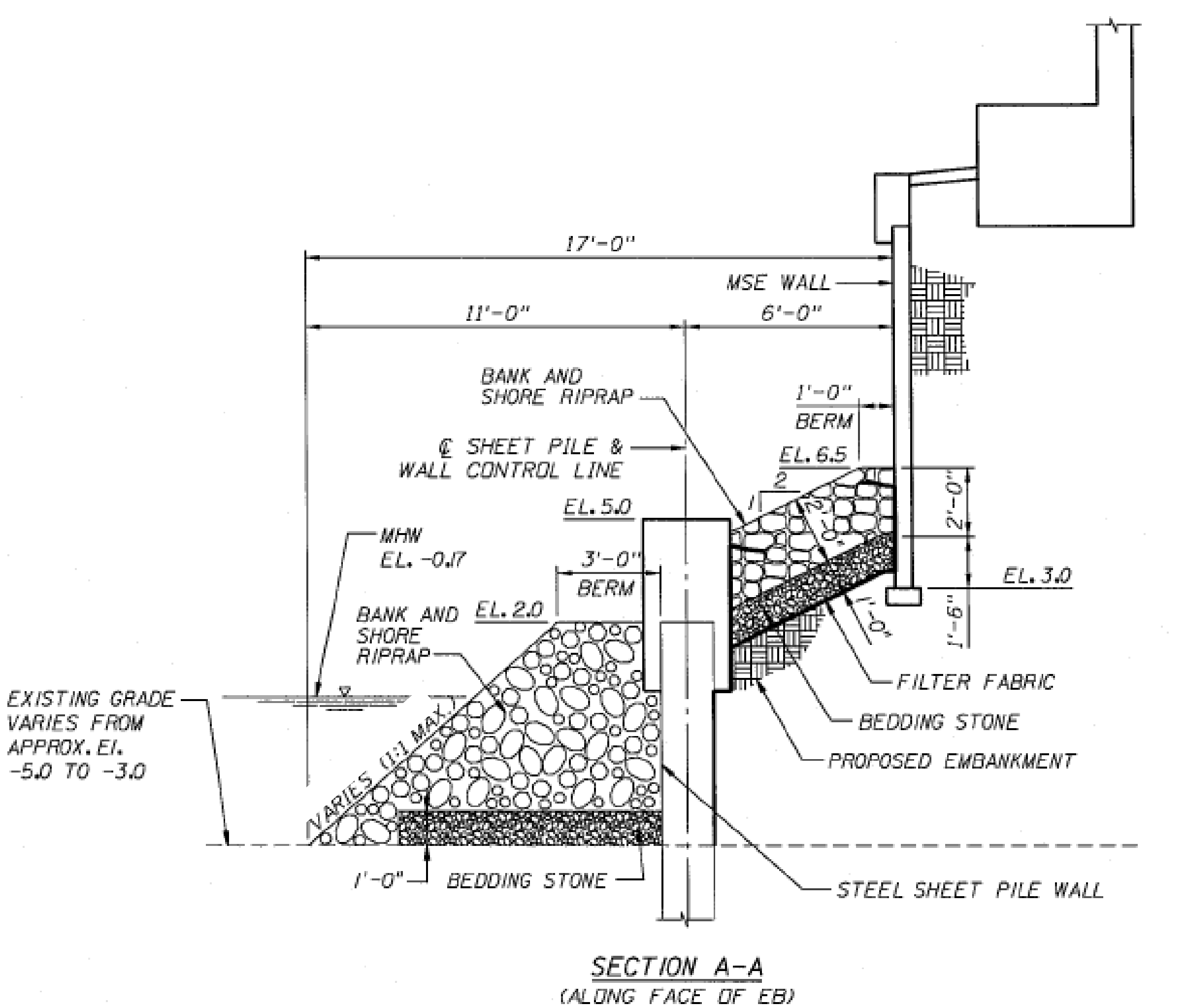


Figure 6-16 Example of Abutment Protection Design Including a Seawall

Figure 6-17 Seawall Failure Following Hurricane Frances (2004)

6.4.2 Scour Protection at Existing Piers

For bridges evaluated as scour critical and where monitoring is not an option, one of the countermeasures that should be considered is a bed armoring countermeasure around the critical pier. As with abutment protection, pier scour protection can take many forms. Examples of these include rubble riprap, articulating concrete block (ACB), grout filled mattress, gabion/marine mattress, and partially grouted riprap. HEC-23 provides design guidance for these protection systems in the following design guidelines (located in Volume 2 of the reference):

- Design Guideline 8 – Articulating Concrete Block Systems at Bridge Piers
- Design Guideline 9 – Grout-Filled Mattresses at Bridge Piers
- Design Guideline 10 – Gabion Mattresses at Bridge Piers
- Design Guideline 11 – Rock Riprap at Bridge Piers
- Design Guideline 12 – Partially Grouted Riprap at Bridge Piers

The guidelines provide procedures for selecting factors of safety, methodologies for sizing the material, and recommendations for designing coverage extents, filter requirements and installation guidelines. Several similarities between the procedures can be drawn. All guidelines recommend ensuring that the top of the protection remain level with the bed of the approach. Suggestions for achieving this include placement of sand-filled geotextile containers within the scour hole to raise the bed elevation and serve as a filter for the overlaying protection. The guidelines all also recommend that the horizontal extent of the protection extend a distance equal to twice the effective diameter of the pier in all directions. For the non-riprap options, the guidelines recommend that the protection slope away from the pier with the edges of the protection buried below the maximum scour depth for the overall cross section (i.e., depth of contraction scour and long-term degradation). A common failure point of the non-riprap protection schemes is at the edges of the protection if the mattress becomes undermined. Thus, it is important to incorporate trenching of the edges and anchoring systems (if appropriate) into the protection design. Another common failure point is at the pier/protection interface. The guidelines suggest grouting this interface to prevent loss of fill for both the ACB and gabion protection systems. Disadvantages and advantages of each system, including construction feasibility and cost, should reviewed by the drainage engineer.

Chapter 7

Deck Drainage

Three options, in order of preference, to drain the deck of a bridge are:

1. Rely on the longitudinal grade of the bridge to convey the deck runoff to the end of the bridge.
2. Use free discharging scuppers or inlets to drain the deck runoff to the area directly below the bridge. These are sometimes referred to as open systems.
3. Collect the discharge from the scuppers or inlets in a pipe system. The pipe system can discharge down a pier or at the ends of the bridge. These systems are sometimes referred to as closed systems.

Spread criteria will control the need to eliminate option 1 and use either option 2 or 3. The inability to discharge to the area below the bridge will control the need to eliminate option 2 and use option 3.

7.1 Bridge End Drainage

If the profile grade of the roadway is sloping off of the bridge, runoff from the bridge is collected by roadway inlets, often immediately beyond the bridge approach slab. Inlets are typically not placed in the approach slab so that runoff does not seep between the concrete approach slab and the roadway inlet. If spread issues mandate an inlet to be placed in the approach slab, obtain concurrence from the District Drainage Engineer and coordinate with the District Structures Design Engineer.

For rural roadways, shoulder gutter is typically used to convey the bridge flow to a shoulder gutter inlet. This inlet is usually located at least 30 feet from the end of the approach slab to allow for the 25 foot transition from the barrier wall to the shoulder gutter shape (FDOT Standard Index 400) and the 5 foot transition from the shoulder gutter into the shoulder gutter inlet (Index 220). The spread at the shoulder gutter inlet should also be checked for the 10-year flow to ensure that runoff does not overtop the shoulder, causing erosion of the embankment (refer to the FDOT Storm Drain Handbook for more information).

If the profile grade is sloping onto the bridge for rural roadways, then the calculations for the deck drainage may need to include roadway runoff flowing onto the bridge. The shoulder gutter transition directs the rainwater from the bridge into the inlet (refer to Figure 7-1). For standard cross slopes of 0.02 for bridge shoulders and 0.06 for roadway shoulders, with a 10 foot wide shoulder, the longitudinal slope of the gutter due to the transition is 2.1%. For this situation, the roadway grade would need to be greater than 2.1% for roadway runoff to flow onto the bridge. Appendix C shows how this slope was determined, and the same method can be used to calculate the slope for other situations.

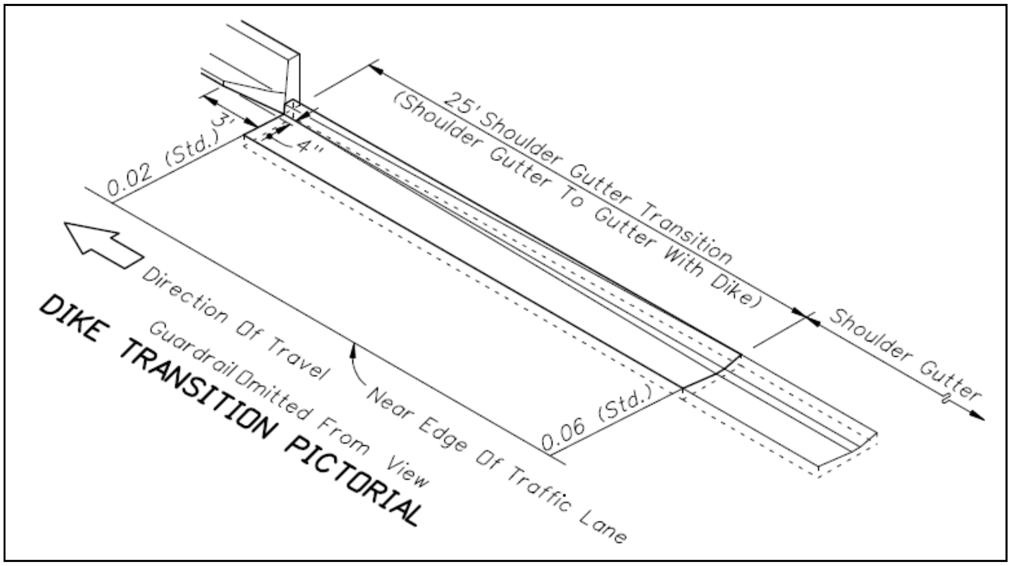


Figure 7-1 Shoulder Gutter Transition at Bridge End

For urban locations, if there is not a barrier wall between the sidewalk and the travel lanes, or if there is no sidewalk, a curb inlet can be placed at the end of the approach slab.

The Drainage Manual does not require bridge sidewalk runoff to be collected on the bridge. Scuppers or drains are not necessary to control the runoff on the bridge sidewalk unless the runoff becomes great enough to overwhelm the collection system at the end of the bridge. Scuppers used to drain the sidewalk must be ADA compliant.

In handling runoff from the sidewalk at the end of the bridge, the best option is to transition the sidewalk slope towards the roadway immediately downstream of the bridge. The flow can then be picked up in the first curb inlet or barrier wall inlet off of the bridge.

7.2 No Scuppers or Inlets (Option 1)

If possible, stormwater should be allowed to flow to the end of the bridge and collected in the roadway drainage system. To determine if this option can be used, the spread should be checked:

- Where the barrier wall or curb ends at the edge of the approach slab.
- At the first inlet off of the bridge.

Spread is calculated based on the Gutter Flow Equation in Section 3.2 of the FDOT Storm Drain Handbook. Spread criteria are given in Chapter 3.9 of the FDOT Drainage Manual. If the spread exceeds the allowable spread criteria, then scuppers or inlets may be needed on the bridge to reduce the spread.

If the spread exceeds criteria, consider adjusting the profile grade to reduce the spread before adding scuppers or inlets on the bridge. Spread will be reduced by:

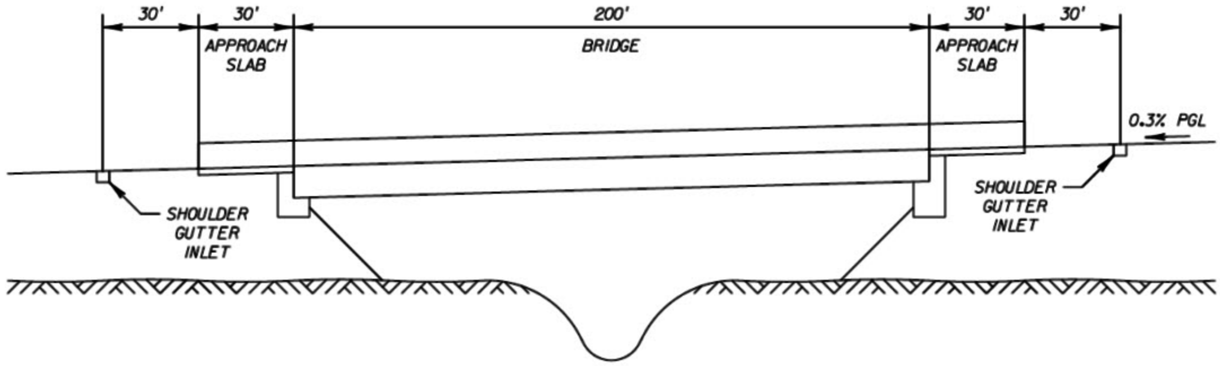
- Steepening the longitudinal slope of the bridge at the bridge ends.
- Including a profile crest in the middle of the bridge rather than using a profile that slopes to only one end of the bridge.

After determining grades that would eliminate the need for scuppers or inlets, talk with the roadway designer to determine the feasibility of adjusting the profile grade.

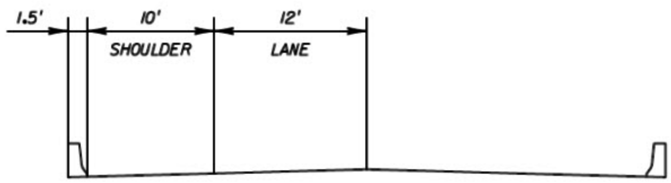
Example 7-1

A bridge for a two lane rural roadway has the following characteristics:

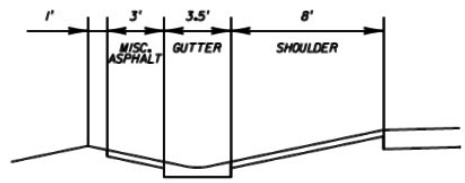
- 200’ length
- 30’ approach slabs
- A longitudinal slope of 0.3%
- Shoulder gutter inlets located 30’ from the uphill approach slab
- The bridge typical section has two 12’ travel lanes, 10’ outside shoulders, 1.5’ wide barriers, 0.02 cross slopes, and is crowned in the middle.



BRIDGE PROFILE



BRIDGE TYPICAL SECTION



SHOULDER GUTTER SECTION

<u>Solution:</u>
Determine the drainage area to the end of the downhill approach slab. On the uphill end of the bridge, the shoulder gutter transition will cause the runoff from the area between the shoulder gutter inlet and the end of the approach slab to flow back to the shoulder gutter inlet. Therefore, the drainage area contributing to the downhill side will include the bridge deck and the approach slabs:

Area = (12+10+1.5) (30+200+30) / 43560 = 0.14 acres

Conversion from square ft. to acres
Approach slab length
Bridge Length
Approach slab length
Width of barrier wall
Width of shoulder
Width of travel lane

The flow is:

Q = CiA = 0.95 (4) (0.14) = 0.53 cfs

where:
C = Rational runoff coefficient

i = Rainfall intensity, inches per hour
(4 in/hr, refer to FDOT Storm Drain Handbook for explanation)
A = Drainage area, acres

Solving the gutter flow equation for spread:

$$Spread = \left[\frac{Qn}{0.56 S_X^{5/3} S^{1/2}}\right]^{3/8} = \left[\frac{(0.53)(0.016)}{0.56(0.02)^{5/3}(0.003)^{1/2}}\right]^{3/8} = 7.1\,ft.$$

The spread at the end of the downhill approach slab is less than 10 feet, the width of the shoulder, therefore scuppers are not necessary.

The spread at the shoulder gutter inlet on the downhill side of the bridge should also be checked. There will be an additional drainage area from the end of the approach slab that needs to be added to the drainage on the bridge. The drainage area to the shoulder gutter inlet is:

Area = 0.14 + (((12+8+3.5+4) (30)) / 43560) = 0.16 acres

Conversion from square ft. to acres
Distance to inlet
Width behind shoulder gutter
Shoulder gutter
Width of shoulder
Width of travel lane
Drainage area from bridge

Assuming that the bridge is in Zone 1 for the IDF curves, the flow to the inlet is:

Q = CiA = 0.95 (7.0) (0.16) = 1.06 cfs

where:
i = The 10-year, 10 minute rainfall intensity = 7.0 inches per hour
(Refer to FDOT Storm Drain Handbook for explanation)

Note that this value is slightly conservative. The 1 foot unpaved strip behind the guardrail was assumed to be paved in this calculation.

The allowable conveyance in the shoulder gutter is K = 28 cfs. Refer to the FDOT Storm Drain Handbook for further explanation of this value. The allowable flow at the shoulder gutter inlet is:

$Q = K\ S^{1/2} = (28)\ (0.003)^{1/2} = 1.53$ cfs

Since the gutter flow just uphill of the shoulder gutter inlet is less than the allowable flow, the deck drainage design is okay.

7.3 Scuppers (Option 2)

Scuppers are typically formed by tying PVC pipe into place prior to pouring the concrete for the bridge deck (Figure 7-2). The deck runoff will flow into the scuppers, through the deck, and then freefall to the ground or water surface below the bridge.

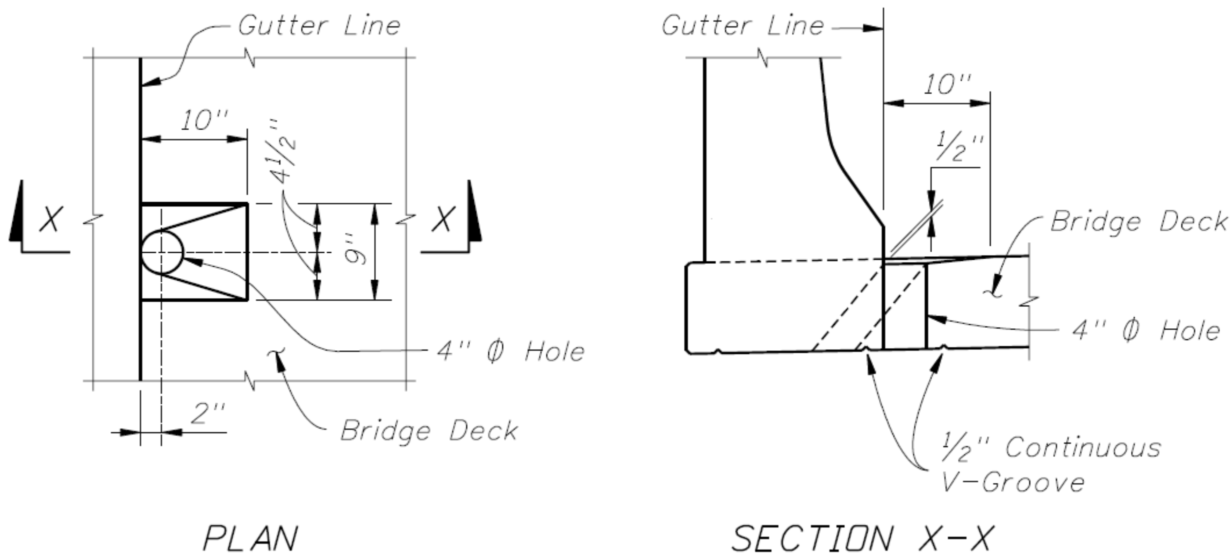


Figure 7-2 Standard FDOT Scupper Detail

Scuppers should be avoided over certain areas due to the direct discharge. These areas include:

- Over driving lanes, railroad tracks, and sidewalks
- Over major navigation channels
- Over bridge bents
- Over erodible soil, unless the free discharge is at least 25 feet above the soil
- Over environmentally sensitive water bodies as negotiated with permitting agencies
- Over wildlife shelves, unless the bottom of the bridges is 25 feet or more above the shelf

As stated in Section 4.9.4 of the Drainage Manual, the standard scupper drain is 4-inch scupper drains spaced at 10 foot centers. This spacing will provide adequate drainage for most bridges. The intercepted flow for 4-inch bridge scuppers on a grade can be evaluated using the capacity curves in Figure 7-3 and Figure 7-4. The curves were derived from laboratory studies performed at the University of South Florida (Anderson, 1973).

Grated scuppers or inlets as shown in Figure 7-5 are more uncommon, especially as free draining scuppers. Although grated inlets can be used with open systems, they are normally used with closed systems. This type of grated scupper, or perhaps one with a smaller grate, might be used to drain a bridge sidewalk or if significant bicycle or pedestrian traffic is expected on the shoulder. The 4-inch ungrated scuppers will not meet ADA requirements.

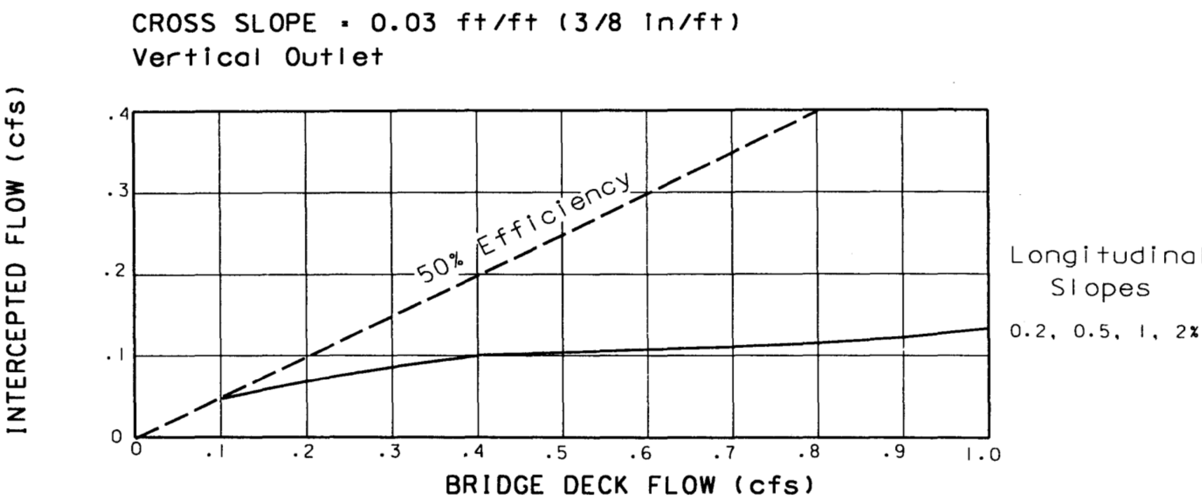


Figure 7-3 Intercepted Flow for 4 inch Bridge Scuppers
Cross Slope = 0.03 ft/ft

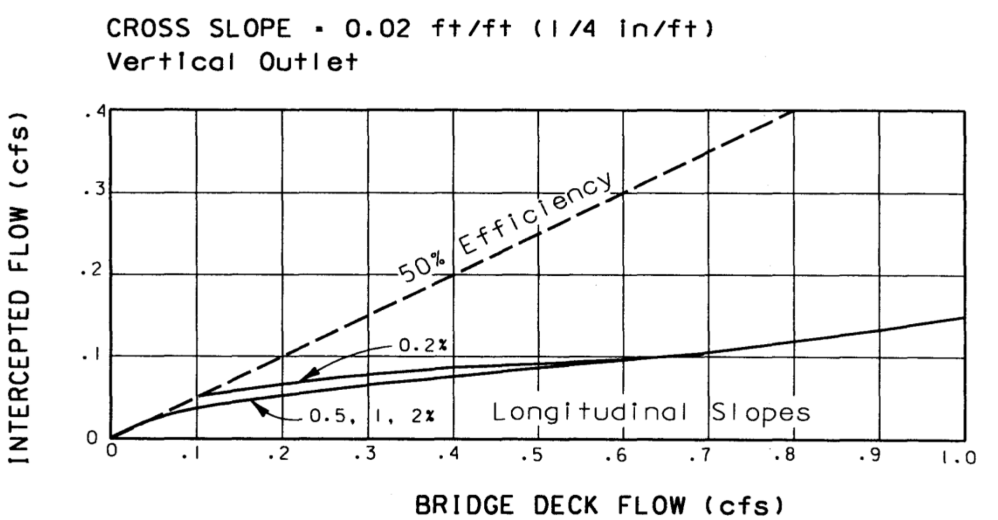


Figure 7-4 Intercepted Flow for 4 inch Bridge Scuppers
Cross Slope = 0.02 ft/ft

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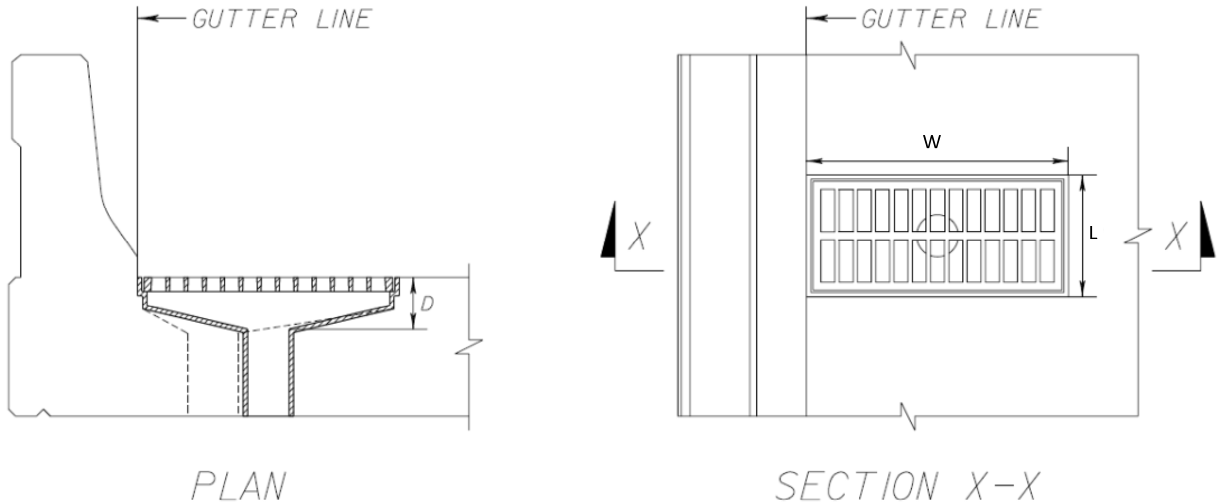


Figure 7-5   Grated Free Draining Scupper

FDOT does not have standard grated scuppers or inlets, therefore does not have capacity charts as with other standard FDOT inlets. Section 3.1 of the FDOT Storm Drain Handbook provides references to documents that can be used to derive inlet capacities. Manufacturers may publish capacity charts for their inlets. The pipe opening at the bottom of the inlet may control the capacity rather than the inlet opening.

The length, width, and depth of the grated inlet will be limited by the reinforcement in the deck of the bridge. The dimensions and locations of the inlets will need to be coordinated with the structural engineer. The hydraulics designer should use standard prefabricated inlets whenever possible. Refer to Section 7.4 for more information on grated scuppers.

Example 7-2

A bridge deck grated scupper is located where the shoulder width is 10 feet and the cross slope is 0.02. The longitudinal grade of the bridge is 1.5%. The dimensions of the grated scupper as defined in Figure 7-5 are:

W = 5 ft.
L = 1 ft.
D = 7 inches
Outlet Pipe Diameter = 8 inches

The flow along the barrier wall at the scupper is 1.65 cfs. Determine the intercepted flow.

Solution:
The spread in the gutter prior to the inlet is:

$$Spread = \left[\frac{Qn}{0.56 S_X^{5/3} S^{1/2}}\right]^{3/8} = \left[\frac{(1.65)(0.016)}{0.56(0.02)^{5/3}(0.015)^{1/2}}\right]^{3/8} = 8.06\,ft.$$

The intercepted flow is calculated using the method presented in FHWA Hydraulic Engineering Circular No. 12, Drainage of Highway Pavements, March 1984 (HEC-12).

The flow directly over the grate is called the frontal flow. The frontal flow can be determined using Equation 7 from HEC-12:

$$E_0 = \frac{Q_W}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{8/3} = 1 - \left(1 - \frac{5}{8.06}\right)^{8/3} = 0.924$$

where:

- E_0 = Ratio of flow in width, W, to the total flow, Q
- Q_W = Flow in width, W, less than T, in cfs
- Q = Total flow, in cfs
- W = Width of flow, W, in feet
- T = Total width of flow (also called the spread), in feet

The frontal flow, $Q_W = E_0 Q = 0.924\ (1.65) = 1.52$ cfs

The inlet will intercept all of the frontal flow unless the velocity is great enough to cause the flow to skip over the grate. This velocity called the Splash-Over Velocity. Chart 7 of HEC-12 can be used to determine the Splash-Over Velocity. Figures 8 through 13 of HEC-12 show the dimensions of the grates in Chart 7. If the grate dimensions do not match one of the grates shown on Chart 7, then the reticuline grate will usually provide a conservative assumption for the splash-over velocity.

Determine the velocity in the gutter:

$$\text{Flow Area} = \frac{S_X T^2}{2} = \frac{0.02(8.06)^2}{2} = 0.650\,ft.$$

$$\text{Gutter Velocity} = \frac{Q}{A} = \frac{1.65}{0.65} = 2.53\,fps$$

The splash-over velocity is conservatively estimated as 2.4 fps from Chart 7, HEC-12. Using Equation 9 from HEC-12, the flow in width, W that is intercepted can be determined:

$$R_F = 1 - 0.09(V - V_0) = 1 - 0.09(2.53 - 2.4) = 0.988$$

where:

R_F = Ratio of the frontal flow intercepted to the total frontal flow
V = Velocity of flow in the gutter, in fps
V_0 = Splash-over velocity, in fps

The intercepted frontal flow is:

$R_F * Q_W = 0.988(1.52) = 1.50$ cfs

The gutter flow that does not flow directly over the grate is called the side flow, Q_S. The side flow can be determined by subtracting the frontal flow from the total gutter flow.

$Q_S = Q – Q_W = 1.65 - 1.52 = 0.13$ cfs

Momentum can carry the side flow past the inlet before all of the flow can turn into the side of the inlet. The amount of flow that turns into the inlet and is intercepted can be calculated using Equation 10 from HEC-12:

$$R_S = 1 \Big/ \left(1 + \frac{0.15V^{1.8}}{S_X L^{2.3}}\right) = 1 \Big/ \left(1 + \frac{0.15(2.53)^{1.8}}{0.02(1)^{2.3}}\right) = 0.0245$$

R_S is the ratio of the side flow intercepted to the total side flow. The intercepted side flow is: $R_S * Q_S = 0.0245(0.13) = 0.00$ cfs. Therefore the total flow intercepted, which is the sum of the frontal and side flows intercepted, is conservatively estimated as 1.50 cfs.

The capacity of the outlet pipe in the bottom of the scupper inlet must also be checked. The capacity can be check by using the orifice equation.

$$Q = CA(2gh)^{1/2}$$

where:

C = Orifice Coefficient = 0.6
A = Area of the orifice opening, in square feet
g = Gravitational Force (32.17 ft/sec^2)
h = Head on the orifice opening, in feet

Assuming that the orifice will not impact the intercepted flow unless the head is equal to the distance from the outlet pipe opening to the top of the grate, D, the outlet pipe capacity is:

$$A = \frac{\pi D^2}{4} = \frac{\pi (8/12)^2}{4} = 0.349\, ft^2$$

$$Q = 0.6(0.349)\left[2(32.17)(7/12)\right]^{1/2} = 1.28 cfs$$

This flow is less than the capacity of the grate, and therefore the outlet pipe controls the interception capacity of the inlet. The actual capacity of the outlet pipe will be slightly greater because the actual head on the pipe will be slightly greater than the top of the grate. However, this value is a conservative estimate of the intercepted flow.

Example 7-3

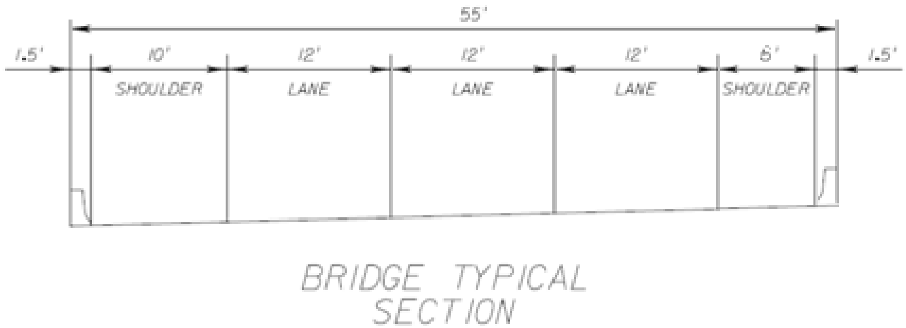
Constant Grade

Scupper flow on bridges with a constant grade will reach an equilibrium state if the bridge is long enough. The equilibrium state occurs when the runoff from the area between scuppers is equal to the flow intercepted by the scuppers.

The spread at scuppers prior to reaching equilibrium will be less than the equilibrium spread. Therefore, equilibrium spread is a conservative estimate for scuppers on a constant grade.

Determine the equilibrium spread for standard scuppers on a bridge with the following characteristics:

- One of dual bridges for a 6 lane divided roadway.
- The deck has a constant 0.02 cross slope.
- The typical section has three 12’ travel lanes, a 10’ outside shoulder, and a 6’ inside shoulder. The barrier walls on each side are 1.5’ wide. The total deck width is 55’.
- The longitudinal grade is a constant 0.2%. (Normally the minimum gutter grade of 0.3% should also be applied to a bridge with flow along its barrier wall. However, older bridges with flatter slopes are sometimes widened rather than replaced. Occasionally, even flat grade bridges are widened.)



Solution:

Since clogging can be a problem for scuppers, it is common to assume that every other scupper is clogged. This assumption doubles the length between functioning scuppers from 10 feet to 20 feet. Using this assumption, the deck runoff generated between each scupper is:

Q = CiA = (0.95)(4)[(55)(20)/43560)] = 0.096 cfs

If the bridge is long enough, the equilibrium flow intercepted by the last scupper will also be equal to this flow rate. Using 0.096 cfs as the intercepted flow, Figure 7-4 can be used to determine the bridge deck flow just upstream of a scupper. Entering the y-axis with the equilibrium intercepted flow of 0.096, an equilibrium flow just upstream of the scupper of 0.61 cfs is read from the x-axis.

The spread just upstream of the scupper is:

$$Spread = \left[\frac{Qn}{0.56S_X^{5/3}S^{1/2}}\right]^{3/8} = \left[\frac{(0.61)(0.016)}{0.56(0.02)^{5/3}(0.002)^{1/2}}\right]^{3/8} = 8.1\,ft.$$

This is the equilibrium spread. Since this value is less than 10 feet, the width of the shoulder, the standard scuppers will be adequate for this bridge.

Usually scuppers will be omitted near the end of the bridge due to potential soil erosion near the abutments. The runoff from this area and the approach slab should be added to the bypass at the last scupper, and the combined Q used to check the spread at the end of the approach slab.

Example 7-4

For this example, use the information for the bridge in Example 7-3 with the following substitutions, scuppers are omitted in the last 50 feet of the bridge, and the bridge has a 30 foot approach slab. Determine the spread at the end of the approach slab.

Solution:

If a bridge has scuppers continuously from the crest of the bridge, then a conservative estimate of the bypass from the last scupper is the equilibrium bypass. From Example 7-3, the equilibrium bypass is:

0.61 cfs – 0.096 cfs = 0.51 cfs

equilibrium bypass

equilibrium scupper interception

equilibrium flow just upstream of scupper

The runoff from the area between the last scupper and the end of the approach slab is:

Q = CiA = 0.95 (4) [(50 +30) 55 / 43560] = 0.38 cfs

Bridge width from Example 7-3

The total flow at the end of the approach slab can be conservatively estimated as:

Q_{Total} = 0.51 + 0.38 = 0.89 cfs

The spread can be conservatively estimated as:

$$Spread = \left[\frac{Qn}{0.56 S_X^{5/3} S^{1/2}}\right]^{3/8} = \left[\frac{(0.89)(0.016)}{0.56(0.02)^{5/3}(0.002)^{1/2}}\right]^{3/8} = 9.3\,ft.$$

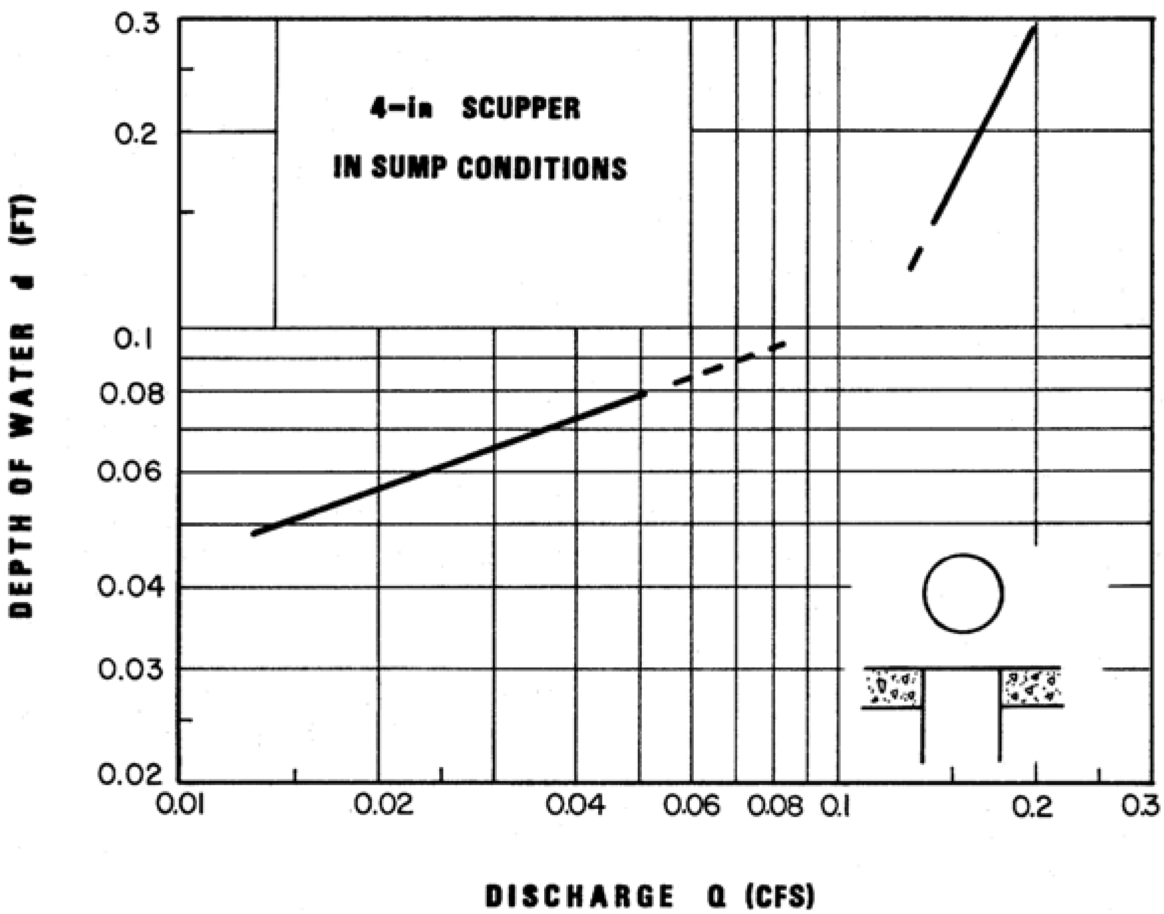
Since the spread is less than 10 feet, the scupper design is okay.

If this estimate exceeded the allowable spread, the bridge deck drainage design does not necessarily need to be changed. The spread can be checked with a more accurate approach that accounts for the flow at each scupper as described in Section 7.4.

Example 7-5

<u>Flat Grade</u>

The capacity of a scupper on a bridge with 0% longitudinal grade can be determined from the figure shown below:



Scupper Capacity in Sump Conditions

Using the bridge of Example 7-3 except with a 0% grade, determine if standard scuppers are adequate.

<u>Solution:</u>

Assuming that every other scupper is clogged, each scupper would need to take the flow from a strip of the bridge deck that is 20 feet wide. The runoff from this area was determined in Example 7-3 to be 0.096 cfs. Entering the above figure with this discharge, the scupper flow will be in the transitional range between weir and orifice flow. The flow conditions are imprecise because of this transition. However, the depth of water above the orifice can be conservatively estimated as 0.11 feet. The spread is:

Spread = depth / Sx = 0.11 / 0.02 = 5.5 feet

Since the spread is less the width of the shoulder, which is 10 feet, standard scuppers meet the criteria.

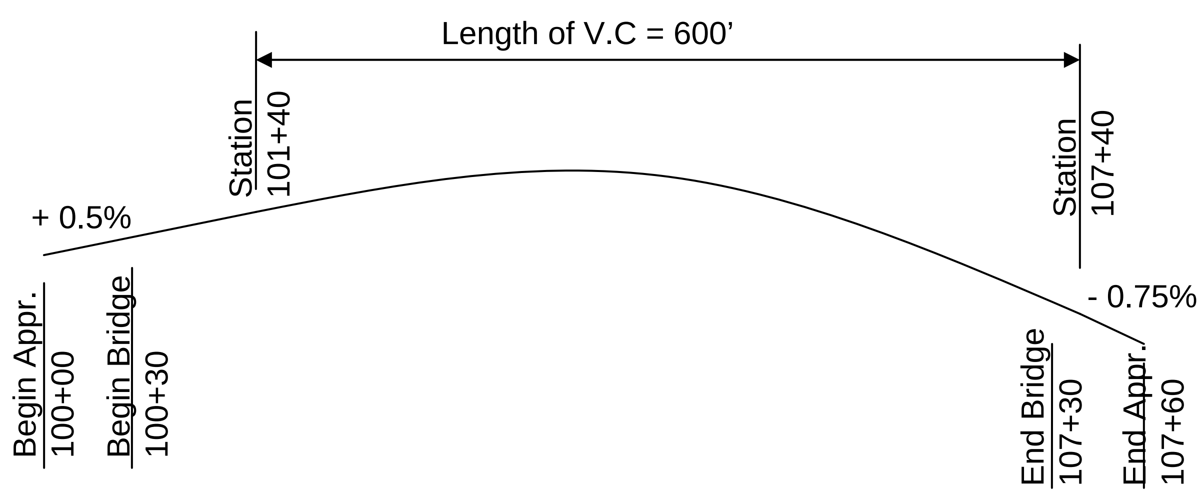
<u>Vertical Curves</u>
Vertical curves complicate the analysis of scupper interception and spacing. However, scuppers on crest curves can be checked at various locations by assuming the grade at that location is a constant grade. This will be conservative for crest vertical curves, but can also be overly conservative. The designer should also consider using a more detailed analysis procedure as described in Section 7.4 before using scupper spacing which deviates from the standard.

At the crest of a vertical curve there is a point where the slope is zero, and depending on the length of the curve there can be a significant portion where the slope is almost flat. The flow depth in this area is not well represented by the Gutter Flow Equation because this equation is a normal depth equation. The flow at the crest will not be at normal depth because it will be experiencing a drawdown due to the combination of steeper slopes and scupper interception downhill. Checking the spread near the crest with the Gutter Flow Equation will be conservative. For slopes less than 0.002, the spread should be checked with the flat grade assumptions if the spread criteria is violated using the Gutter Flow Equation. This is true for both the equilibrium analysis of this section and the more detailed analysis of Section 7.4.

Sag vertical curves should be avoided. If the sag cannot be avoided, then use the more detailed analysis procedure described in Section 7.4.

Example 7-6

Use the bridge of Example 7-3, except with the following roadway profile information:



The ground below the bridge is less than 25 feet below the bottom of the bridge deck for a distance of 50 feet from each bridge end. Determine the required deck drainage features.

Solution:
Determine the location of the high point on the bridge:

$X_{HIGH\ POINT}$ = (g1 x L) / (g2 – g1)
= (0.005 x 600) / (0.0075 – 0.005)
= 240'

Therefore, the high point is located at Station 103+80

The drainage area at the edge of the approach slab at Station 100+00 is:

Area = (55) (380) / 43560 = 0.48 acres

The flow is:

Q = CiA = 0.95 (4) (0.48) = 1.82 cfs

where:
- C = Rational runoff coefficient
- i = Rainfall intensity, inches per hour (Refer to FDOT Storm Drain Handbook for explanation to use 4 in/hr)
- A = Drainage area, acres

Solving the gutter flow equation for spread:

$$Spread = \left[\frac{Qn}{0.56 S_X^{5/3} S^{1/2}}\right]^{3/8} = \left[\frac{(1.82)(0.016)}{0.56(0.02)^{5/3}(0.005)^{1/2}}\right]^{3/8} = 10.3\,ft.$$

The spread exceeds the allowable spread of 10 feet. Minor changes to the roadway and bridge profile would reduce the spread to an acceptable amount, less than 10 feet. However, if after discussions with the roadway and the bridge engineers, the roadway grade cannot be adjusted, then the use of standard scuppers can be considered. For this example, we will assume the roadway grade cannot be adjusted.

The drainage area and flow are the same at the other bridge end at Station 107+60. The spread is:

$$Spread = \left[\frac{Qn}{0.56 S_X^{5/3} S^{1/2}}\right]^{3/8} = \left[\frac{(1.82)(0.016)}{0.56(0.02)^{5/3}(0.0075)^{1/2}}\right]^{3/8} = 9.5\,ft.$$

Since this spread is less than 10 feet, scuppers are not needed from the high point of the bridge at Station 103+80 to the bridge end at Station 107+30.

Omitting scuppers within 50 feet of the bridge end, standard scuppers would be placed every 10 feet starting at Station 100+80 and ending at Station 103+70. The next step is to determine if this design meets spread criteria. The previous examples show this design will work:

- Example 7-5 shows that standard scuppers on this bridge will meet the spread criteria for flat grades. Therefore, scuppers at the top of the vertical curve where the longitudinal slope is less than 0.002 will meet the spread criteria.
- Example 7-3 shows that standard scuppers on this bridge will meet the spread criteria for grades equal to or greater than 0.002.
- Example 7-4 shows that the spread at the end of the approach slab will also meet criteria.

Therefore, the deck drainage design for this bridge is standard scuppers starting at Station 100+80 and ending at Station 103+70.

7.4 Closed Collection Systems (Option 3)

The third option is a closed system. A closed system will be needed if:

- The spread criteria is exceeded without scuppers or inlets on the bridge,
- The deck drainage cannot be allowed to freefall to the area below the bridge, and
- The roadway profile or shoulder width cannot be adjusted.

Closed systems should use grated inlets to minimize debris in the piping system. Refer back to Section 7.3 for guidance on determining the interception capacity of grated inlets. The dimensions and locations of the inlets will need to be coordinated with the structural designer. The above deck design (i.e., size and location of the grated inlets) should be analyzed using a more detailed procedure rather than the equilibrium assumptions from the previous sections. A typical procedure is illustrated in Table 7-2. An example of this procedure is presented in Appendix D.

Table 7-2 Typical Inlet Location Analysis

Inlet Location	Drainage Area	Discharge	Spread	Bypass
Station 1				
Station 2				
⋮				
Station n				

Station 1: The first inlet downhill of the crest
Drainage Area: The area between the inlet and the crest for the first inlet. For subsequent inlets, the area uphill to the previous inlet.
Discharge: The sum of the discharge from Drainage Area plus the bypass from the previous inlet.
Spread: Calculated using the Gutter Flow Equation or the flat area assumptions.
Bypass: Determined by the inlet or scupper capacity.

The below deck system will have a network of pipes to convey the discharge collected by the inlets to an outlet location. There are two types of systems. One type discharges downward at the piers or bents. This type of system is more commonly used for overpasses. The inlets are typically located near the pier, so there are few horizontal segments of pipe and flow is not combined from multiple inlets. Therefore, the controlling point hydraulically will typically be the entrance to the piping system at the inlet.

The other type of system discharges at the bridge ends. The system will require longitudinal pipes along the bridge which will carry the combined flow of multiple inlets. The below deck piping system should be designed using a procedure similar to the procedure in the FDOT Storm Drain Handbook. The procedure may be modified to use the driver visibility limiting rainfall intensity of 4 inches per hour.

Besides the hydraulic capacity of the piping system, the layout of the system should also consider:

- Minimum cleaning velocities – Three feet per second is recommended.
- Cleanout locations – The locations should consider both access to all segments of the pipe system and access to the cleanout by maintenance personnel.
- Design the underdeck closed drainage system to minimize sharp bends, corner joints, junctions, etc. These occasionally reduce the hydraulic capacity of the system but, more importantly, these features provide opportunities for debris to

snag and collect. Y-connections and bends should be used for collector pipes and downspouts to help prevent clogging in mid-system.

- Pipes should be UV resistant. If not, then pipes should be located to prevent UV exposure. Tucking the pipe system behind the bridge beams will prevent UV exposure.

The optional material for bridge collection pipes can be found in Chapter 22 of the Structures Detailing Manual. No matter what type of pipe is used, attention must be given to the design of a hanger system which should be designed by or in coordination with the bridge design engineer. If the collection system is connected to a roadway structure, the engineer may need to call for a resilient connector. Coordination with the structures engineer is critical for proper design.

Chapter 8

Bridge Hydraulics Report Format and Documentation

Section 4.11.2 of the FDOT Drainage Manual lists the minimum information that must be included in the BHR. The minimum requirements are broken down for:

- Bridge and bridge culvert widening
- Bridge culverts
- Category 1 and 2 bridges

The introduction to Section 4.11.2 has a concise set of rules to guide production of all sections in the BHR. Review of this brief paragraph before compiling the documentation can help focus the BHR. Additional general guidance to follow while preparing the BHR is:

- The BHR should be presented in clear and concise language, and it should not repeat information or have unsubstantiated comments.
- Graphics should address the technical aspects of the project with the public's point of view in mind.
- There should be consistency of report format as well as consistency in units with alternative units presented where appropriate.

8.1 Bridge Hydraulics Report Preparation

Although the level of detail will vary depending on the type of work (i.e., bridge widening, bridge replacement, or a new bridge crossing), the complexity of the hydrology and hydraulics of the site, and the regulatory requirements, the following general chapter outline is sufficient for most reports:

- Executive Summary
- Introduction
- FEMA / Regulatory Requirements
- Hydrology
- Hydraulics
- Scour
- Deck Drainage
- Appendices

8.1.1 Executive Summary

The Executive Summary should be a concise statement of findings. The existing and proposed bridges should be described. Include a summary of all design recommendations for the proposed bridge crossing. The objective of the Executive Summary is to provide the findings in an opening statement so that when the report is reviewed in the future, the reviewer would immediately understand why the particular bridge was chosen. The following list is in Section 4.11 of the Drainage Manual:

1. Bridge Length, including locations (stations) of abutments
2. Channel Excavation requirements (if channel excavation is required by project)
3. Minimum Vertical Clearance:
   - Provide this information as a minimum elevation with the associated datum. Briefly describe clearance requirements that were considered (debris, navigation, environmental corrosion, wave forces, etc.) and identify which one controlled the minimum elevation.
4. Minimum Horizontal Clearance
5. FEMA / Regulatory Requirements
6. Abutment Type and Orientation
7. Pier Orientation
8. Scour Depths:
   - The scour elevation should be provided for both the Design Event and the Check Event
9. Scour protection requirements for abutments, piers, and channel. For spill-through abutments, recommendations shall include:
   - Abutment Slope
   - Type of Protection (rubble riprap is standard)
   - Horizontal and Vertical Extent of Protection
10. Deck Drainage requirements
11. Wave and surge parameters and forces
12. There should be a brief conclusion recounting why the proposed bridge length was selected. The discussion should include other bridge considerations that were pertinent or had an important influence on this project. The important influences might include the following:
    - Costs
    - Maintenance of traffic
    - Roadway geometrics that affect bridge length

- Hydrology
- Hydraulics
- Scour
- Stream geomorphology
- Constructability
- Environmental concerns
- Wildlife shelf requirements
- Other unique concerns particular to the site

For bridge widening, this discussion is not necessary.

Include a discussion of any variations from policies in the Drainage Manual, Plans Preparation Manual, or Structures Manual.

8.1.2 Introduction

The introduction should briefly describe the location of the bridge, including the name of the water body being crossed. Giving the latitude and longitude and/or the township, range, and section will enhance the location description. A figure showing a location map is recommended.

Describe the waterway and floodplain at the proposed crossing. Describe the existing crossing, if any, including the bridge, relief bridges, and roadway embankment within the floodplain. The description of bridges should only include details that affect the hydraulics:

- Bridge length
- Span lengths
- Foundation type and sizes
- Low member elevations
- Deck and beam heights
- Other details that affect the hydraulics

Also, describe the purpose of the project (widening, replacement, etc).

Describe the land use in the area potentially affected by backwater from the crossing. Discuss any nearby buildings or other structures that will potentially control the allowable backwater from the crossing.

State the date of the site visit, and include photographs as figures.

Describe any pertinent information from the Bridge Inspection Report (BIR), and consider including a copy of the report in an appendix. Discuss any information obtained from contact with FDOT Maintenance.

State the associated datums for each data source and provide datum conversions needed to convert elevations between differing datums.

8.1.3 Floodplain Requirements

Discuss requirements of FEMA and other regulatory agencies (Section 2.2) that may influence the design of the crossing. Consider including an appendix with the correspondence, meeting minutes, phone notes, etc. from coordination efforts with the agencies. If the original FEMA model was obtained include a copy in the appendix.

8.1.4 Hydrology

Discuss the methods used to determine and check the flow rates used in the analysis. Include a summary table of frequencies and discharges used in the final analysis.

The hydrologic calculations, computer input and output, or documentation obtained from others used to establish the design flow rates should be included in an appendix.

8.1.5 Hydraulics

8.1.5.1 One Dimensional Model Setup

Identify and briefly describe the computer program used to calculate the water surface elevations. Include a figure showing the location of the cross sections used in one dimensional models. Figures 8-1 and 8-2 are examples of cross section location figures. Describe the following aspects of the model development:

- How the data for all the cross sections were obtained and how cross section locations were selected.
- How the starting water surface elevations (tailwater conditions) were determined.
- How the Manning’s roughness coefficients were selected.

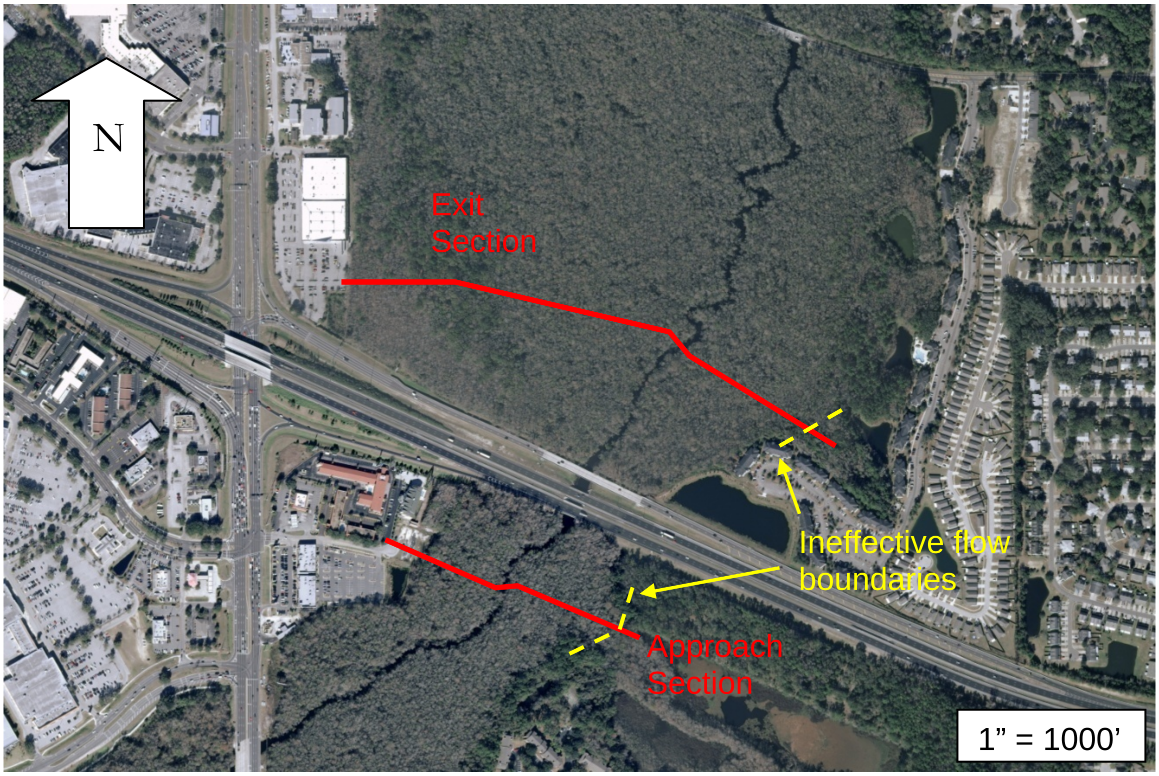


Figure 8-1 Example Cross Section Location Figure on an Aerial

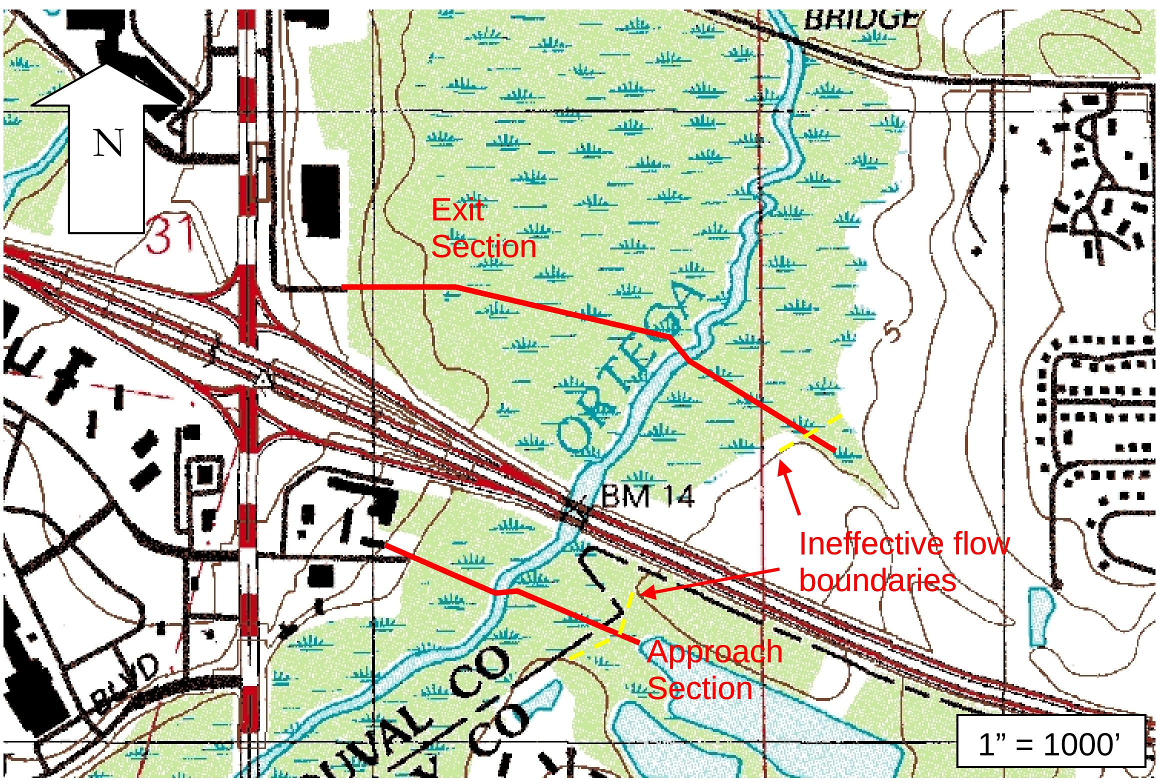


Figure 8-2 Example Cross Section Location Figure on a Quadrangle Map

If warning messages remain in the final output, describe any attempts to eliminate the warnings and the reasoning for not resolving them. Input and output from the computer programs used to analyze the crossing should be included in the appendices. Electronic copies of the input files will also be provided to the Department.

In some cases, such as bridge widenings that do not affect the water surface profiles, calculations may not be performed. However, the flood data at the site is still required in the plans per FHWA requirements. If the flood data is not calculated, then it must be obtained from another source. Typical sources that can be used are hydraulic reports for the existing crossing or FEMA Flood Insurance Studies. Document the source in the report.

Compare water surface elevations for the existing and proposed alternative bridges. The location of the Approach Section may vary between the existing bridge and each of the alternative proposed bridges. The water surface elevation comparisons should be done at a section that is at a common location in each model in order for the comparison to be valid. As illustrated in Figure 8-3, the comparison should be made at the location of the Approach Section that is furthest upstream.

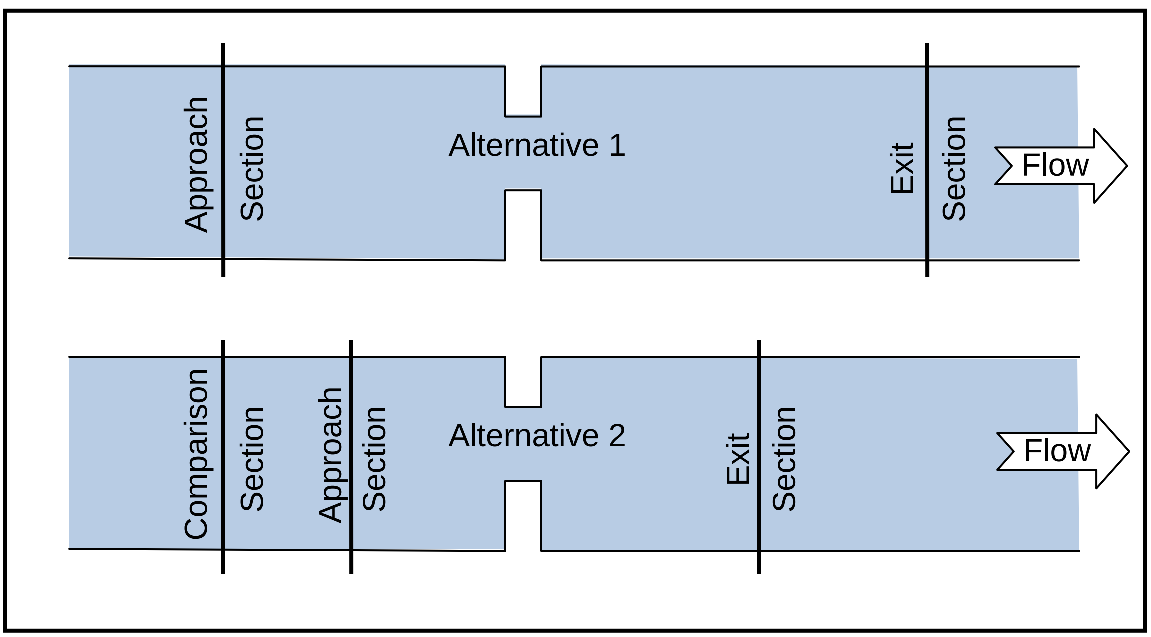


Figure 8-3 Water Surface Elevation Comparisons

Include a table that summarizes the water surface elevations for the existing and alternative bridges. Table 8-1 is an example of a table comparing water surface elevations.

Table 8-1 Example Water Surface Elevation Comparison

	50 Year Elevation	100 Year Elevation	500 Year Elevation
Existing Conditions	57.4	57.8	59.0
Proposed Conditions	57.2	57.8	59.1

Elevations are NGVD 1929. Elevations shown on the BHRS in the Appendix have been converted to NAVD 88. The elevations are adjusted by subtracting 0.65'.

8.1.5.1 Two-Dimensional Model Setup and Results

If two-dimensional modeling was performed as part of the hydraulic analysis of the bridge, the BHR should contain sufficient documentation of the model development and simulation to provide the reviewer, and subsequent readers of the report, a clear understanding of both the modeling process and the results of the modeling. This begins with a description of the model selected and justification for that selection. The report should document who or what agency developed the model (e.g., FHWA's FESWMS model) as well as the features of either the model or the physical features of the study area that make the model the appropriate choice.

Documentation of the model development should include the following:

- A description of the survey data employed (including horizontal and vertical datums).
- A description of the boundary conditions as well as sufficient documentation of their development.
- Documentation of the selected friction specification.
- A listing of other model input parameters (e.g., turbulent closure parameters, time step size, etc.).
- Graphic representations of the model mesh clearly displaying both elevation contours and elements (e.g., Figure 8-4 through Figure 8-6). Figures should display both the model domain as well as a close-up of the bridge location to ensure documentation of the resolution of the study area.

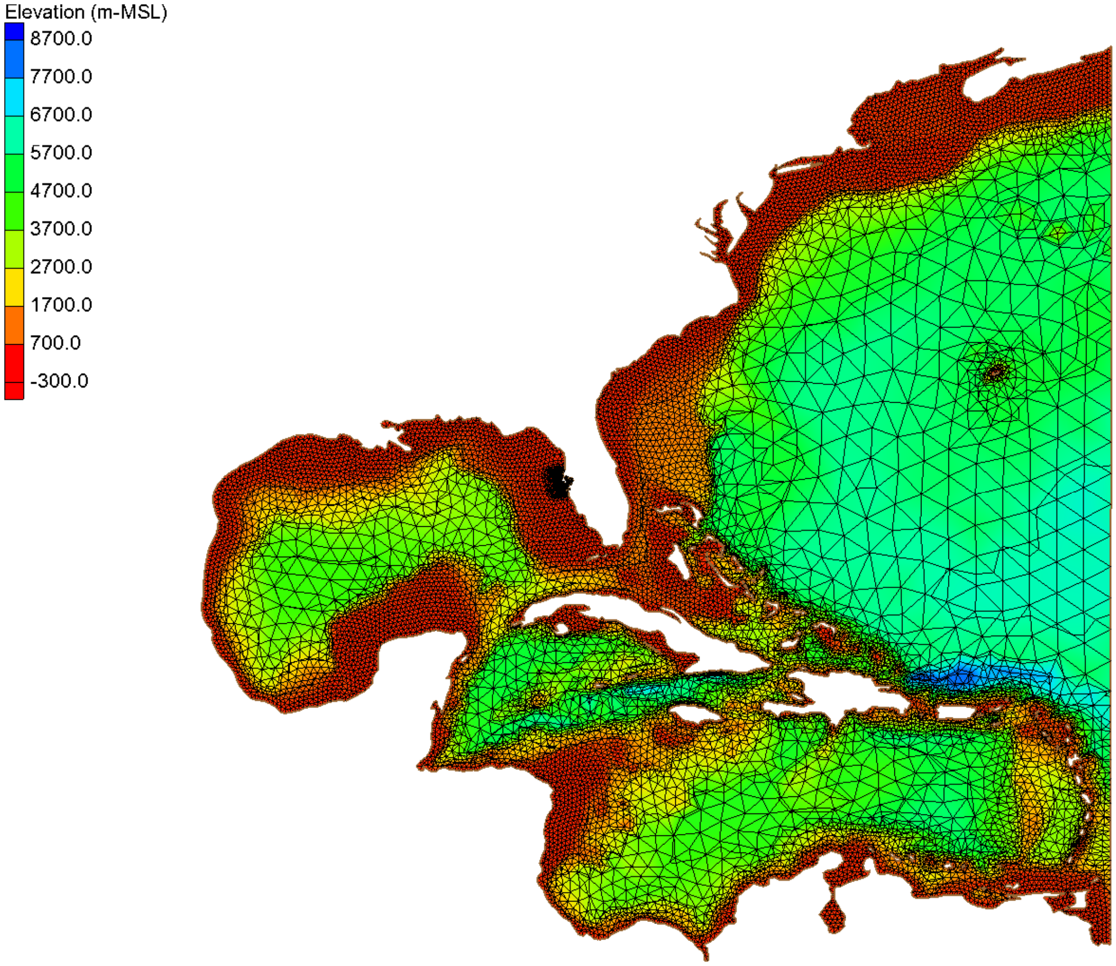


Figure 8-4 Tampa Bay Model Mesh Domain

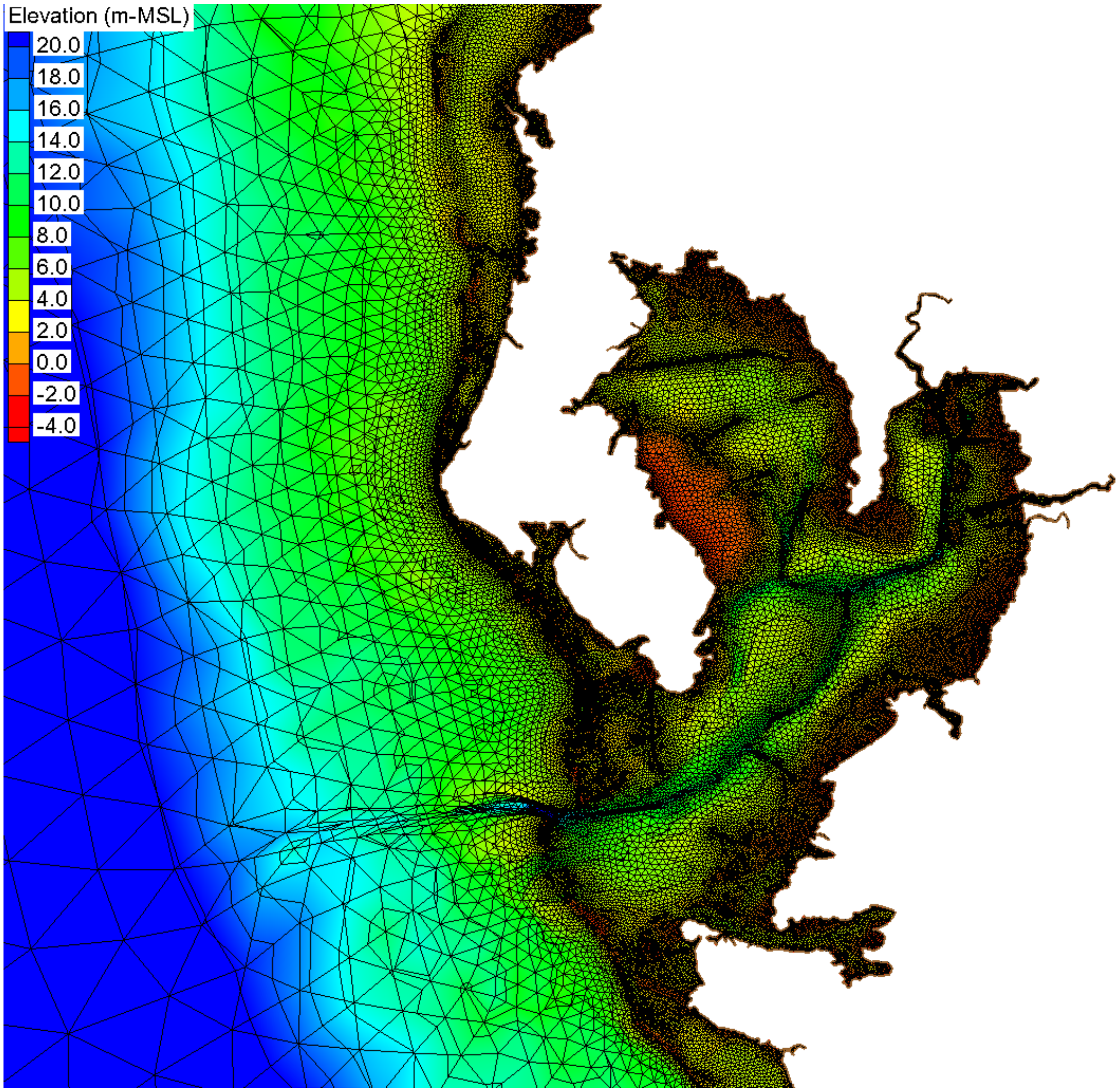


Figure 8-5 Model Mesh at Tampa Bay

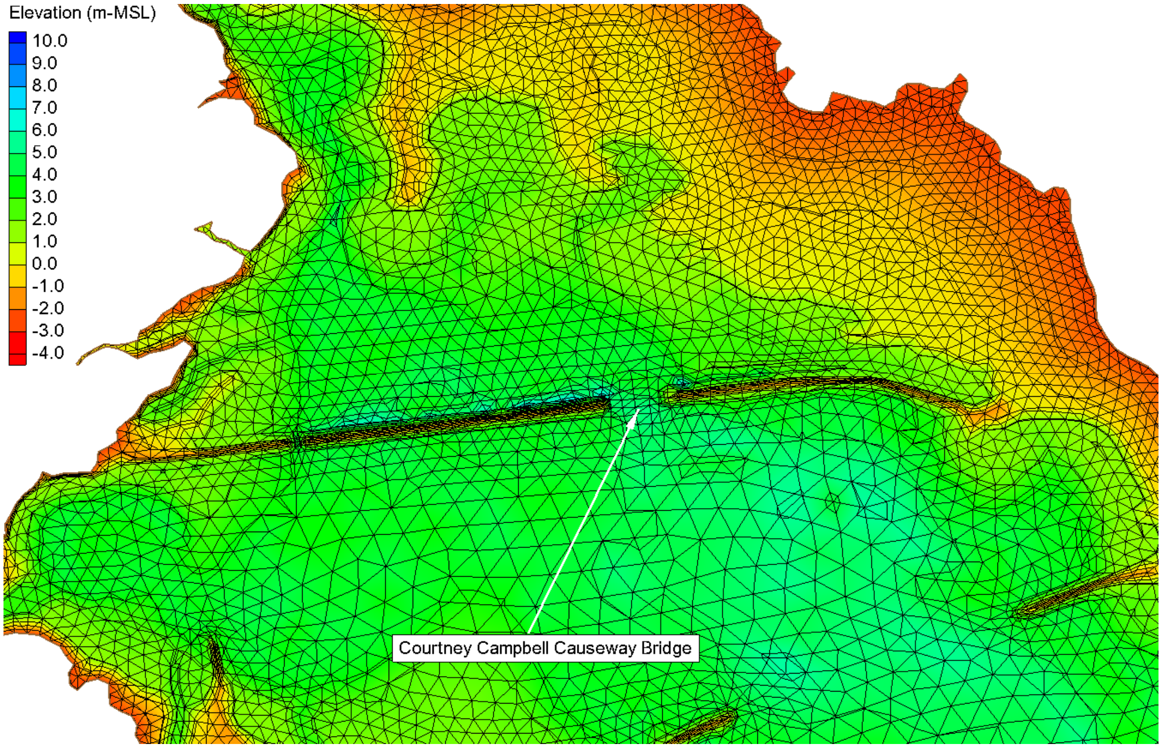


Figure 8-6 Model Mesh at the Courtney Campbell Causeway Bridge

Documentation of the two-dimensional model should include:

- A complete description of the calibration process
    - o Calibration data
    - o The simulations
    - o Parameters changed to achieve calibration
    - o Parameters of the model
- Both a qualitative and quantitative description of the model's capability to predict measured data
    - o Calculation of mean error
    - o Standard deviation
    - o Percentage error, etc. over time series, between observed high water marks, measured stages, or comparison with predicted tidal ranges.

Examples of qualitative descriptions are provided in Figure 8-7 and Figure 8-8 which show comparisons between measured and modeled water surface elevations and flow rates.

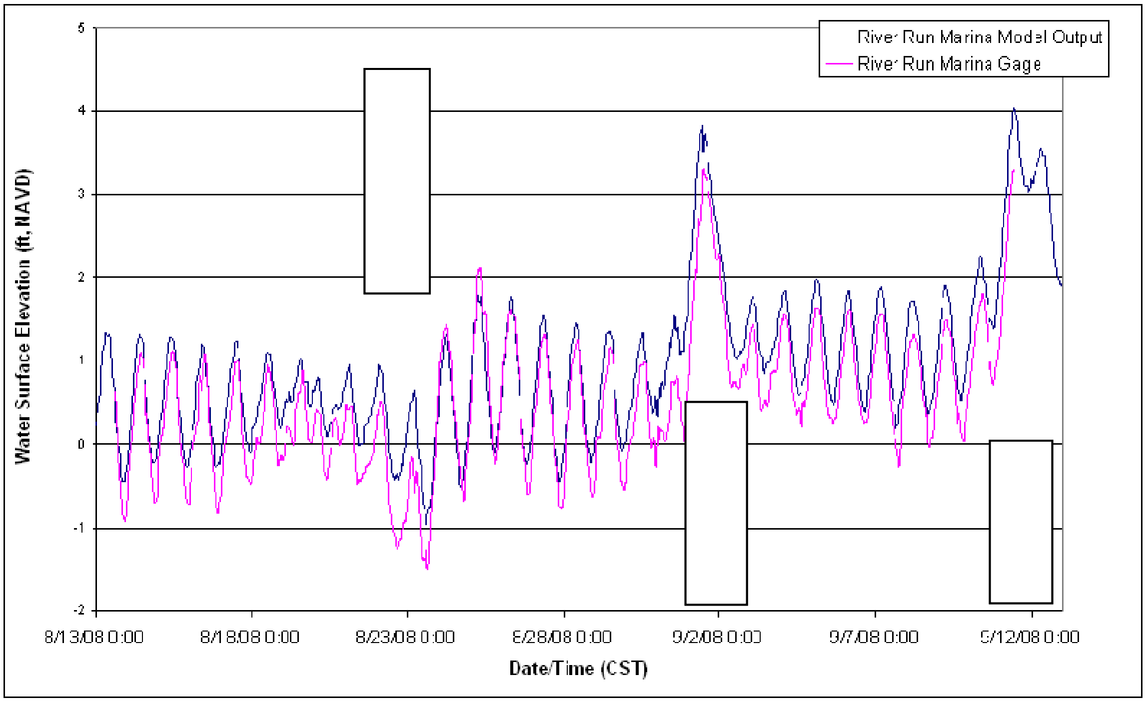


Figure 8-7 Model Calibration Plot for the US-90 Bridge over Macavis Bayou Replacement Project at the River Run Marina

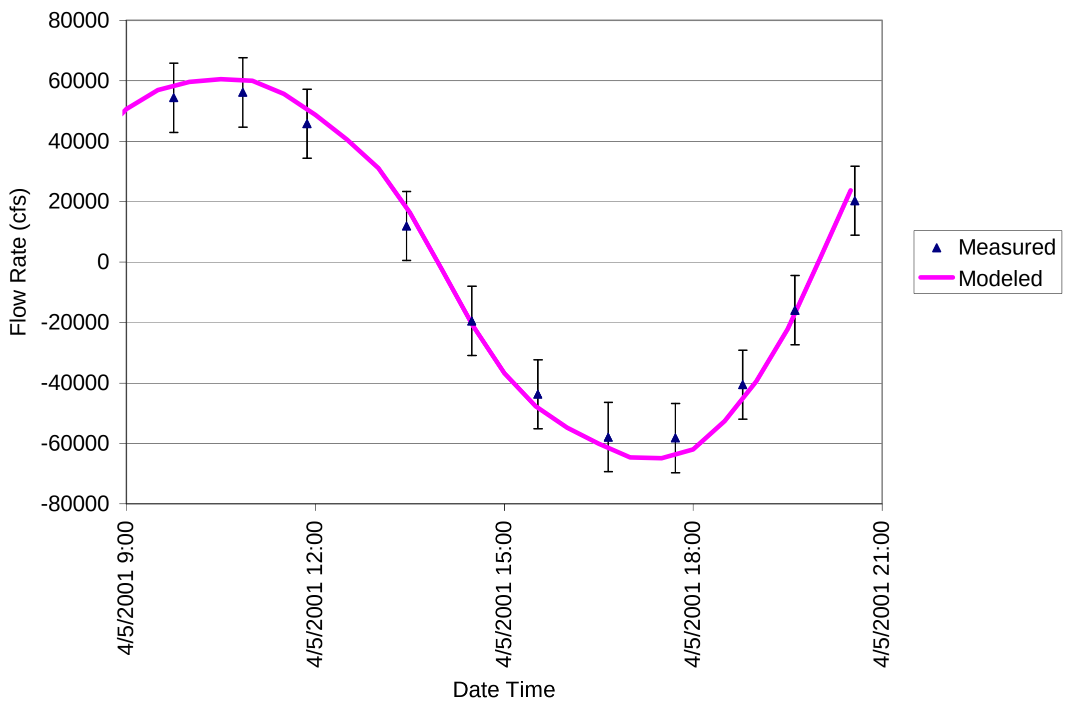


Figure 8-8 Flow Rate Calibration at Lake Worth Inlet (Error Bars Indicate 10% Error)

Documentation of two-dimensional modeling simulation results should include, at a minimum:

- Table of max conditions for each simulation at the bridge
- Figures of each simulation (Figure 8-9)
    - o Display contours of velocity magnitude
    - o Velocity vectors displaying the direction of the flow across bridge
- For long bridges, hydraulic parameters at each pier or groups of piers should list
    - o Max stage
    - o Max flow rate
    - o Max velocity
    - o Angle of attack
- Tidal Analysis (Time dependent simulation)
    - o Time series plot of design values for stage, velocity and flow rate (Figure 8-10 through Figure 8-12)

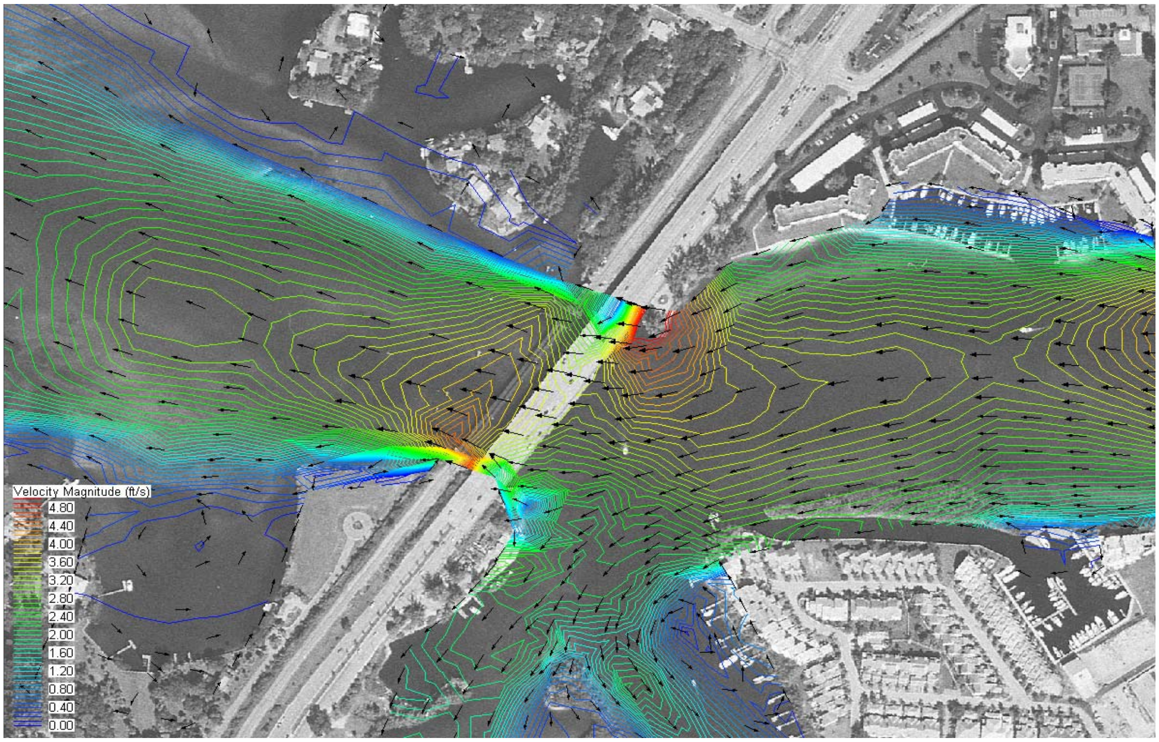


Figure 8-9 Velocity Magnitude Contours and Velocity Vectors at the Time of Maximum Velocity during the 100-year Storm Surge Event at the SR-A1A Bridge over the Loxahatchee River

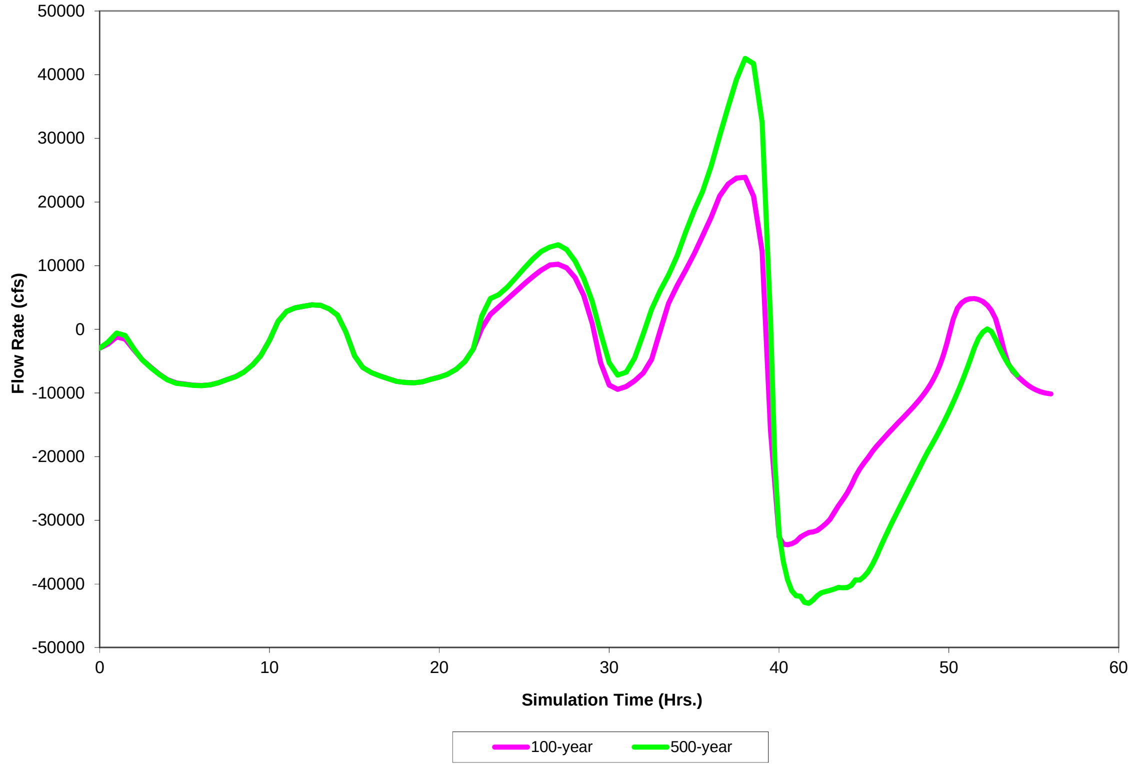


Figure 8-10 Flow Rate Time Series during the Design and Check Event at the SR-A1A Bridge over the Loxahatchee River

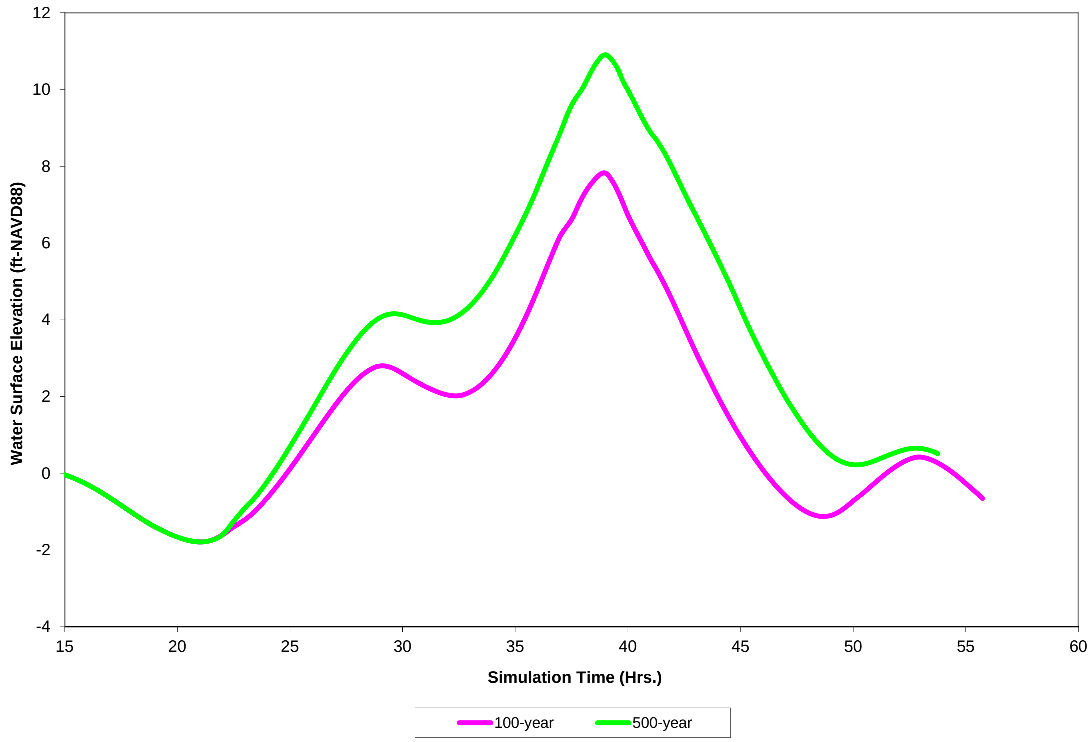


Figure 8-11 Water Surface Elevation Time Series during the Design and Check Event at the SR-A1A Bridge over the Loxahatchee River

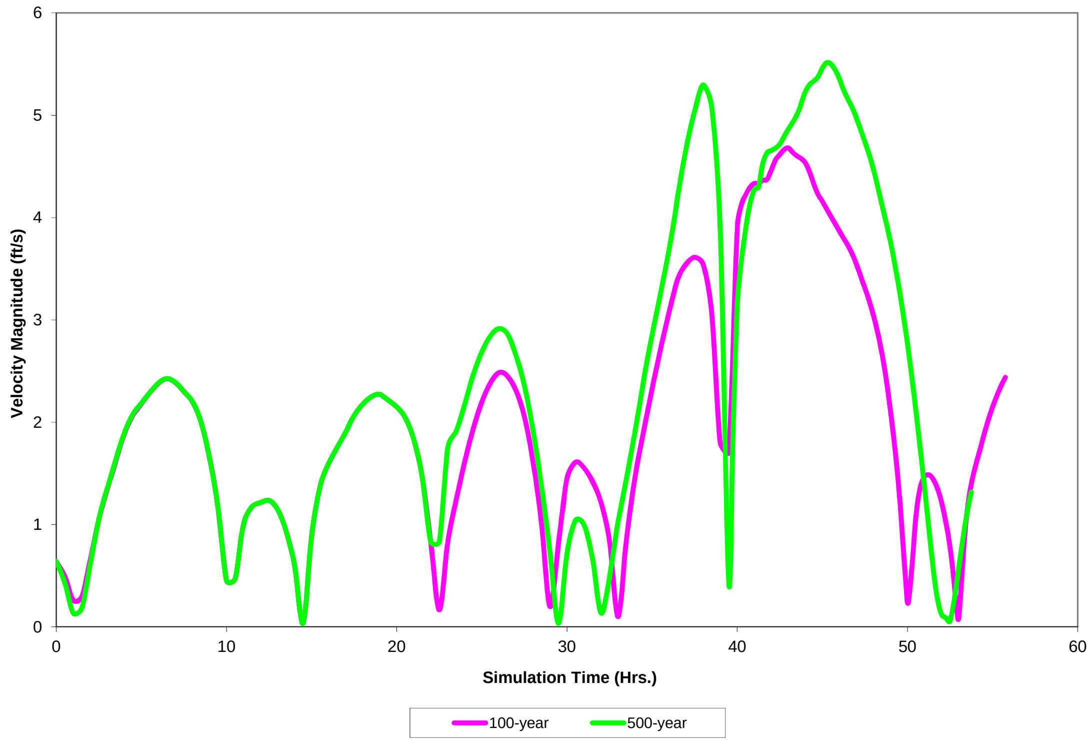


Figure 8-12 Velocity Magnitude Time Series during the Design and Check Event at the SR-A1A Bridge over the Loxahatchee River

Required documentation of two-dimensional wave modeling is almost identical to that for hydraulic analyses. The only difference is in the parameters themselves. At a minimum, the wave parameters should include the highest significant wave height at the bridge cross section, the associated peak period, the maximum wave height, and the maximum crest elevation with all parameters associated with the 100-year return period conditions.

8.1.5.2 Alternatives Analysis

This section will not be needed for bridge widening projects. For new and replacement bridges, this section should document the cost analysis, environmental impacts, and other impacts on adjacent properties. Each alternative should still meet the design standards, but if exceptions must be made for an alternative, then the exception should be included in the comparisons. This section must document the reasons for selecting the recommended alternative.

8.1.6 Scour

There should be a discussion of the stream geomorphology, the scour history, long-term aggradation or degradation, and the scour values, including discussion of the methods used to determine each. Scour depths should be plotted in a figure.

Discuss the proposed abutment protection. If the one of the standard abutment protection designs given in Section 4.9 of the FDOT Drainage Manual is used, abutment scour need not be calculated and plotted. Other abutment protection designs may be used in certain circumstances, but not without prior approval from the District Drainage Office.

8.1.7 Deck Drainage

Document the proposed method of deck drainage. Justify the use of longitudinal collection systems. Include in the appendix spread and interception calculations as well as capacity calculations for any longitudinal collections systems.

8.1.8 Appendices

Calculations and other backup documentation should be included as appendices to the BHR to avoid disrupting the flow of the main body of the report. Items to consider including in the appendices are:

- Hydrology calculations
- Hydrology reports from other sources
- Hydraulic calculations
- Hydraulic reports from other sources
- FEMA report excerpts and maps
- Scour computations
- Cost calculations for alternatives
- Deck drainage calculations
- Regulatory requirements and permits
- Memos, meeting minutes, and phone notes

8.2 Bridge Hydraulics Report Process

Exhibit 24-A of the FDOT Plans Preparation Manual (PPM), Volume 1, gives the Approval and Concurrence Process for the Bridge Hydraulics Report. Section 27.4 of the PPM specifies the multidisciplinary approach to follow for scour considerations, along with submittal requirements. The BHR must be prepared in conjunction with the

Bridge Development Report and preliminary Structures Plans. Exhibit 27-A of the PPM outlines a flow chart for the Structural Plans Development Process.

The process flow chart in Figure 8-13 shows the general sequence of events necessary to prepare a Bridge Hydraulics Report. Additional coordination may be needed especially for projects involving floodways or for other complex projects.

After the hydraulics engineer has a relatively good idea of the approximate structure length and location, a field review should be conducted. The preliminary structure length and location, along with preliminary scour depths and low member elevations should then be submitted to the Structures Design Office for their preliminary evaluation. After the proposed bridge configuration and foundation type have been developed and submitted back to the hydraulics engineer, the final hydraulic and scour analyses should be performed and submitted back to the Structures and Geotechnical Departments.

The hydraulics engineer should then have the BHR and BHRS reviewed internally (or by an outside consultant, if necessary). After all comments have been addressed, the hydraulics engineer should **approve** the BHR and BHRS and submit them to the FDOT for **concurrence**. After the BHR and BHRS receive concurrence from the FDOT, the final BHR and BHRS should be submitted to the structural and geotechnical engineers so that the BDR and geotechnical reports may be completed.

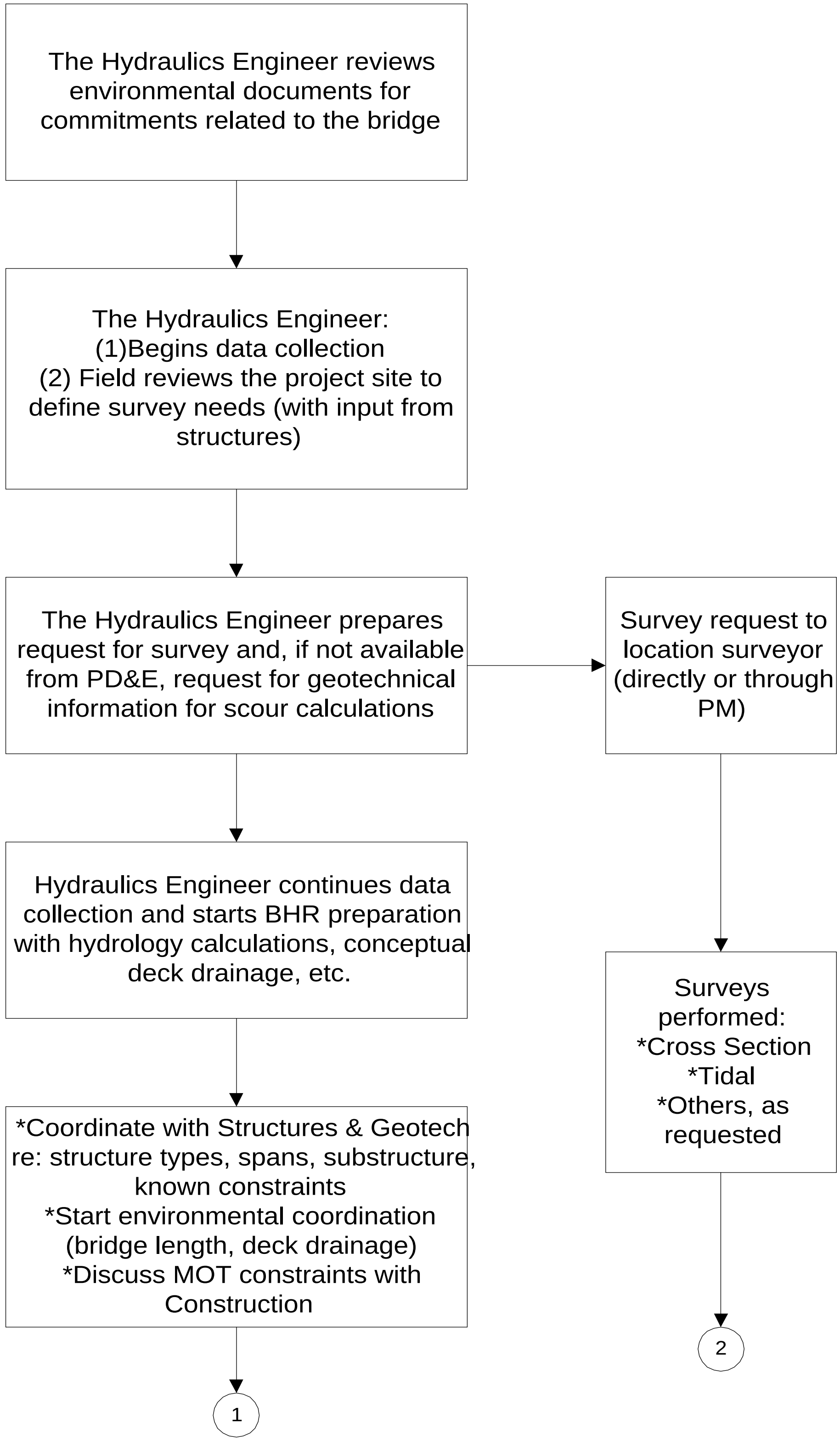


Figure 8-13 Bridge Hydraulics Report Process

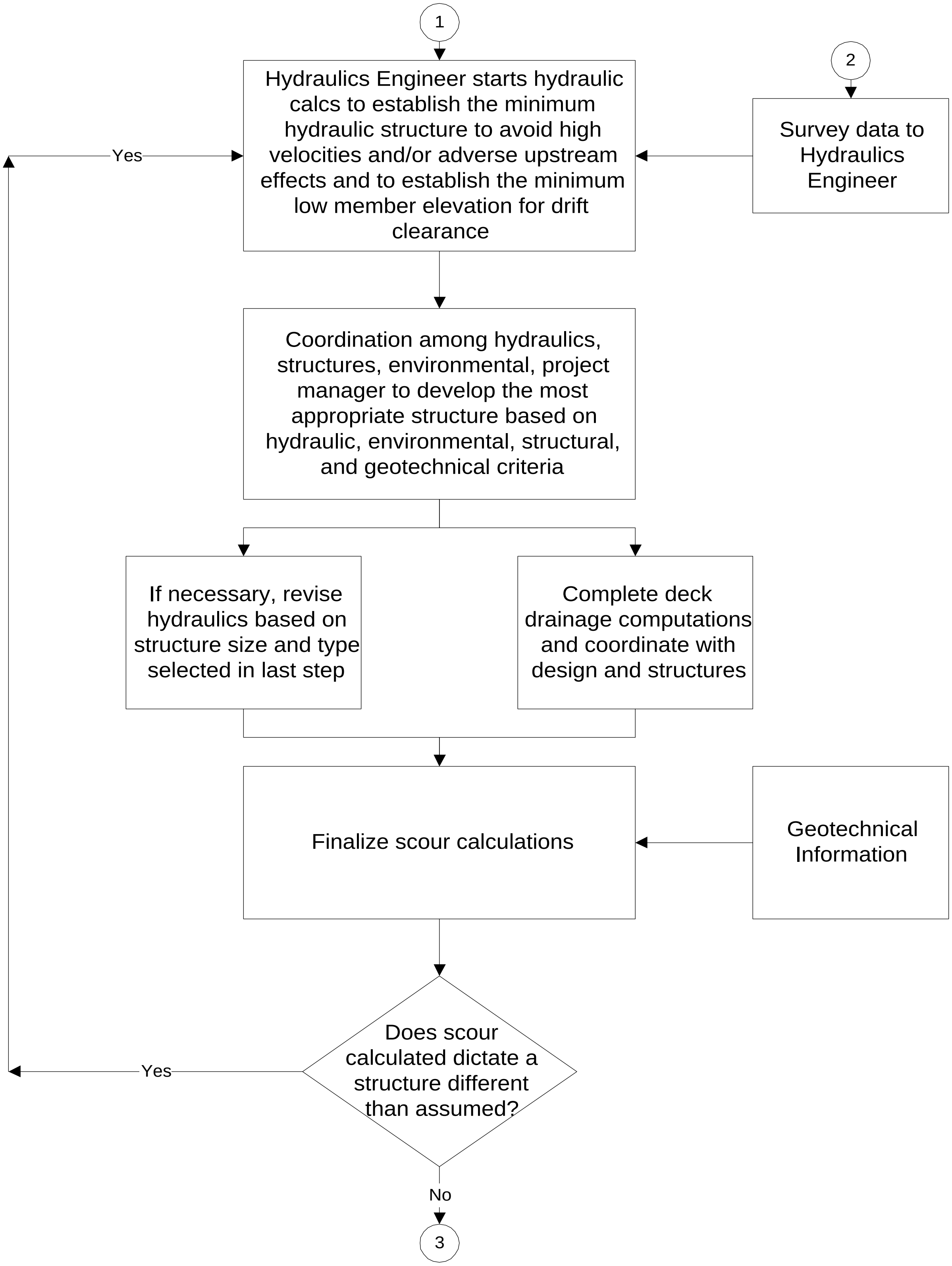


Figure 8-13 (cont.) Bridge Hydraulics Report Process

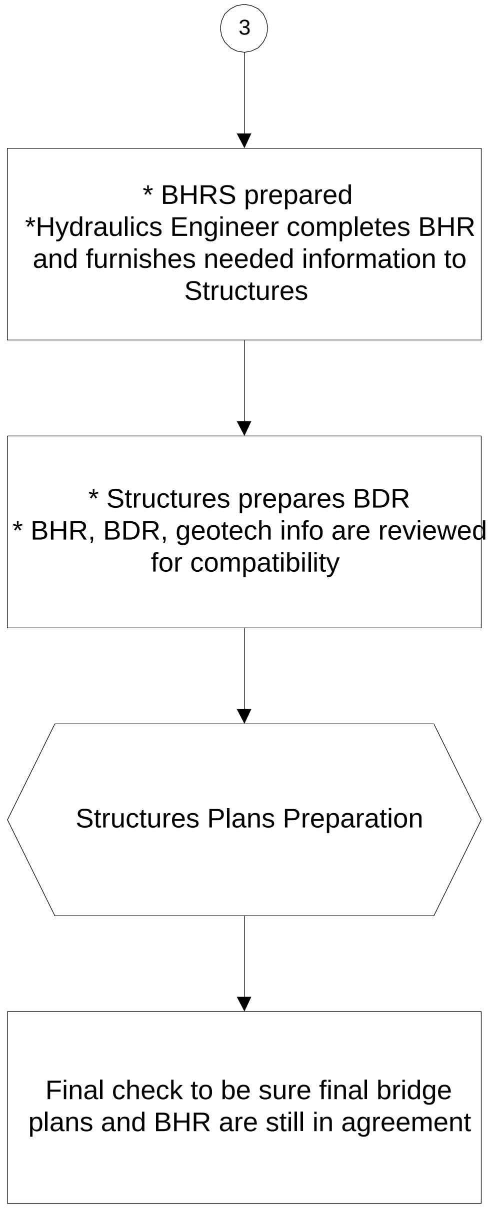


Figure 8-13 (cont.) Bridge Hydraulics Report Process

8.3 Common Review Comments

By far, the most frequent comments associated with the BHR and BHRS address omissions or requests for supporting documentation. The following checklist should provide an additional resource to ensure a quality product for submission to the FDOT:

- Draft Bridge Hydraulics Report
    - Verify that the report contains the following information:
        - Bridge location
        - Bridge number (if available)
        - Florida County
        - Description of all data collected in the office data collection
        - Description of all data collected in the field data collection
        - List of relevant datums (e.g. NAVD 88, NGVD 29, etc.). Provide the difference between datums if supporting documents, new data, and the Plans use different datums
        - Description of the model hydrology
        - Description of the constructed hydraulic model
        - Description of the modeling procedures (inputs, boundary conditions, etc.)
        - Quantitative and qualitative presentation of the calibration simulation results
        - Presentation of the simulation results
        - Description of scour calculation procedures
        - Aggradation/degradation calculation (methodology and results)
        - Channel migration calculation results (methodology and results)
        - Contraction scour mode and calculation results (inputs and output)
        - Local scour calculations and results (inputs and output)
        - Total Design scour prediction
        - Total Check Event scour prediction
        - Recognize that maximum scour for the above events can occur at a flow less that the associated return interval flow rate, i.e. if overtopping occurs before either the Total Design scour or Total Check Event scour
        - Wave climate/wave modeling discussion
        - Wave force calculation procedure and results (inputs and output)

        - Abutment protection recommendations and calculations (inputs and output)
        - Deck drainage discussion
    - Check the report for the following:
        - Language is clear and concise
        - Presentation graphics address the technical aspects of the project with the public's point of view in mind
        - Consistency of report format
        - Consistency in units with alternative units presented where appropriate
        - Cross referencing of figures, tables, section numbers within the document
- Draft Bridge Hydraulics Recommendations Sheet
    - Verify that the BHRS contains the following information:
        - Plan View
            - Stationing, scale, and north arrow. Include the channel baseline if one was created.
            - Existing topography (including existing bridge) and contours (show elevations)
            - Label the name of the water body
            - Arrows showing the direction of the flow
            - Proposed bridge begin and end station
            - Limits and type of abutment protection
            - Right-of-way lines
        - Profile View
            - Stationing and scale
            - Existing surveyed cross section
            - Road profile for the proposed structure with stationing and elevations
            - Proposed bridge with begin and end station, low member, and pier locations
            - Abutment locations (toe of slope) and abutment protection
            - Design flood elevation
            - Normal High Water / Mean High Water

  - New Bridge Number
- Drainage Map and Location Map
  - Location map with north arrow
  - Range and township and an arrow showing the project location.
  - Entire drainage area for the proposed structure
  - Calculated drainage area
  - Water elevations on date of survey
- Existing Structures, Hydraulic Design Data and Hydraulic Recommendations
  - Existing Structures
  - Proposed Structure
  - Foundation
  - Overall Length
  - Span Length
  - Type of Construction
  - Area of Opening
  - Bridge Width
  - Elevation of Low Member
- Hydraulic Information
  - Normal High Water (Non-Tidal)
  - Control (Non-Tidal)
  - Mean High Water (Tidal)
  - Mean Low Water (Tidal)
  - Max. Event of Record
  - Design Flood Information
  - Base Flood Hydraulic and Scour Information
  - Overtopping Flood/Greatest Flood Hydraulic and Scour Information
  - Begin Bridge Station
  - End Bridge Station
  - Skew Angle

        - Navigation Clearances – required and provided
        - Drift Clearances – required and provided
        - Abutment protection description – begin and end bridge
        - Deck Drainage
        - Remarks
- Final Bridge Hydraulics Report
    - Verify that the report contains the following information:
        - Changes to the report as specified by the responses to comments following the FDOT review process
- Final Bridge Hydraulics Recommendations Sheet
    - Verify that the BHRS contains the following information:
        - Changes to the BHRS as specified by the responses to comments following the FDOT review process

8.4 Bridge Hydraulics Recommendations Sheet (BHRS)

The Bridge Hydraulics Recommendations Sheet (BHRS) provides a single reference that summarizes the findings and recommendations of the hydraulic analysis. The BHRS flood data must match that given in the BHR and computer output.

The BHRS is divided into four regions:

- Plan View
- Profile View
- Location Map and Drainage Area
- Existing Structures, Hydraulic Design Data and Hydraulic Recommendations

The minimum requirements of the first three regions are given in the Volume 2, Chapter 5 of the FDOT Plans Preparation Manual (PPM). In addition, consider the following items:

- In the Plan View, the PPM requires that the limits of riprap be shown. However, abutment protection other than riprap may be proposed. Show the horizontal extents and label the protection type in either the plan or profile view.
- Plot and label the profile of the existing natural ground in the Profile View, and note the existing elevation at each end.
- When practical, the profile of the expected design scour (contraction and long-term scour along the entire unprotected cross section and the local scour at the intermediate piers/bents) should be shown. Local scour holes shall be displayed

as beginning at the foundation element edges at the design scour depth and extending up at a 1V:2H slope to meet the profile illustrating the contraction/long-term scour profile.

- Although the profile grade line must be plotted in the Profile View, percent of grade need not be shown. The PC, PI, and PT of vertical curves should be plotted using their respective standard symbols; however, no data (station, elevation, length of curve) needs be noted. Begin and end bridge stations shall be flagged.

Figure 8-14 shows a larger view of the region of the BHRS that includes Existing Structures, Hydraulic Design Data, and Hydraulic Recommendations. The hydraulic design data and hydraulic recommendations are for the proposed structure. The required data is identified by bold numbers in parentheses and a brief description is provided on the following pages.

(REFERENCE)	EXISTING STRUCTURES (1) (1)	(2)	(3)	(4)	PROPOSED (2) STRUCTURE
FOUNDATION (3)					
OVERALL LENGTH (4)					
SPAN LENGTH (5)					
TYPE CONSTRUCTION (6)					
AREA OF OPENING @ D.F. (7)					
BRIDGE WIDTH (8)					
ELEV. LOW MEMBER (9)					

HYDRAULIC DESIGN DATA

NOTE:
The hydraulic data is shown for informational purposes only to indicate the flood discharges and water surface elevations which may be anticipated in any given year. This data was generated using highly variable factors determined by a study of the watershed. Many judgements and assumptions are required to establish these factors. The resultant hydraulic data is sensitive to changes, particularly antecedent conditions, urbanization, channelization and land use. Users of this data are cautioned against the assumption of precision which cannot be obtained.

TERMS:
Design Flood: Utilized to assure a desired level of hydraulic performance.
Base Flood: Has a 1% chance of being exceeded in any given year (100 year frequency)
Overtopping Flood: Causes flow over the highway, over a watershed divide, or thru emergency relief structures.
Greatest Flood: The most severe that can be predicted where overtopping is not practicable.

WATER SURFACE ELEVATIONS: N.H.W. (Non-Tidal) (10) M.H.W. (Tidal) (12)
CONTROL (Non-Tidal) (11) M.L.W. (Tidal) (13)

FLOOD DATA:	(14) MAX. EVENT OF RECORD	(15) DESIGN FLOOD	(16) BASE FLOOD	(17) ☐ OVERTOPPING or ☐ GREATEST FLOOD
STAGE ELEV. NGVD (ft) (18)				
DISCHARGE (cfs) (19)				
AVERAGE VELOCITY (f/s) (20)				
EXCEEDANCE PROB. (%) (21)				
FREQUENCY (yr.) (22)				

SCOUR PREDICTIONS FOR PROPOSED STRUCTURE DESCRIBED ABOVE:

PIER INFORMATION NUMBERS	SIZE AND TYPE	LONG TERM SCOUR ELEV.	TOTAL SCOUR ELEVATION WORST CASE < 100 yr. FREQ. (yr.) (23)	WORST CASE < 500 yr. FREQ. (yr.) (24)
(25)	(26)	(27)	(28)	(29)

HYDRAULIC RECOMMENDATIONS

1. BEGIN BRIDGE STATION (30) END BRIDGE STATION (31) SKEW ANGLE (32)
2. CLEARANCE PROVIDED: NAV: HORIZ. (33) VERT. (34) ABOVE EL. (35) DRIFT: HORIZ. (36) VERT. (37) ABOVE EL. (38)
3. MINIMUM CLEARANCE: NAV: HORIZ. (39) VERT. (40) ABOVE EL. (35) DRIFT: HORIZ. (41) VERT. (42) ABOVE EL. (38)
4. ABUTMENTS:

	BEGIN BRIDGE	END BRIDGE
RUBBLE GRADE:	(43)	(43)
SLOPE:	(44)	(44)
BURIED OR NON-BURIED HORIZ. TOE:	(45)	(45)
TOE HORIZ. DISTANCE:	(46)	(46)
LIMIT OF PROTECTION:	(47)	(47)

5. DECK DRAINAGE: (48)

REMARKS: (49)(50)

Figure 8-14 BHRS Required Data

(1) Existing Structures: Structure 1 refers to the structure being replaced or modified. Structures 2, 3 & 4 refer to relief structures, immediate upstream and downstream structures and those structures that affect the hydraulics of the proposed structure.

(2) Proposed Structure: This column should have information pertaining to the proposed structure.

(3) Foundation: This row should have information describing the type of foundation (e.g., timber piles, concrete piles, etc.).

(4) Overall Length (ft): This row should give the total length of the structure in feet. The length should be measured from the top of the abutments. For the proposed structure, this length should match the total length shown in the final plans.

(5) Span Length (ft): This row should give the span length of the structure in feet. This length should be based on the length at the main span.

(6) Type Construction: This row should have information describing the material(s) used for construction of the structure (e.g., steel, concrete, steel and concrete, etc.).

(7) Area of Opening (ft^2) @ D.F.: This row should have the area of opening in square feet below the design flood elevation less the assumed pile area, if significant, at the bridge section.

(8) Bridge Width (ft): The bridge width should be from rail to rail, including the rails, in feet.

(9) Elev. Low Member (ft): This elevation in feet should be the lowest point along the low member of the structure.

(10) N.H.W. (Non-Tidal) (ft): The Normal High Water at the bridge. This water surface elevation in feet only applies to non-tidal areas.

(11) Control (Non-Tidal) (ft): The water surface elevation in feet controlled by the operation of pump stations, dams or other hydraulic structures.

(12) M.H.W. (Tidal) (ft): The Mean High Water elevation in feet at the bridge. This water surface elevation only applies to tidal areas.

(13) M.L.W. (Tidal) (ft): The Mean Low Water elevation in feet at the bridge. This water surface elevation only applies to tidal areas.

(14) Max. Event of Record: This column provides information related to the maximum event recorded based on historical information (if available).

(15) Design Flood: This column provides information related to the design flood.

(16) Base Flood: This column provides information related to the base flood.

(17) Overtopping Flood/Greatest Flood: If the overtopping flood has a lower return period than the greatest flood, then the block indicating overtopping flood is checked and the information related to the overtopping flood is shown. Otherwise, the block indicating greatest flood is checked and the information related to the greatest flood is shown.

(18) Stage Elev. NAVD 88 or NGVD 29 (ft): For freshwater flow, the elevation in feet typically taken from the hydraulic model at the Approach Section for the design flood and/or base flood, overtopping flood, greatest flood. Proper engineering judgment is required for long bridges since it may not be realistic to use the elevation at the Approach Section because the losses between the bridge and Approach Section are large.

For tidal flow, the maximum elevation during the flood or ebb storm surge at the bridge for the design flood and/or base flood, overtopping flood, greatest flood. Add a remark that stage, discharge, and the velocity described in the flood data do not occur at the same time.

(19) Discharge (cfs): For freshwater flow, the total discharge in cubic feet per second used in the simulations for the design flood, base flood, overtopping flood and/or greatest flood.

For tidal flow, the maximum discharge during the flood or ebb storm surge at the bridge for the design flood, base flood, overtopping flood and/or greatest flood. Add a remark that stage, discharge, and the velocity described in the flood data do not occur at the same time.

(20) Average Velocity (fps): For freshwater flow, the average velocity in feet per second taken from the computer simulations at the Bridge Section for the design flood, base flood, overtopping flood and/or greatest flood.

For tidal flow, the maximum average velocity at the bridge section during the flood or ebb storm surge for the design flood, base flood, overtopping flood and/or greatest flood.

(21) Exceedance Prob. (%): The probability that the conditions are exceeded. Determined as 100% times unity over the return interval (e.g., 100%*(1/100) = 1%).

(22) Frequency (yr): The return period of the conditions in years.

(23) Frequency (yr): The frequency (return period) in years of the worst case scour condition up through the design return period flow conditions.

(24) Frequency (yr): The frequency (return period) in years of the worst case scour condition up through the design check period flow conditions

(25) Pier No.: The pier number or range of pier numbers which correspond to the pier size and type in Column 26 and the scour elevations in Columns 27, 28 and 29.

(26) Pier Size and Type: The proposed pier size and type which produces the greatest scour. If necessary for clarity, place a reference to the appropriate details of the bridge plans. If the space provided is not adequate, place the information in the plan or profile view.

(27) Long-Term Scour (ft): Applicable only to structures required to meet extreme event vessel collision load. See section 6.2 for definition of long-term scour. If it is not applicable, state so.

(28) Total Scour Elevation (< 100-year) (ft): The predicted total scour elevation in feet for the worst case scour condition up through the scour design flood frequency. This includes aggradation or degradation, channel migration, local scour (pier and abutment) and contraction scour.

(29) Total Scour Elevation (< 500-year) (ft): The predicted total scour elevation in feet for the worst case scour condition up through the scour design check flood

frequency. This includes aggradation or degradation, channel migration, local scour (pier and abutment) and contraction scour.

(30) Begin Bridge Station: The station for the beginning of the bridge.

(31) End Bridge Station: The station for the end of the bridge.

(32) Skew Angle (degrees): The angle in degrees at which the structure is skewed from the centerline of construction. See the Department's Design Standards Index 289 Sheet 1 of 7, Schematic "B" for further explanation.

(33) Navigation Clearance (Horiz.) (ft): The actual horizontal navigation clearance in feet provided between fenders or piers.

(34) Navigation Clearance (Vert.) (ft): The actual vertical navigational clearance in feet provided between fenders or piers.

(35) Navigation Clearance (Above El.) (ft): For freshwater flow, the elevation (NAVD 88 or NGVD 29, ft) at the normal high water (NHW) elevation or control elevation.

For tidal flow, this is the elevation at mean high water (MHW).

(36) Drift Clearance (Horiz.) (ft): The actual minimum horizontal clearance in feet provided.

(37) Drift Clearance (Vert.) (ft): The actual minimum vertical clearance in feet provided above the design flood.

(38) Drift Clearance (Above El.) (ft): For freshwater flow, this is the design flood elevation (NAVD 88 or NGVD 29, ft) and either of two values is appropriate. In many cases, it is reasonable to use the elevation at the Approach Section, realizing that this will be slightly higher than actual elevation at the bridge.

For tidal flow, use the maximum stage associated with an average velocity of 3.3 fps through the bridge section during the flood or ebb for the storm surge for the design flood. If the maximum velocity due to the storm surge is less than 3.3 fps, use the stage associated with the maximum velocity through the bridge section. If either of these stages causes the profile to be higher than the profile of the bridge approaches, consider other alternatives. One alternative is to discuss with personnel in the Structures Design Office, the potential of having less drift clearance and designing the structure for debris loads. Another alternative is to do a more rigorous and site specific analysis to set the stage above which to provide the standard drift clearance. Investigate and address these situations on a site specific basis.

(39) Navigation Clearance (Horiz.) (ft): The minimum horizontal navigation clearance in feet required. Section 4.6 of the FDOT Drainage Manual lists the minimum requirements. Other agencies may have minimum clearance requirements.

(40) Navigation Clearance (Vert.) (ft): The minimum vertical navigation clearance in feet required. FDOT minimum clearances are discussed in Section 2.4. Other agencies may have minimum clearance requirements.

(41) Drift Clearance (Horiz.) (ft): The minimum horizontal debris drift clearance in feet required. FDOT minimum clearances are given in Section 4.6 of the FDOT Drainage Manual.

(42) Drift Clearance (Vert.) (ft): The minimum vertical debris drift clearance in feet required above the design flood. FDOT minimum clearances are discussed in Section 2.4.

(43) Rubble Grade: Grade of rubble (e.g., Riprap (Bank & Shore), etc.) to be constructed at the begin and end bridge abutments. References can be made to details sheets if non-standard riprap is employed.

(44) Slope: Slope of the abutments at the begin and end bridge (e.g., 1H:2V, etc.).

(45) Non-buried or Buried Horiz. Toe: Indicate whether the toe of the abutment will be non-buried or buried when extended horizontally from the bridge. See Section 6.4 of this handbook for details.

(46) Toe Horizontal Distance (ft): Horizontal extent in feet of the rubble protection measured from the toe of the abutment. See Section 6.4 of this handbook for details.

(47) Limit of Protection (ft): Distance measured parallel to the stationing in feet, from the edge of the rubble protection to the bridge begin/end station. If the distance is different on each side, indicate both distances with their corresponding sides.

(48) Deck Drainage: Type of deck drainage to be used for the proposed structure (e.g., scuppers, storm drain system, etc.)

(49) Remarks: This space is available to record any pertinent remarks.

(50) Wave Crest Elevation (ft): The 100-year design wave crest elevation in feet including the storm surge elevation and wind setup. The vertical clearance of the superstructure shall be a minimum of 1 foot above the wave crest elevation.

Appendix A

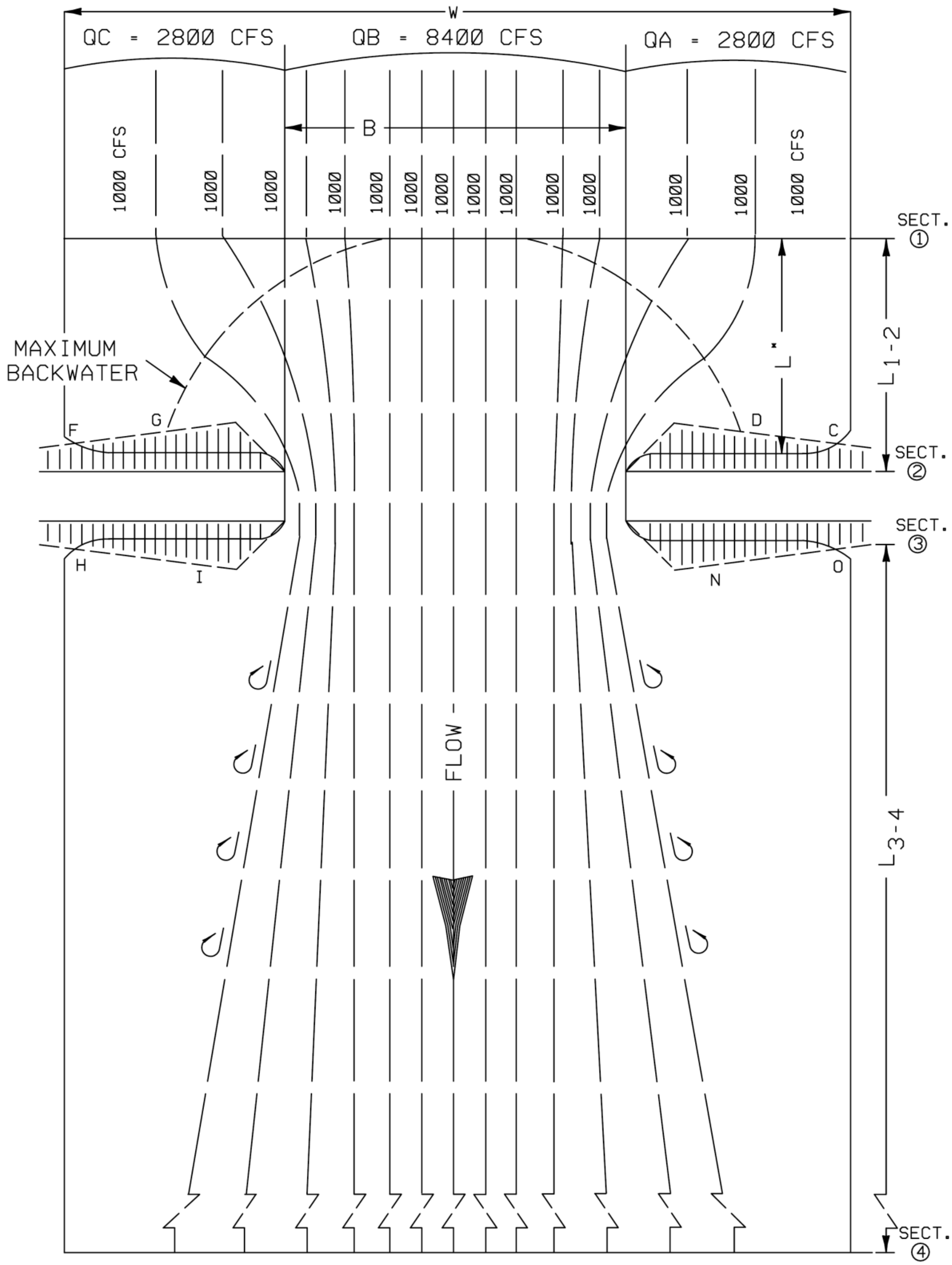
Bridge Hydraulics Terminology

A.1 Backwater

It is seldom economically feasible or necessary to span the entire width of a stream at flood stages. Where conditions permit, approach embankments are extended onto the flood plain to reduce costs, recognizing that in doing so the embankments will constrict the flow of the stream during flood stages. Normally, this is an acceptable practice, provided that water surface profile and scour conditions are properly evaluated.

The manner in which flow is contracted in passing through the channel constriction is illustrated in Figure A-1. The flow bounded by each adjacent pair of streamlines is the same (1000 cubic feet per second). Note that the channel constriction appears to produce practically no alteration in the shape of the streamlines near the center of the channel. A very marked change occurs near the abutments, however, since the momentum of the flow from both sides (or floodplains) must force the advancing central portion of the stream over to gain entry to the constriction. Upon leaving the constriction, the flow gradually expands (5 to 6 degrees per side) until normal conditions in the stream are reestablished.

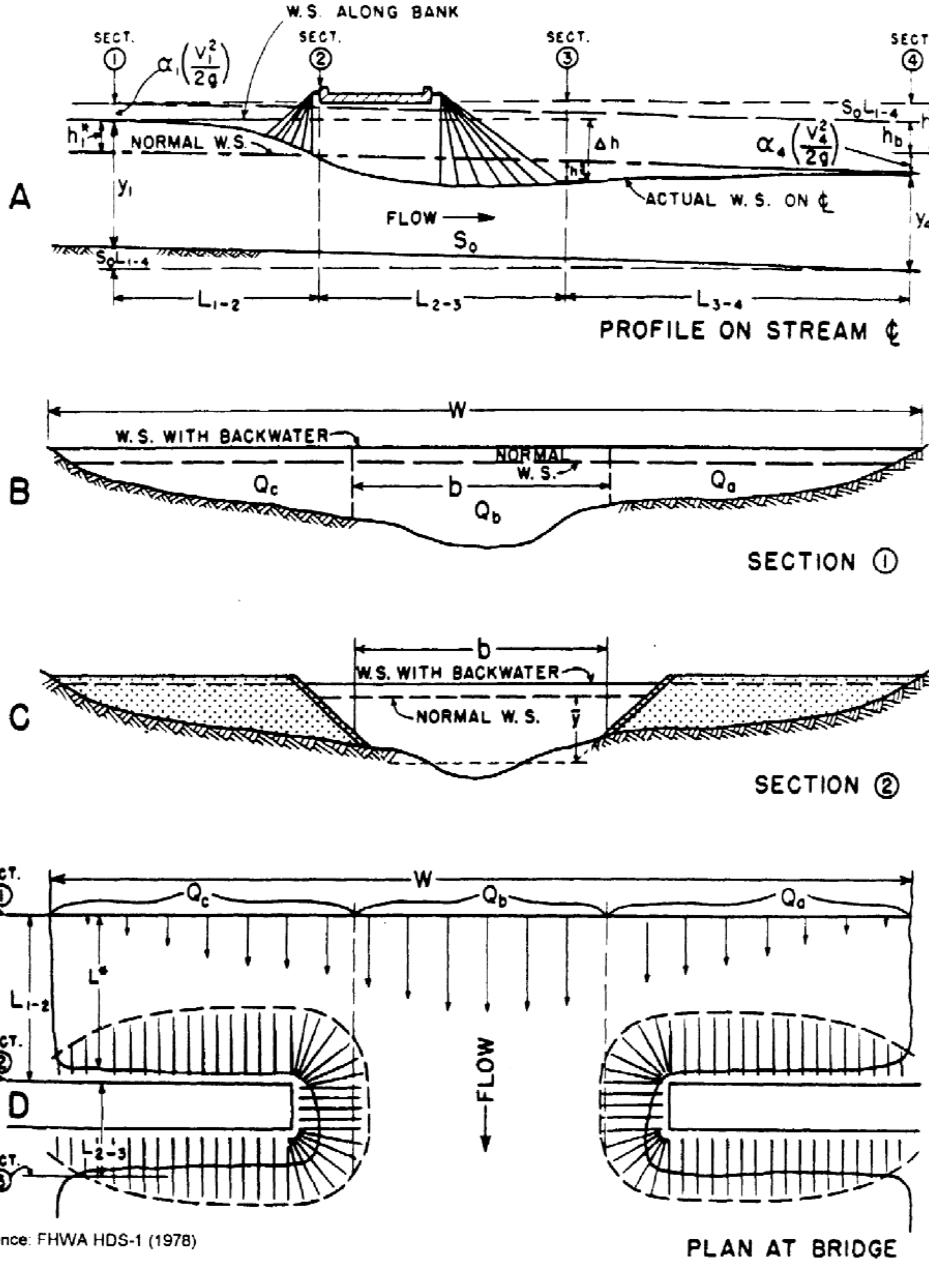
Constriction of the flow causes a loss of energy, with the greater portion occurring in the re-expansion downstream. This loss of energy is reflected in a rise in the water surface and in the energy line upstream of the bridge. This is best illustrated by a profile along the center of the stream, as shown in Figure A-2 (Part A). The normal stage of the stream for a given discharge, before constricting the channel, is represented by the dashed line labeled "normal water surface". The nature of the water surface after constriction of the channel is represented by the solid line "actual water surface". Note that the water surface starts out above normal stage at Section 1, passes through normal stage close to Section 2, reaches minimum depth in the vicinity of Section 3, and then returns to normal stage a considerable distance downstream, at Section 4. Determination of the rise in water surface at Section 1 is denoted by the symbol h_1* and referred to as the <u>bridge backwater</u>.



NOTE: L* = B (DEFINED AS ONE BRIDGE LENGTH IN WSPRO)

Reference: USDOT, FHWA HDS-1 (1978)

Figure A-1 Flow Lines for Typical Bridge Crossing



Reference: FHWA HDS-1 (1978)

Figure A-2 Normal Crossings: Spill-through Abutments

A.2 Conveyance

Conveyance is a measure of the ability of a channel to transport flow. In streams of irregular cross section, it is necessary to divide the water area into smaller but more or less regular subsections, assigning an appropriate roughness coefficient to each and computing the discharge for each subsection separately. By rearranging the Manning's Equation, the following relationship is derived:

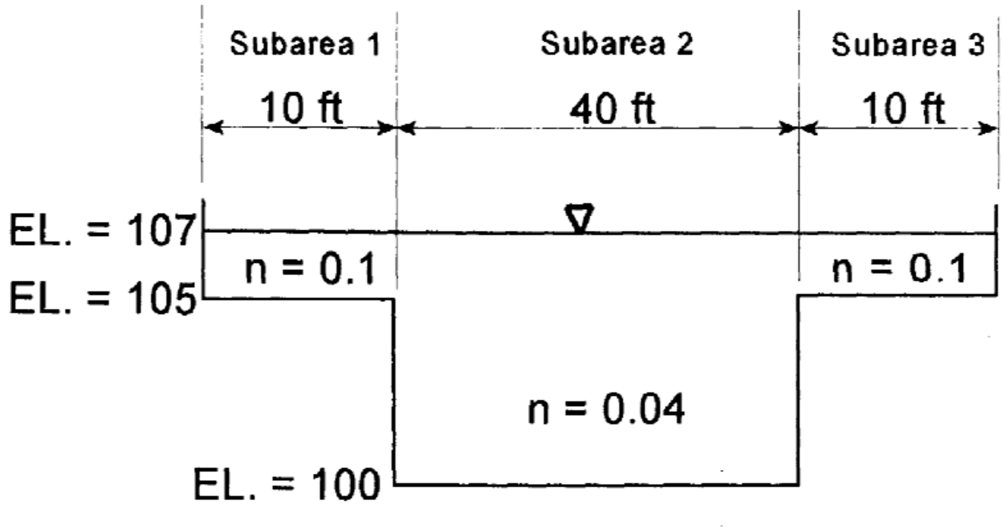
$$k = \frac{q}{S^{1/2}} = \frac{1.49}{n} ar^{2/3} \qquad \text{Equation 1}$$

where:

- k = Channel subsection conveyance
- q = Subsection discharge, in cubic feet per second
- S = Channel bottom slope, feet/feet
- n = Manning's roughness coefficient
- a = Subsection cross-sectional area, in square feet
- r = Subsection hydraulic radius, in feet

Conveyance can therefore, be expressed either in terms of flow factors or strictly geometric factors. In bridge waterway computations, conveyance is used as a means of approximating the distribution of flow in the natural river upstream from a bridge. Total conveyance (K) is the summation of the individual conveyances comprising the particular section. Example A-1 illustrates a conveyance computation.

Example A-1 - Computing Conveyance



a. Compute the conveyance for the cross section shown above.

Solution:

Step 1: Compute the area, hydraulic radius and conveyance for each of the subareas:

Subarea 1:

a_1 = 10 ft. * 2 ft. = 20 ft^2
wp_1 = 10 ft. + 2 ft. = 12 ft.
r_1 = a_1/wp_1 = 20 ft^2/12 ft. = 1.67 ft.

$$k_1 = \frac{1.49}{n_1} a_1 r_1^{2/3} = \frac{1.49}{0.1}(20\ ft.^2)(1.67\ ft.)^{2/3} = 419.5$$

Subarea 2:

a_2 = 40 ft. * 7 ft. = 280 ft^2
wp_2 = 40 ft. + 5 ft. + 5 ft. = 50 ft.
r_2 = a_2/wp_2 = 280 ft^2/50 ft. = 5.60 ft.

$$k_2 = \frac{1.49}{n_2} a_2 r_2^{2/3} = \frac{1.49}{0.04}(280\ ft.^2)(5.60\ ft.)^{2/3} = 32890.9$$

Subarea 3:

a_3 = 10 ft. * 2 ft. = 20 ft^2
wp_3 = 10 ft. + 2 ft. = 12 ft.
r_3 = a_3/wp_3 = 20 ft^2/12 ft. =1.67 ft.

$$k_3 = \frac{1.49}{n_3} a_3 r_3^{2/3} = \frac{1.49}{0.1}(20\ ft.^2)(1.67\ ft.)^{2/3} = 419.5$$

Total Conveyance (K_{total}) = $k_1 + k_2 + k_3$
= 419.5 + 32890.9 + 419.5
= <u>33729.9</u>

b. Assuming the total discharge for the water surface elevation of 107.0 feet in part (a) is 4000 cubic feet per second, determine the discharge distribution for each subarea.

Solution:

Subarea 1:

$$Q_1 = \frac{k_1}{k_{total}} * Q_{total} = (\frac{419.5}{33729.9}) * 4000\ ft^3/s = 49.8\ ft^3/s$$

Subarea 2:

$$Q_2 = \frac{k_2}{k_{total}} * Q_{total} = (\frac{32890.9}{33729.9}) * 4000\ ft^3/s = 3900.5\ ft^3/s$$

Subarea 3:

$$Q_3 = \frac{k_3}{k_{total}} * Q_{total} = (\frac{419.5}{33729.9}) * 4000\ ft^3/s = 49.8\ ft^3/s$$

A.3 Velocity Head

The velocity head represents the kinetic energy of the fluid per unit volume and is computed by:

$$h_v = \frac{\propto Q^2}{2g\ A^2} \qquad \text{Equation 2}$$

where:

- Q = Discharge at the section in cubic feet per second
- h_V = Velocity Head, feet.
- $\propto$ = Kinetic correction factor for nonuniform velocity distribution
- A = Total cross sectional flow area in square feet

As the velocity distribution in a river varies from a maximum at the deeper portion of the channel to essentially zero along banks, the average velocity head, computed as $(Q/A_1)^2/2g$ for the stream at Section 1 of Figure A-1, does not a give a true measure of the kinetic energy of the flow. A weighted average value of the kinetic energy is obtained by multiplying the average velocity head above by a kinetic energy coefficient ($\propto_1$) defined as:

$$\propto_1 = \frac{\Sigma\,(qv^2)}{Qv_1^2} \qquad \text{Equation 3}$$

where:

- $\propto_1$ = Kinetic energy coefficient, before the bridge
- Q = Discharge in same subsection, in cubic feet per second
- v = Average velocity in a subsection, in feet per second
- Q = Total river discharge, in cubic feet per second
- v_1 = Average velocity in river at Section 1, or Q/A_1, in feet per second

Typical values of velocity coefficient, α, are shown in Table A-1:

Table A-1 Typical Values of Velocity Coefficient

Channel Types	Value of α		
	Min.	Avg.	Max.
Regular Channels, Flumes, and Spillways	1.1	1.15	1.2
Natural Streams	1.15	1.30	1.5
River Valleys, Overflooded	1.5	1.75	2.0

From Chow, Open Channel Hydraulics

Additional guidelines on velocity coefficients can be found in the Techniques of Water-Resource Investigations (TWRI) Reports of the United States Geological Survey.

In general, the more subdivisions in a cross section the higher the alpha (α) value.

A.4 Friction Losses

The friction loss is computed as:

$$h_f = L\, S_f \qquad \text{Equation 4}$$

where:

L = Flow length in feet
S_f = Average friction slope in feet/feet

The average friction slope can be calculated using either the geometric mean slope method, the average conveyance method, the average friction slope method, or the harmonic mean friction slope method. WSPRO uses the geometric mean slope method as the default option. The geometric mean slope is computed as:

$$S_f = \frac{[0.5\,(Q_1 + Q_2)]^2}{K_1 K_2} \qquad \text{Equation 5}$$

where:

S_f = Average friction slope in feet/feet
Q_1 = Discharge at Section 1 in cubic feet per second
Q_2 = Discharge at Section 2 in cubic feet per second
K_1 = Conveyance at Section 1
K_2 = Conveyance at Section 2

A.5 Expansion/Contraction Losses

Expansion Losses

The expansion loss is computed as:

$$h_e = k_e (h_{v2} - h_{v1})$$ Equation 6

where:

k_c = Expansion loss coefficient
h_{V1} = Velocity Head in Section 1 in feet
h_{V2} = Velocity Head in Section 2 in feet

The expansion loss coefficient varies from 0.0 to 1.0 from ideal transitions to abrupt transitions. HEC-RAS uses an expansion value of 0.3 as its default. WSPRO uses an expansion value of 0.5 as its default. Brater and King's Handbook of Hydraulics provides additional guidance for selection of expansion coefficients.

Contraction Losses

The contraction loss is computed as:

$$h_c = k_c (h_{v2} - h_{v1})$$ Equation 7

where:

k_c = Contraction loss coefficient
h_{V1} = Velocity Head in Section 1 in feet
h_{V2} = Velocity Head in Section 2 in feet

The contraction loss coefficient varies from 0.0 to 0.5 from ideal transitions to abrupt transitions. HEC-RAS uses a contraction value of 0.1 as its default. WSPRO uses a contraction value of 0.0 as its default. Brater and King's Handbook of Hydraulics provides additional guidance for selection of contraction coefficients.

A.6 Step Backwater Computations

HEC-RAS and WSPRO computational procedure employs the Standard Step Method for profile computations. The procedure used is similar to that described by Chow. The standard step method is based on the principle of conservation of energy, i.e., the total energy head at an upstream section must equal to the total energy head at the downstream section plus any energy losses that occur between the two sections.

Energy Equation

The energy equation between two adjacent cross sections may be written:

$$h_1 + h_{v1} = h_2 + h_{v2} + h_f + h_e + h_c$$ Equation 8

where:

h_1 = Water Surface Elevation in Section 1 in feet

h_{V1} = Velocity Head in Section 1 in feet
h_2 = Water Surface Elevation in Section 2 in feet
h_{V2} = Velocity Head in Section 2 in feet
h_f = Friction Loss between Sections 1 and 2 in feet
h_e = Expansion Loss between Sections 1 and 2 in feet
h_c = Contraction Loss between Sections 1 and 2 in feet

A direct solution of Equation 8 is not possible when either h_1 or h_2 is unknown, since the associated velocity head and the energy loss terms are then also unknown. Therefore, an iterative procedure must be used to determine the unknown elevation. The WSPRO model computes the difference in total energy between two sections, H, as

$$\Delta H = (h_1 + h_{v1}) - (h_2 + h_{v2} + h_f + h_e + h_c) \qquad \text{Equation 9}$$

Successive estimates of unknown elevations are used to compute the unknown velocity head and the energy loss terms until the equation yields an absolute value of ΔH that is within an acceptable tolerance. Generally a tolerance between 0.01 and 0.05 is sufficient to obtain satisfactory results. Slightly higher results may be satisfactory for some higher velocity situations. However, if a tolerance value exceeding 0.1 is required to obtain a satisfactory solution, then there would be reason to suspect data inadequacies (example: insufficient cross sections).

Computational Procedure:

Given: Discharge Q and WSE at one cross section and the fact that the flow is subcritical. We want to compute the WSE at the next upstream cross section.

Step 1: Calculate all the geometric and hydraulic properties of the downstream most station using the known flows and WSE at that location.

Step 2: Estimate water surface elevation at the next upstream station.

Step 3: Calculate hydraulic properties that correspond to estimated water surface elevation.

Step 4: Determine energy losses that correspond to estimated water surface elevation.

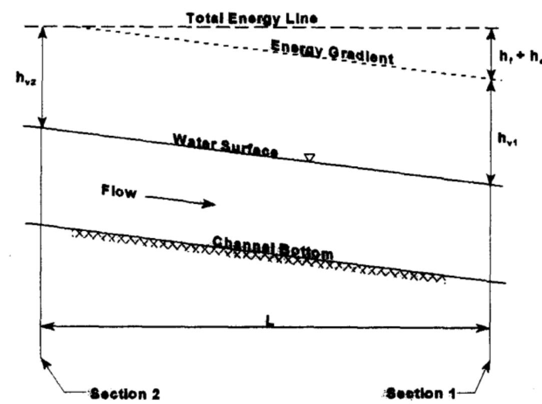
Step 5: Calculate water surface elevation using energy equation and energy losses determined in Step 4.

Step 6: Compare estimated and computed water surface elevations.

Step 7: If the computed and estimated elevations do not agree within some predetermined limit of error, try another value and start the procedure again beginning with Step 2.

Example A-2 illustrates a step backwater computation.

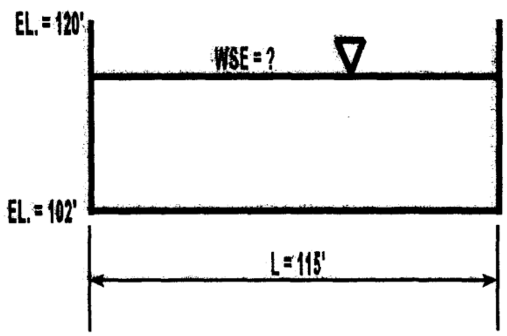
EXAMPLE 2 - Standard Step backwater Computation



GIVEN:

Cross Section (Section 2) at River Station 15+00

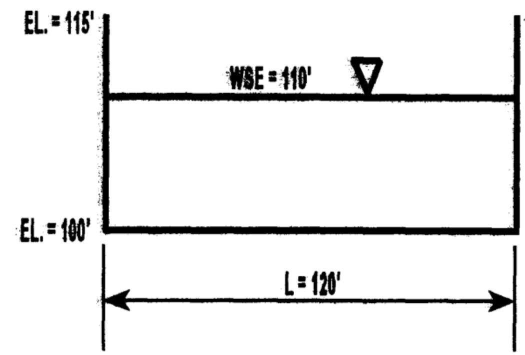
Q = 11,000
N = 0.025
WSE = ?
C_e = 0.5
C_c = 0.0



GIVEN:

Cross Section (Section 1) at River Station 2+00

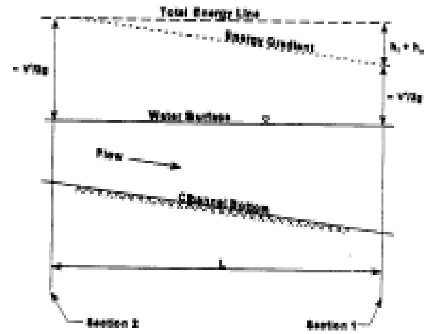
Q = 11,000
N = 0.02
WSE = 110'



EXAMPLE 2 - Standard Step backwater Computation (Cont.)

Q = 11,000 cfs

C_c = 0.0 C_e = 0.5



Cross Section No.	Water Surface Elevation		Area (A)	Hydraulic Radius (R)	$R^{2/3}$	n	K	$\overline{S}_f$	L	h_f	K^3/A^2 (10^6)	ALPHA (α)	V	$\alpha \frac{V^2}{2g}$	$\Delta(\alpha \frac{V^2}{2g})$	h_o	Δ (Water Surface Elevation)
	Assumed	Computed															
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
2+00	110.0		1200	8.57	4.19	0.020	374,586				36.5	1.00	9.17	1.31			
15+00	110.8	111.53	1012	7.63	3.87	0.025	233,420	0.00138	1300	1.79	12.4	1.00	10.87	1.83	-0.52	0.26	1.53
15+00	111.5	111.46	1093	8.16	4.05	0.025	263,828	0.00122	1300	1.59	15.3	1.00	10.06	1.57	-0.26	0.13	1.46

(8) $K = \dfrac{1.49\,A\,R^{2/3}}{n}$

(9) $\overline{S}_f = \dfrac{[0.5\,(Q_1 + Q_2)]^2}{K_1 K_2}$

(11) $h_f = L\,\overline{S}_f$

(13) $\alpha = \dfrac{(A_T)^2\,\Sigma\,(K_I^3 / A_I^2)}{(K_T)^3}$

Where: I = incremental value
T = total value

(14) $V = Q / A_T$

(16) $\Delta(\alpha\, V^2/2g) = (\alpha\, V^2/2g)$ downstream - $(\alpha\, V^2/2g)$ upstream

(17a) $h_o = C_e \left|\Delta(\alpha\, V^2/2g)\right|$ for $\Delta(\alpha\, V^2/2g) < 0$

(17b) $h_o = C_c \left|\Delta(\alpha\, V^2/2g)\right|$ for $\Delta(\alpha\, V^2/2g) > 0$

(18) Δ(water surface elevation) = $\Delta(\alpha\, V^2/2g) + h_f + h_o$ = Columns (16) + (11) + (17)

A.7 Tidal Bridge Scour Glossary

Accretion	Buildup of land or bottom elevation.
Bay	A recess in the shore or an inlet of a sea between two capes or headlands, not as large as a gulf but larger than a cove.
Diurnal Tide	One high tide and one low tide per day.
Ebb Phase	The period when the tide level is falling.
Estuary	Body of water affected by tidal influence as well as freshwater inflows from a riverine system.
Flood Phase	The period when the tide level is rising.
Hindcast	To retrospectively employ measured data to develop a model wind or wave field of a specific historical event.
Inlet	A short, narrow waterway connecting a bay, lagoon, or similar body of water with a large parent body of water.
Mean Higher High Water (MHHW)*	The average of the higher high water height of each tidal day observed over the National Tidal Datum Epoch. For stations with shorter series, comparison of simultaneous observations with a control tide station is made in order to derive the equivalent datum of the National Tidal Datum Epoch.
Mean High Water (MHW)	The average height of high waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. All high water heights are included in the average where the type of tide is semi-diurnal or mixed. Only the higher high water heights are included in the average where the type of tide is diurnal. So determined, mean high water in the latter case is the same as mean higher high water.
Mean Lower Low Water (MLLW)*	The average of the lower low water height of each tidal day observed over the National Tidal Datum Epoch. For stations with shorter series, comparison of simultaneous observations with a control tide station is made in order to derive the equivalent datum of the National Tidal Datum Epoch.
Mean Low Water	The average height of the low waters over a 19-year period. For

(MLW)	shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. All low water heights are included in the average where the type of tide is either semi-diurnal or mixed. Only lower low water heights are included in the average where the type of tide is diurnal. So determined, mean low water in the latter case is the same as mean lower low water.
Mean Sea Level (MSL)	The arithmetic mean of hourly heights observed over the National Tidal Datum Epoch. Shorter series are specified in the name; i.e. monthly mean sea level and yearly mean sea level.
Mean Tide Level (MTL)	The arithmetic mean of mean high water and mean low water.
National Tidal Datum Epoch (NTDE)	The specific 19-year period adopted by the National Ocean Service as the official time segment over which tide observations are taken and reduced to obtain mean values (e.g., mean lower low water, etc.) for tidal datums. It is necessary for standardization because of periodic and apparent secular trends in sea level. The present NTDE is 1983 through 2001 and is actively considered for revision every 20-25 years. Tidal datums in certain regions with anomalous sea level changes (Alaska, Gulf of Mexico) are calculated on a Modified 5-Year Epoch.
Neap Tide	Tide of decreased range occurring semimonthly as the result of the moon being in quadrature.
Semi-Diurnal Tide	Two high tides and two low tides per day.
Significant Wave Height	The average height of the one-third highest waves of a given wave group. Note that the composition of the highest waves depends upon the extent to which to lower waves are considered.
Storm Surge	Long wave generated offshore that may propagate into coastal bays/estuaries, the five components of storm surge are wind setup, atmospheric pressure setup, Coriolis effect, wave setup, and the rainfall effect.
Spit	A small point of land or a narrow shoal projecting into a body of water from the shore.
Spring Tide	A tide that occurs at or near the time of new or full moon and which rises highest and falls lowest from the mean sea level.

Swell	Wind-generated waves that of traveled out of their generating area. Swell characteristically exhibits a more regular and longer period and has flatter crests than waves within their fetch.
Thalweg	In hydraulics, the line joining the deepest points of an inlet or channel.
Wave Height	The vertical distance between a wave's crest and the preceding trough.
Wave Radiation Stress	Excess flow of momentum in the horizontal plane due to waves.
Wave Runup	The vertical distance above the still water level that breaking waves propel water up a sloping surface.
Wave Setup	Vertical increase in the water surface above the still water level near shore due to onshore mass transport of water due to wave radiation stresses.
Wave Shoaling	Transformation of wave profile due to inshore propagation.
Wind Setup	The vertical rise above the still water level on the leeward side of a water body due to wind stresses on the surface of the water.
Wind Set-Down	The vertical drop below the still water level on the windward side of a water body due to wind stresses on the surface of the water.
Wind Wave	Waves being formed and built up by the wind.

* For locations with diurnal tides – one high tide and one low tide per day – this datum will be unavailable. At most locations, there are semi-diurnal tides – the tide cycles through a high and low twice each day, with one of the two high tides being higher than the other and one of the two low tides being lower than the other.

A.8 Tidal Bench Marks

Tidal datums are vertical elevations that describe the tidal fluctuation at a particular location. Several tidal datums are in common use including mean high water, which is the base elevation for structure heights, bridge clearances, etc., and mean low water (MLW), which is the officially designated navigational chart datum for the United States and its territories. In order that they may be recovered when needed, such datums are referenced to fixed points known as bench marks. NOAA maintains numerous tidal bench marks throughout the State of Florida which are available from the Center for Operational Oceanographic Products and Services (COOPS) website (http://www.co-ops.nos.noaa.gov/). The Florida Department of Environmental Protection (FDEP) is an additional source of this information. The FDEP website LABINS (Land Boundary Information System) contains a water boundary data map interface that lists not only the MLW and MHW at the NOAA bench mark locations, but also these datums at interpolated locations along interior tidal waterways. The LABINS website information (http://www.labins.org/survey_data/water/water.cfm) is recommended for locations where NOAA tidal bench marks are either unavailable or display a wide range of vertical variation around the project location.

Several other tidal datums are available and should be documented for each tidally controlled or influenced project.

The east coast of Florida experiences semi-diurnal tides and the panhandle experiences diurnal tides. The coastline from the tip of the peninsula to Apalachicola experiences mixed tides – tides characterized by a conspicuous diurnal inequality in the higher high and lower high waters and/or higher low and lower low waters. Figure A-3 and Table A-2 below display an example of tidal bench mark information and gage data (with tidal datums) for Key West, FL.

Table A-2 Elevations of tidal datums in ft-NAVD88 for NOAA tidal bench mark #8724580 (Key West, FL) for the 1983-2001 tidal epoch

MEAN HIGHER HIGH WATER (MHHW)	+0.05
MEAN HIGH WATER (MHW)	-0.24
MEAN TIDE LEVEL (MTL)	-0.88
MEAN SEA LEVEL (MSL)	-0.87
MEAN LOW WATER (MLW)	-1.52
MEAN LOWER LOW WATER (MLLW)	-1.76

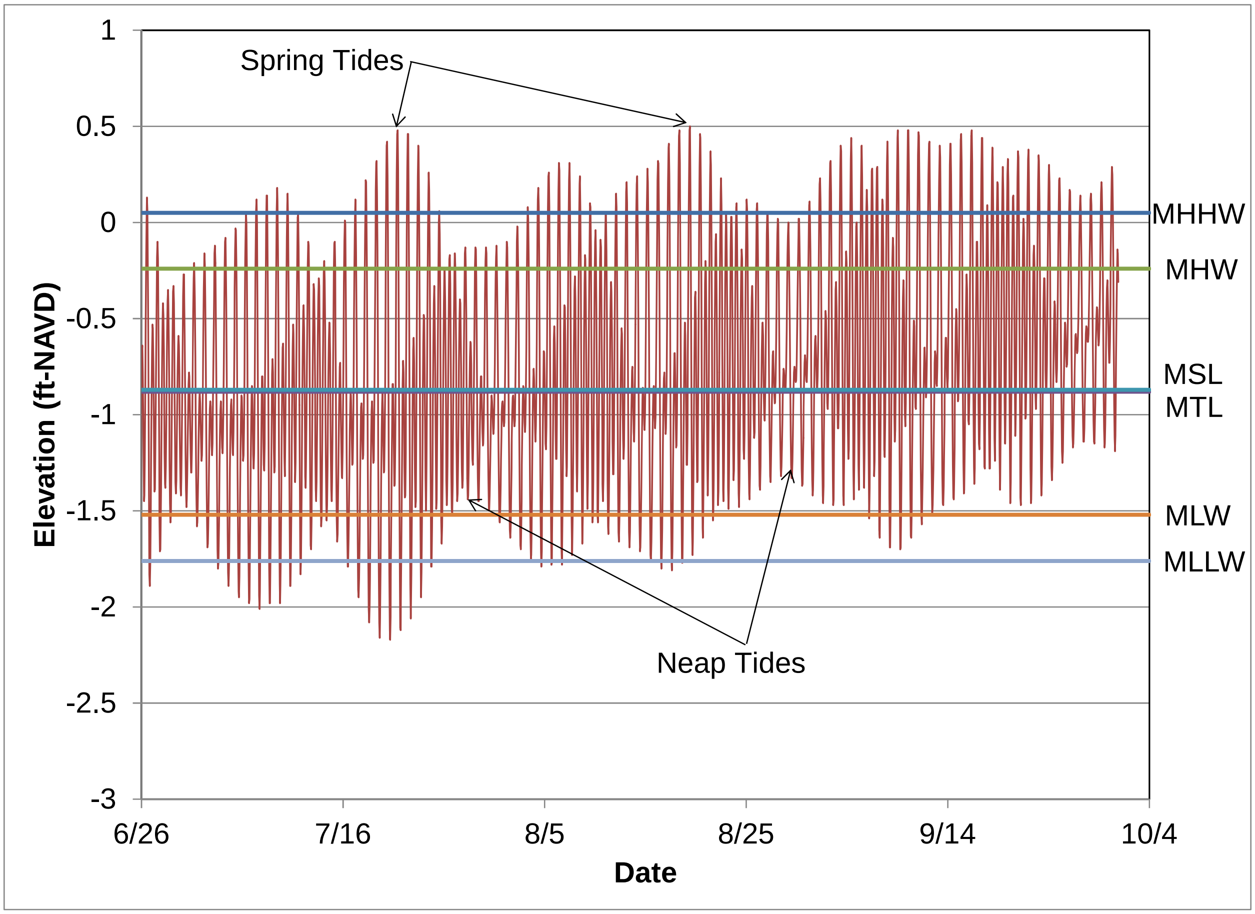


Figure A-3 Measured Tides at Key West and Tidal Datums

Appendix B

Risk Evaluations

B.1 Risk Evaluation

All designs with floodplain encroachments should include an evaluation of the inherent flood-related risks to the highway facility and to the surrounding property. In the traditional design process, the level of risk is seldom quantified, but is instead implied through the application of predetermined design standards. For example, the design frequency, backwater limitations, and limiting velocity are parameters for which design standards can be set.

Two other approaches, however, are available that quantify risk on projects that involve highway facilities designed to encroach within the limits of a floodplain. These are risk assessment and economic analysis.

Consideration of capital costs and the risks should include, as appropriate, a risk analysis or risk assessment which includes:

The overtopping flood or the base flood, whichever is greater, or the greatest flood which must pass through the highway drainage structure(s), where overtopping is not practicable.

B.1.1 Risk Assessment

A risk assessment is a subjective analysis of the risks engendered by various design alternatives, without detailed quantification of flood risks and losses. It may consist of developing the construction costs for each alternative, and subjectively comparing the risks associated with each alternative. A risk assessment is usually more appropriate for small structures, or for structures whose size is highly influenced by non-hydraulic constraints. There are no well-defined procedures or criteria for performing risk assessments. However, an attempt should be made to screen projects and determine the level of analysis required. Some of the items to consider:

- Backwater
    a. Is the overtopping flood greater than the design flood (100-year)?
    b. Is the overtopping flood greater than the check flood (500-year)?
    c. Is there potential for major flood damage for the overtopping flood?
    d. Could flood damage occur even if the roadway crossing wasn't there?
    e. Could flood damage be significantly increased by the backwater caused by the proposed structure?
    f. Could flood damage occur to offsite property owners?
- Traffic Related Losses
    a. If the design flood is exceeded and the roadway is overtopped, is there a detour available?

- Roadway and/or Structure Repair Costs
  a. Is the overtopping flood greater or less than a design (100-year) flood?
  b. Is the embankment constructed from erosion resistant material such as a clay type soil?
  c. Does the embankment have good erosion resistant vegetation cover?
  d. How long will the duration of overtopping be?
  e. Will the cost of protecting the roadway and/or structure from damage exceed the cost of providing a relief structure?
  f. Is there damage potential to the structure caused by scour, debris, or other means during the lesser of the overtopping flood or the design (100-year) flood?

If the risk assessment indicates the risks warrant additional study, a detailed analysis of alternative designs (economic analysis) is necessary in order to determine the design with the least total expected cost (LTEC) to the public.

B.1.2 Economic Analysis

An economic analysis (sometimes called risk analysis) encompasses a complete evaluation of all quantifiable flood losses and the costs associated with them for each structure alternative. This can include damage to structures, embankments, surrounding property, traffic-related losses, and scour or stream channel change.
The level of expense and effort required for an economic analysis is considerably higher than for a risk assessment, and selection of the process to be used should be based on the size of the project and the potential risk involved.

Further details of the economic analysis process and procedures for using it have been documented in HEC-17 (USDOT, FHWA, 1981). The full-scale detailed risk analysis described in HEC-17 would not be necessary for normal stream crossings, but would apply to unusual, complex, or high-cost encroachments involving substantial flood losses.

An example of a simple risk analysis follows in Example B.1.

Example B.1 Sample Risk Analysis

<u>Alternates considered:</u>

Alternate 1: Extend existing double 10' x 4' CBC (60' total length) with no change to road. Overtops at about a 17 year frequency; flooding at the site has not caused any accidents.

Alternate 2: New quad 10' x 5' CBC (60' total length). Raise road to meet FDOT 50 year HW criteria and closely match existing 100 year HW. Overtops at frequencies greater than 50 year.

Alternate 3: Bridge

	Alternate 1	Alternate 2	Alternate 3
Annual Capital Costs $ (i.e. Construction Costs)			
Annual Risks Costs $			
Total Costs $			

Calculations for Alternate 1:

Capital Costs (Quantities from the Department's Culvert Design Program)

Extend 20' right	Concrete	43.1 CY	Steel	6622 lbs
Extend 8' left		23.5 CY		3283 lbs
Total quantity	Concrete	66.6 CY	Steel	9905 lbs

Unit prices $477 / CY $0.53 / lb

Total capital cost = $37,018 = $31,768 + $5,250

To convert to annual capital cost use capital recovery factor (CRF) based on a discount rate of 7% and a 20-year design life.

$$CRF = \frac{i}{1-(1+i)^{-n}}$$ where: n = 20 and I = 0.07

Annual capital costs = $37,018 x 0.0944 = $3,494

Additional Economic Costs

The following is an estimation of the additional losses associated with extending the existing culvert and allowing the road to overtop. The losses usually consist of embankment (and pavement), backwater, and traffic.

There are not expected to be any embankment losses. The existing road and culvert overtop and there is no history of embankment or pavement loss.

There will not be any additional backwater losses compared to Alternate 2. Both Alternate 1 and Alternate 2 have essentially the same backwater characteristics.

There may be additional traffic losses associated with Alternate 1 when compared with Alternate 2, which would raise the road to reduce overtopping potential. Traffic related costs consist of running time costs, lost time costs, and accident costs. Running time costs were estimated, lost time costs were ignored (detour length added only 1 mile to the travel distance), and accident costs were estimated but were found to be insignificant.

Assume traffic would have to be detoured:

1 day for 25-year storm (roadway tops at about a 17 year event)
2 days for 50-year storm
3 days for 100-year storm
4 days for 200-year storm

The additional detour distance = 0.5 mile on 2-lane undivided and 0.5 mile on 4-lane divided.

<u>Additional running costs</u> = Cost per mile x ADT x additional detour length (miles)
Assume cost per mile = $0.35 / mile

$\$_{25\ yr}$ = $0.35 x 27250 vpd x 1.0 mi x 1 day = $9,538

$\$_{50\ yr}$ = $0.35 x 27250 vpd x 1.0 mi x 2 days = $19,075

$\$_{100\ yr}$ = $0.35 x 27250 vpd x 1.0 mi x 3 days = $28,615

$\$_{200\ yr}$ = $0.35 x 27250 vpd x 1.0 mi x 4 days = $38,150

<u>Additional accident costs</u>: these are additional costs due to increased travel distance due to detour.

Additional detour length is 0.5 mi on 2-lane undivided and 0.5 mi on 4-lane divided.

Accident cost = crash rate x vehicle miles x cost per crash

Vehicle miles = ADT x additional detour distance x number of days of detour

Get the crash rate and the cost per crash from the FDOT Safety Office.

Crash rate = 1.9 crashes / million vehicle miles for urban 2-lane, undivided; 0.8 crashes / million vehicle miles for urban 4-lane, divided

Cost per crash = $28,000 for urban 2-lane, undivided
$26,000 for urban 4-lane divided

$\$_{25}$ = (\$28,000 x [27,250 x 0.5 x 1] x 1.9) + (\$26,000 x [27,250 x 0.5 x 1] x 0.8)

$\$_{25}$ = \$1008.25

Using same method, with 50 year detour = 2 days, 100 year detour = 3 days, and 100-year detour = 4 days:

$\$_{50}$ = \$2016.50

$\$_{100}$ = \$3024.75

$\$_{200}$ = \$4033.00

Traffic losses in the following table are the sum of increased running costs and increased accident losses.

Summary of Economic Losses				
Frequency (yr)	Losses ($)			Total Losses ($)
	Embankment & Pavement	Backwater	Traffic	
5	0	0	0	0
10	0	0	0	0
15	0	0	0	0
25	0	0	9538 + 1008.25 = 10,546.25	10, 546.25
50	0	0	21,091.50	21,091.50
100	0	0	31,639.75	31,639.75
200	0	0	42,183.00	42,183.00

Summary of Annual Risk Costs					
Freq. (yr)	Exceed. Prob.	Losses ($)	Average Loss ($)	Delta Prob.	Annual Risk Costs ($)
5	0.2	0			
10	0.1	0			
15	0.07	0			
			5,273.13	0.03	158.19
25	0.04	10,546.25			
			15,818.88	0.02	316.38
50	0.02	21,091.50			
			26,365.63	0.01	263.66
100	0.01	31,639.75			
			36,911.38	0.005	184.56
200	0.005	42,183.00			
			42,183.00	0.005	210.92
	0	42,183.00			
Total Annual Risk Costs					1,133.71

	Alternate 1	Alternate 2	Alternate 3
Annual Capital Costs $ (i.e. Construction Costs)	3,494		
Annual Risks Costs $	1,134		
Total Costs $	4,628		

Alternate 2: Replace with quad 10' x 5' CBC

Capital Costs include:

Concrete (from box culvert program) = 219.7 cy @ $477/cy = $104,797
Steel (from box culvert program) = 42,251# @ $0.53/# = $22,393

Rebuild 400' of Roadway

Structural Course (2' x 24') = 1067 sy @ $3.40/sy = $3,628
Base group 9 = 1067 sy @ $6.16/sy = $6,573
Neglect earthwork costs

Total Capital Costs = $137,391

Annual Capital Cost = Total x CRF = $12,970

This alternate would overtop at frequencies greater than 50-year and would, therefore, have some annual risk costs. These were not calculated because the annual cost alone is greater than the total cost for Alternate 1. If the capital costs for Alternate 2 were less than the total cost for Alternate 1, it would be necessary to calculate the other costs associated with this alternate.

	Alternate 1	Alternate 2	Alternate 3
Annual Capital Costs $ (i.e. Construction Costs)	3,494	12,970	
Annual Risks Costs $	1,134	>0	
Total Costs $	4,628	>12,970	

Alternate 3: 57’ long x 44’ wide flat slab bridge

Capital Costs:

- 57’ x 44’ x $40 / sf = 2508 sf x $40/sf = $100,320
- Annual cost using CRF = 0.0944 = $9,470

Costs not estimated:

Roadway fill and new base and asphalt. At a minimum 900’ of roadway would have to be rebuilt to raise the grade to meet the bridge. (Bridge would be raised to meet FDOT drift clearance requirements.)

Standard 1H:2V front slopes encroach into roadside ditches. Since the upstream roadside ditch conveys substantial flow, it may not be possible or wise to reduce its capacity. Vertical walls and/or additional right-of-way may be necessary.

Miscellaneous factors include driveway connections within the raised roadway section, and the aesthetics of the raised road and bridge.

	Alternate 1	Alternate 2	Alternate 3
Annual Capital Costs $ (i.e. Construction Costs)	3,494	12,970	9,740
Annual Risks Costs $	1,134	>0	>0
Total Costs $	4,628	>12,970	>9,740

Alternate 1 is the most economical alternate and the most desirable when considering other impacts.

Appendix C

Shoulder Gutter Transition Slope

<u>Slope Created by the Shoulder Gutter Transition:</u>
If the Profile Grade Line of the road is flat, there will be a slope away from the bridge created by the shoulder gutter transition. This slope will depend on the width of the shoulder and the cross slopes of the bridge deck and the roadway shoulder. Figure C.1 shows a transition with a 10' shoulder and standard cross slopes for the bridge deck and roadway shoulder.

The drop from the edge of the travel lane to the bottom of the gutter at the end of the bridge barrier wall is:

0.02 (10.33) = 0.206'

- 10.33 → Distance from edge of travel lane to bottom of gutter
- 0.02 → Shoulder cross slope

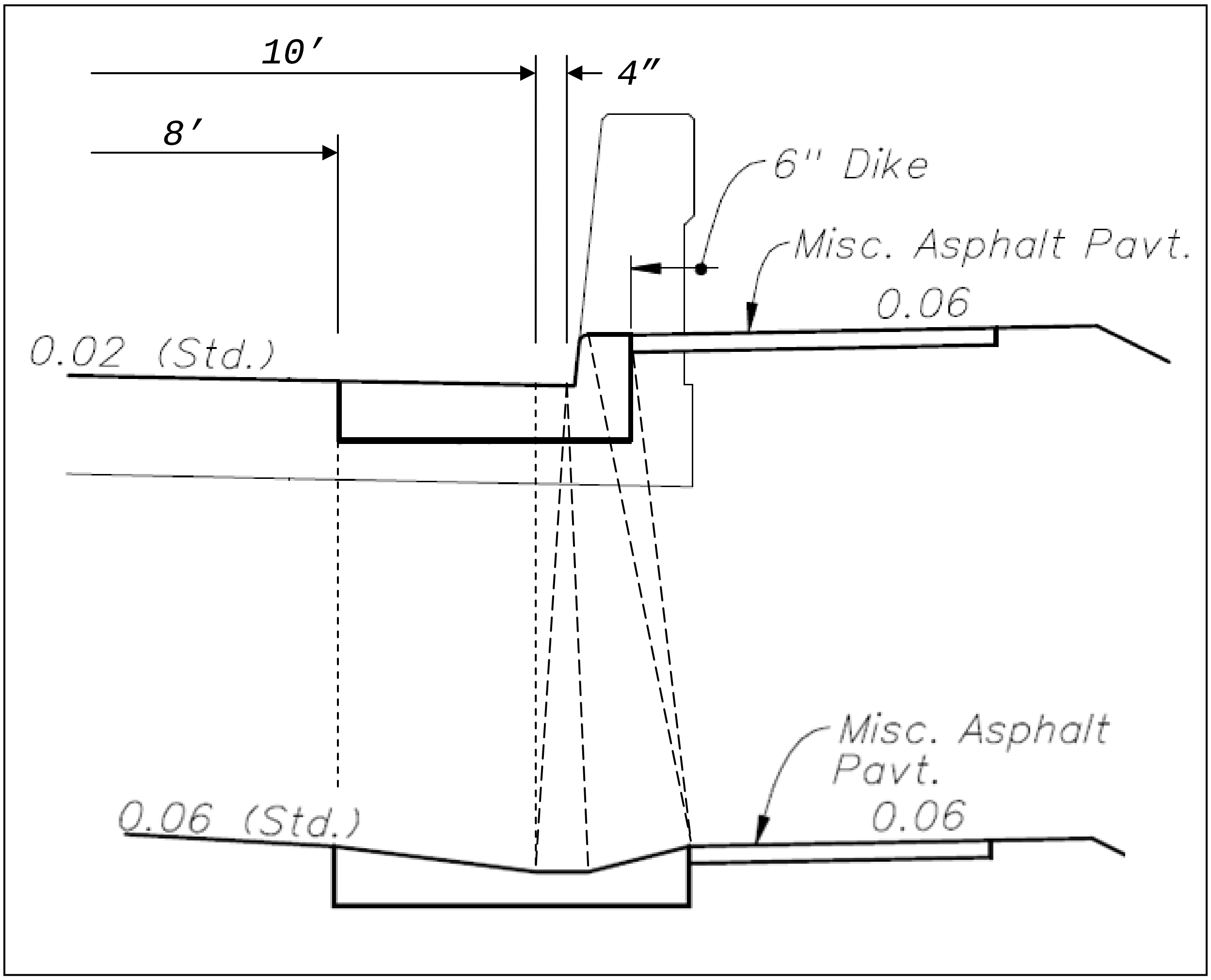


Figure C.1 Shoulder Gutter Transition at Bridge End

The drop from the edge of travel lane to the bottom of the gutter at the end of the transition is:

0.06 (8) + 0.25 = 0.730’

3” drop from lip of gutter to bottom of gutter
Distance from edge of travel lane to lip of gutter
Shoulder cross slope

The drop of gutter bottom in the transition is 0.730 – 0.206 = 0.524’. The length of the transition is 25’. The slope of the bottom of the gutter is 0.524 / 25 = 0.0210, or 2.10%.

Appendix D

Spreadsheet Solution of Example 7-6

Spreadsheet Solution of Example 7-6

Example 7-6 was evaluated using the simplifying, but conservative assumptions of equilibrium flow. If the design had failed to meet criteria under the conservative assumptions, then a more detailed analysis can be performed to evaluate the design. The following will illustrate the detailed analysis procedure and explain how a spreadsheet can be used to automate the analysis.

Rows 1 through 8 of the spreadsheet are shown below. The values in each of these cells can be entered as shown; i.e., none of these cells have formulae.

	A	B	C	D	E	F	G	H	I	J	K
1	Spacing =	20		Curve Data							
2	Sx =	0.02		G1 =	0						
3	n =	0.016		G2 =	0.005						
4	Width =	55		L =	240						
5											
6	Distance	Dr. Area	Flow	Slope	Spread	Int. Flow	Bypass			0.002	0.005
7	(feet)	(acres)	(cfs)	(ft./ft.)	(feet)	(cfs)	(cfs)				
8	0	0	0	0	0	0	0	Station 103+80			

Although the scupper spacing is 10', the spacing was entered as 20' to conservatively assume that every other scupper is clogged.

The vertical curve data is not entered in the same manner as listed on the Profile Sheets in the Plans or in Geopak. For the formulation in this spreadsheet, the peak of the vertical curve must be determined, and all distances referenced from the peak. The slopes must be entered so that the calculated slopes always have a positive value. G1 should be the slope at the uphill end, and G2 the slope at the downhill end.

The remaining rows will have formulae in some of the cells.

	A	B	C	D	E	F	G	H	I	J	K
9	20	0.025253	0.09596	0.000417	5.431296	0.05165	0.04431			0.05165	0.037082
10	40	0.025253	0.14027	0.000833	5.499062	0.05997	0.0803			0.05997	0.042832
11	60	0.025253	0.176259	0.00125	5.552224	0.063001	0.113258			0.063001	0.047151
12	80	0.025253	0.209218	0.001667	5.609947	0.066198	0.14302			0.066198	0.051383
13	100	0.025253	0.238979	0.002083	5.655207	0.069672	0.169307			0.070067	0.055847
14	120	0.025253	0.265267	0.0025	5.683265	0.071202	0.194064			0.073485	0.05979
15	140	0.025253	0.290024	0.002917	5.709225	0.07267	0.217354			0.076703	0.063504
16	160	0.025253	0.313314	0.003333	5.731697	0.073554	0.23976			0.079331	0.066331
17	180	0.025253	0.33572	0.00375	5.75362	0.073989	0.261731			0.081572	0.068572
18	200	0.025253	0.357691	0.004167	5.776778	0.07438	0.28331			0.083769	0.070769
19	220	0.025253	0.37927	0.004583	5.800493	0.074733	0.304537			0.085927	0.072927
20	240	0.025253	0.400497	0.005	5.824365	0.07505	0.325447			0.08801	0.07505

In Row 9, enter the following formulae in each Column:

Column A: =A8+$B$1

Column B: =(A9-A8)*$B$4/43560
Column C: =G8+0.95*4*B9
Column D: =($E$3-$E$2)*A9/$E$4+$E$2
Column E: =(C9*$B$3/0.56/$B$2^(5/3)/D9^0.5)^(3/8)
Column F: =IF(D9<0.002,J9,(IF(D9>0.005,K9,(J9+(K9-J9)*(D9-0.002)/0.003))))
Column G: =C9-F9
Column J: =IF(C9>1,Chart!$B$15,PERCENTILE(Chart!$B$4:$B$15,
PERCENTRANK(Chart!$A$4:$A$15,C9,20)))
Column K: =IF(C9>1,Chart!$E$15,PERCENTILE(Chart!$E$4:$E$15,
PERCENTRANK(Chart!$D$4:$D$15,C9,20)))

Column A keeps track of the distance from the upstream end.

Column B determines the drainage area between the current scupper and the previous scupper uphill. This spreadsheet assumes that the bridge has a constant width along the length of bridge being analyzed.

Column C determines the flow immediately upstream of the current scupper using the Rational Equation. The rainfall intensity is assumed to be 4 inches per hour and the Runoff Coefficient is assumed to be 0.95. The bypass from the previous scupper is combined with the runoff from the area between the scuppers.

Column D determines the slope of the Profile Grade at the current scupper.

Column E determines the spread using the Gutter Flow Equation.

Column F determines the intercepted flow rate based Figure 7-4. If the slope is less than 0.002, the curve labeled ‘0.2%’ is used. If the slope is greater than 0.005, the curve labeled ‘0.5, 1, 2%’ is used. If the slope is between 0.002 and 0.005, a value is interpolated between the two curves. Values for these two curves are determined in Columns J and K.

Column G determines the scupper bypass flow.

Columns J and K read the flows for the two curves of Figure 7-4. In the formulation of this spreadsheet, the curves are represented on another sheet named “Chart”. The values for chart are presented on the next page.

At the end of the vertical curve (or in this case, at the Begin Vertical Curve Station since the flow is in the opposite direction of the stationing) the Profile Grade slope becomes a constant value. The formula in Column D is change to constant of 0.005 as shown below.

	A	B	C	D	E	F	G	H	I	J	K
21	260	0.025253	0.421407	0.005	5.936589	0.077141	0.344266			0.088428	0.077141
22	280	0.025253	0.440226	0.005	6.034651	0.079023	0.361203			0.088805	0.079023
23	300	0.025253	0.457163	0.005	6.120691	0.080716	0.376447	Station 100+80		0.089143	0.080716
24	380	0.10101	0.760285	0.005	7.40696			Station 100+00			

The last scupper is at Station 100+80, which is 300 feet from the crest. The final row, Row 24 checks the spread at the edge of the Approach Slab. Since the spread at each scupper and at the edge of the Approach Slab is less than the shoulder width of 10', the design meets criteria.

As noted above, a separate sheet named "Chart" is included to represent the two curves in Figure 7-4. The values entered on "Chart" are shown below:

	A	B	C	D	E	F
1	Cross Slope = 0.02					
2	S = 0.002			S => 0.005		
3	Total	Intercepted		Total	Intercepted	
4	0	0		0	0	
5	0.056	0.028		0.056	0.028	
6	0.105	0.057		0.1	0.038	
7	0.2	0.065		0.2	0.05	
8	0.3	0.078		0.3	0.065	
9	0.4	0.088		0.4	0.075	
10	0.5	0.09		0.5	0.085	
11	0.6	0.095		0.6	0.095	
12	0.7	0.105		0.7	0.105	
13	0.8	0.12		0.8	0.12	
14	0.9	0.133		0.9	0.133	
15	1	0.148		1	0.148	

Appendix E

Chapter 3 Example Problems

Example 3-1

In Figure E-1, the section given has a constant n value for the entire cross section. The four calculations shown represent four methods of calculating total flow (conveyance) depending on how the cross section is subdivided.

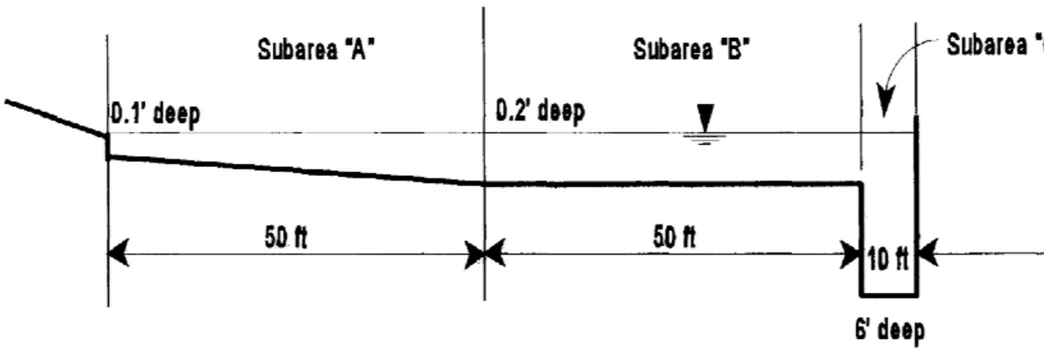


Figure E-1 Effects of Subdivision on a Panhandle Section

Given: $K = 1.49/n\,(AR^{2/3})$
n is constant over cross section
Factor out 1.49 and compare $AR^{2/3} = K'$.

Note: K' varies as to the number of sections selected as a function of R, or more specifically Wp.

(Method 1) Consider K_1' as one section encompassing subareas "A", "B", and "C".

$$K_1' = AR^{2/3}\,;\ K_1' = [(6x10) + (50x0.2) + (50x0.15)]\left[\frac{(6x10) + (50x0.2) + (50x0.15)}{(0.1 + 50 + 50 + 5.8 + 10 + 6)}\right]^{2/3} = \underline{57.3}$$

(Method 2) Consider K_2' as two sections, "A" and "B" combined and "C".

$$K_2' = [(6x10)(60/21.8)^{2/3}] + [(50x0.2) + (50x0.15)]\left[\frac{(50x0.2) + (50x0.15)}{100.1}\right]^{2/3} = 117.8 + 5.5 = \underline{123.3}$$

(Method 3) Consider K_3' as section "C" and ignore sections "A" and "B".

$$K_3' = (6\times 10)\left(\frac{60}{5.8 + 10 + 6}\right)^{2/3} = \underline{117.8}$$

(Method 4) Consider K_4' with "A","B", and "C" treated as independent sections.

$$K_4' = [(6 \times 10)(60/21.8)^{2/3}] + [(50 \times 0.2)(10/50)^{2/3}] + [(50 \times 0.15)(7.5/50.1)^{2/3}]$$
$$K_4' = 117.8 + 3.4 + 2.1 = \underline{123.3}$$

Method one is incorrect. The problem is the method neglects the impact the hydraulic radii of the shallow areas have on the overall flow calculation. This can be seen by looking at method three which shows conveyance in just the main channel as being greater. Two reasons why method one is incorrect are:

1. The total conveyance must be the sum of the conveyance of a channel's subsections.
2. Combining significantly different geometric sections of a cross section to simplify a calculation is a misuse of the conveyance equation and will yield an incorrect answer.

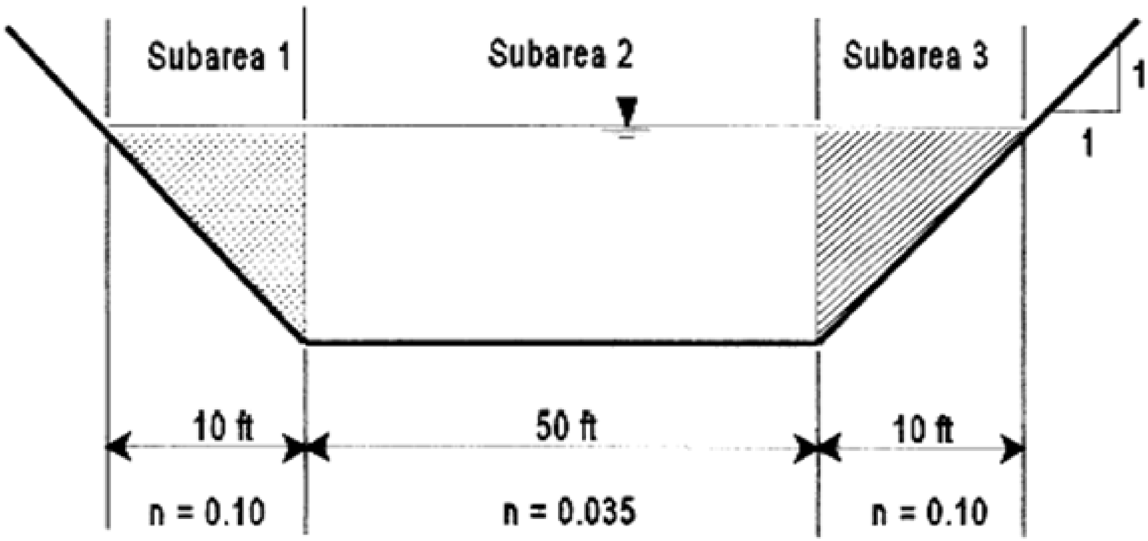
Method two is correct. Method two combines subareas of the channel cross section which have similar hydraulic properties to yield a reasonable answer of total conveyance. If n values between section "A" and "B" were significantly different, combining them to determine conveyance might not provide the desired accuracy.

Method three is incorrect but exemplifies how easily one can underestimate total conveyance by not considering the conveyance from the other subareas. Obviously the total conveyance cannot be less than that contained in one section.

Method four is correct. This may be considered overkill but technically it is the most accurate solution. If n values were significantly different between section "A" and "B", this type of subdivision for determining conveyance would be essential.

Example 3-2

In Figure E-2, a trapezoidal cross section having heavy brush and trees on the banks has been subdivided near the bottom of each bank because of the abrupt change of roughness there. A large percentage of the wetted perimeters (P) of the triangular subareas (A_1 and A_3) and of the main channel (A_2) have been eliminated. A smaller wetted perimeter abnormally increases the hydraulic radius (R = A/P), and this in turn results in a computed conveyance different from the conveyance determined for a section with a complete wetted perimeter. In Figure E-2, the total conveyance (K_T) has been computed to be 102,000 for the cross section. This would require a composite n value of 0.034. This is less than the n values of 0.035 and 0.10 that describe the trapezoidal shape. The basic shape should be left unsubdivided, and an effective value of n somewhat higher than 0.035 should be assigned to this cross section, to account for the additional drag imposed by the larger roughness on the banks.



$$K_T = K_1 + K_2 + K_3 = 102{,}000$$

$A_1 = A_3 = 50$	$A_2 = 500$
$P_1 = P_3 = 14.14$	$P_2 = 50$
$R_1 = R_3 = 3.54$	$R_2 = 10$

Composite Solution:

$A_c = 600$

$P_c = 78.3$

$R_c = 7.66$

$$n_c = \frac{1.49\, A_c\, R_c^{2/3}}{K_T} = 0.034$$

Figure E-2 Effects of Subdivision on a Trapezoidal Section

REFERENCES

American Association of State Highway and Transportation Officials, 2008. Guide Specifications for Bridges Vulnerable to Coastal Storms. 1st edition. AASHTO Publication, Washington, D.C.

Ayres Associates, 1994. Development of Hydraulic Computer Models to Analyze Tidal and Coastal Stream Hydraulic Conditions at Highway Structures, Final Report, Phase I HPR552, South Carolina Department of Transportation, Columbia, SC.

Ayres Associates, 1997. Development of Hydraulic Computer Models to Analyze Tidal and Coastal Hydraulic Conditions at Highway Structures, Phase II Report, prepared for the South Carolina Department of Transportation.

Ayres Associates, 2002. Development of Hydraulic Computer Models to Analyze Tidal and Coastal Hydraulic Conditions at Highway Structures, Phase III Report, prepared for the South Carolina Department of Transportation.

Donigian, A. S., 2002. "Watershed Model Calibration and Validation: The HSPF Experience." Proceedings of the Water Environment Federation, National TMDL Science and Policy. Phoenix, AZ

Douglass, Scott L., and Joe Krolak. 2008. Highways in the Coastal Environment Hydraulic Engineering Circular 25 Second Edition. Office of Bridge Technology Federal Highway Administration, June.

FEMA 2007. Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update. Federal Emergency Management Agency. Guidelines and Specifications for Flood Hazard Mapping Partners, Appendix D.

Froelich, D.C. 2002. User's Manual for FESWMS FST2DH, Two-dimensional Depth-averaged Flow and Sediment Transport Model. FHWA Report No. FHWA-RD-03-053. National Technical Information Service, Springfield, VA.

Gosselin, M., D.M. Sheppard, and S. McLemore 2006. NCHRP Web-Only Document 106: Criteria for Selecting Hydraulic Models. National Cooperative Highway Research Program, Transportation Research Board. http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_w106.pdf

Holland, G. J., 1980. "An analytic model of the windand pressure profiles in hurricanes." *Monthly Weather Review*, 108, 1212–1218.

Jiang, H., J. B Halverson, J. Simpson, and E. J Zipser. 2008. Hurricane "Rainfall Potential" Derived from Satellite Observations Aids Overland Rainfall Prediction. Journal of Applied Meteorology and Climatology 47, no. 4: 944–959.

Lagasse, P.F., Clopper, P.E., Pagán-Ortiz, J.E., Zevenbergen, L.W., Arneson, L.A., Schall, J.D., and Girard L.G. 2009 Bridge Scour and Stream Instability Countermeasures. Hydraulic Engineering Circular No. 23. FHWA NHI 09-111, Federal Highway Administration, Washington, D.C.

Lagasse, P.F., Schall, J.D., and Richardson, E.V. 1991. Stream Stability at Highway Structures. Hydraulic Engineering Circular No. 20. FHWA NHI 01-002, Federal Highway Administration, Washington, D.C.

Laursen, E.M., 1960, "Scour at Bridge Crossings," Journal Hydraulic Division, American Society of Civil Engineers, Vol. 86, No. HY 2.

Matyas, C. J. 2005. Relating Tropical Cyclone Rainfall Patterns to Storm Size. In West Palm Beach, FL., November 21.

Neill, C.R. (ed.) 1973. Guide to Bridge Hydraulics. Roads and Transportation Association of Canada, University of Toronto Press, Toronto.

Ochi, M.K. (2004) "Prediction of Hurricane Landfall Wind Speed along the Florida Coast". UF Report UFL/COEL-TR/136, Gainesville, FL.

Pope, B. F., G. D. Tasker, and J. C. Robbins. 2001. Estimating the Magnitude and Frequency of Floods in Rural Basins of North Carolina, Revised. United States Geological Survey.

Richardson, E.V., & Davis, S.R. 2001. Evaluating Scour at Bridges Fourth Edition,Hydraulic Engineering Circular No. 18. FHWA NHI 01-001, Federal Highway Administration, Washington, D.C.

Richardson, E.V., Simons, D.B., and Lagasse, P.F. (2001) Highways in the River Environment, Hydraulic Series No. 6. FHWA NHI 01-004, Federal Highway Administration, Washington, D.C.

Sheppard, D. M., & Renna, R. (2005). Florida Bridge Scour Manual. Florida Department of Transportation. Tallahassee: FDOT.

Sheppard, D.M. and W. Miller, Jr. 2003. "Design Storm Surge Hydrographs for the Florida Coast." Final Report to Florida Department of Transportation.

Sheppard, D. M., D. Slinn, and S. Hagen. 2006. Design Hurricane Storm Surge Pilot Study. FDOT Research Office Report. FDOT Contract No. BD 545#42.

U.S. Army Corps of Engineers. 1984. Shore Protection Manual. 4th ed., 2 vols. U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, U.S. Government Printing Office, Washington D.C.

U.S. Army Corps of Engineers. 2002. Coastal Engineering Manual. Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C.

U.S. Army Corps of Engineers. (2006). Coastal Engineering Manual. Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C. (in 6 volumes)

U.S. Army Corps of Engineers. (1995). Design of Coastal Revetments, Sea Walls, and Bulkheads. EM 1110-2-1614. Corps of Engineers, Waterways Experiment Sation, Vicksburg.

Zevenbergen, L.W., P.F. Lagasse, and B.L. Edge. 2005. Tidal Hydrology, Hydraulics and Scour at Bridges Hydraulic Engineering Circular 25 First Edition. Office of Bridge Technology Federal Highway Administration, December.