CHAPTER 5: BRIDGE HYDRAULICS

BRIDGE HYDRAULICS

5.1 PROJECT APPROACH AND MISCELLANEOUS CONSIDERATIONS

The material in this section addresses background information and initial decision making needed in preparation for a bridge hydraulic design. The following sections present more detailed design guidance.

Most bridge projects in Florida receive funding from the Federal Highway Administration (FHWA). Even if the project is not planned to receive 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.

Title 23 Code of Federal Regulations (CFR) 650A outlines the principal hydraulic analysis and design requirements that you must satisfy to qualify bridge projects (as well as any other project involving floodplain encroachments) for Federal Aid. The requirements in 23 CFR 650A are very comprehensive, so you, as the drainage engineer, should become familiar with them. The Title 23 CFR with FHWA's supplemental information is available at https://www.fhwa.dot.gov/legsregs/directives/cfr23toc.htm

5.1.1 Identify Hydraulic Conditions

Before beginning any hydraulic analysis of a bridge, first you must determine the mode of flow for the waterway. For purposes of bridge hydraulics, the Department separates the mode of flow into three 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 oceanward 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, and open sections of the Intracoastal Waterway typically are tidally dominated, so much so that even extreme rainfall events have little influence on the design flows in these systems. Tidally dominated areas with negligible upland influx require only examination of design storm surge conditions.

3. Tidally influenced flow—Both river flow and tidal fluctuations affect flows in tidally influenced crossings, such as tidal creeks and rivers opening to tidally dominated waterways. 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 you to examine 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 (see Figure 5.1-1) 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, plan to include a coastal engineer for these bridge projects.

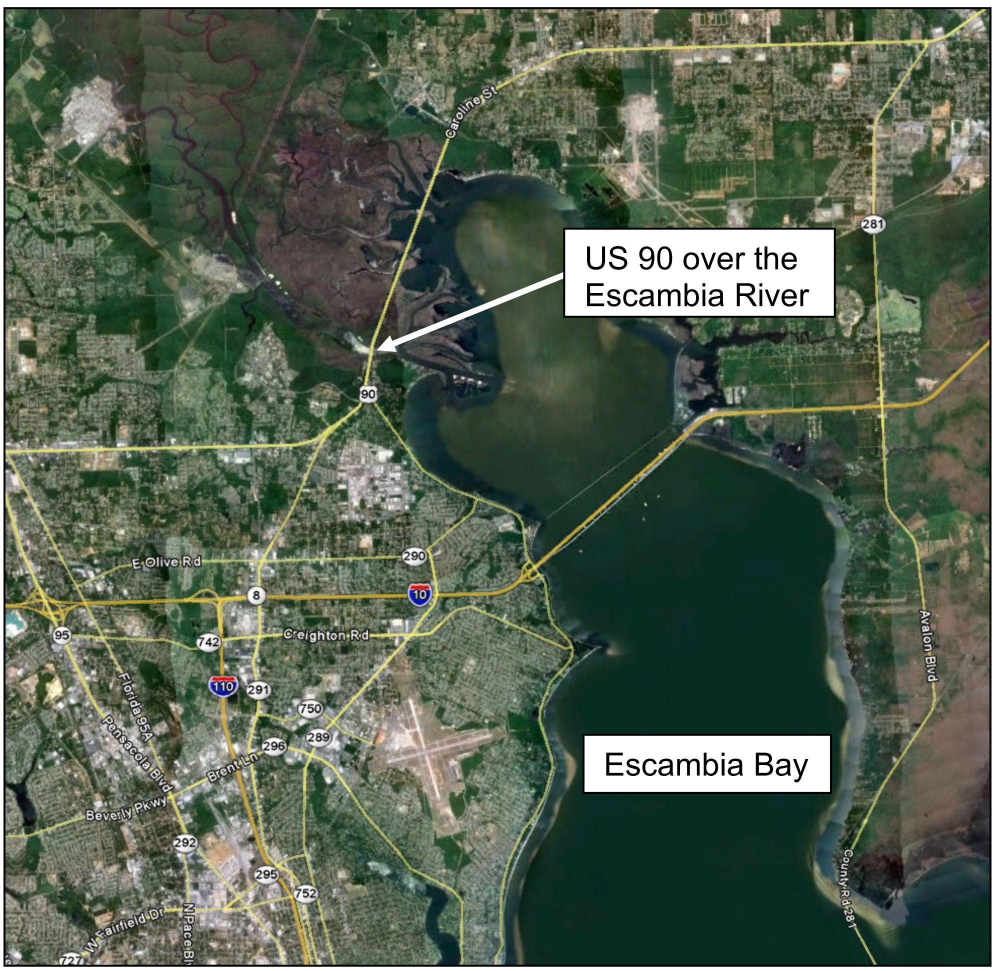


Figure 5.1-1: Example of a Bridge Requiring both Riverine and Tidal Analyses (US-90 over Escambia Bay)

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 (see Figure 5.1-2). 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).

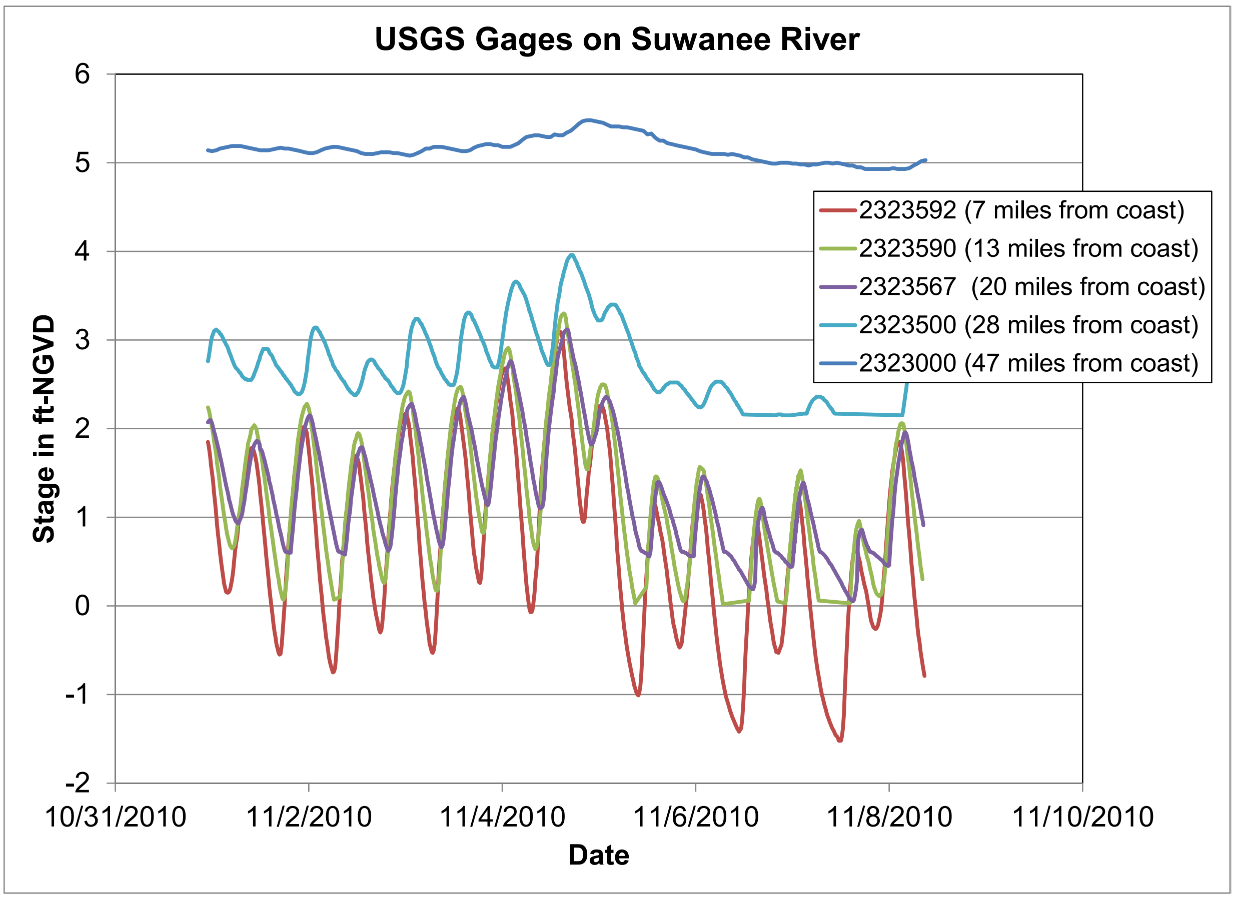


Figure 5.1-2: USGS Gage Data from the Suwannee River with Increasing Distance from the Coast

For the purposes of Department work, a coastal engineer is an engineer who holds a Master of Science or doctoral degree 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.

5.1.2 Floodplain Requirements

Address potential floodplain impacts during the Project Development and Environment (PD&E) phase of the project. Usually, you will not prepare a Bridge Hydraulics Report (BHR) during PD&E studies. However, if you do not prepare a BHR 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 Part 2, Chapter 24 of the FDOT *Project Development and Environment Manual* for more information on floodplain assessment during PD&E.

5.1.2.1 FEMA Requirements

All bridge crossings must be consistent with the National Flood Insurance Program (NFIP), which 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 5.1-3). 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 5.1-4).

Figure 5.1-5 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.

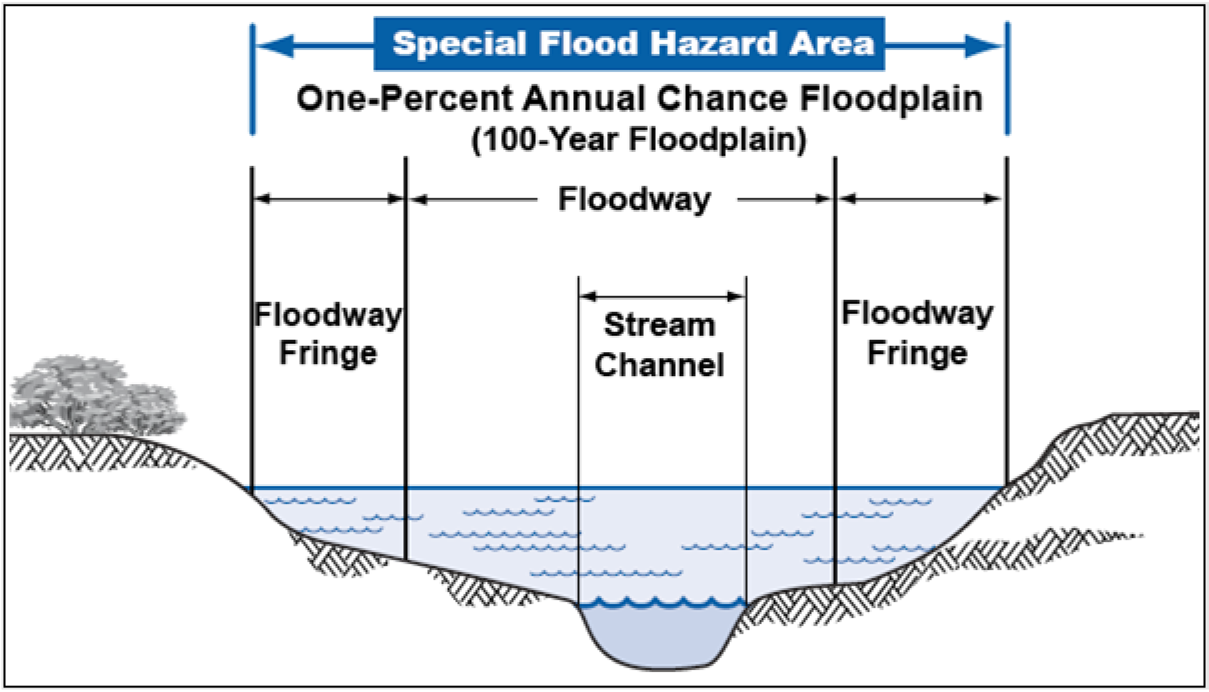


Figure 5.1-3: Special Flood Hazard Area

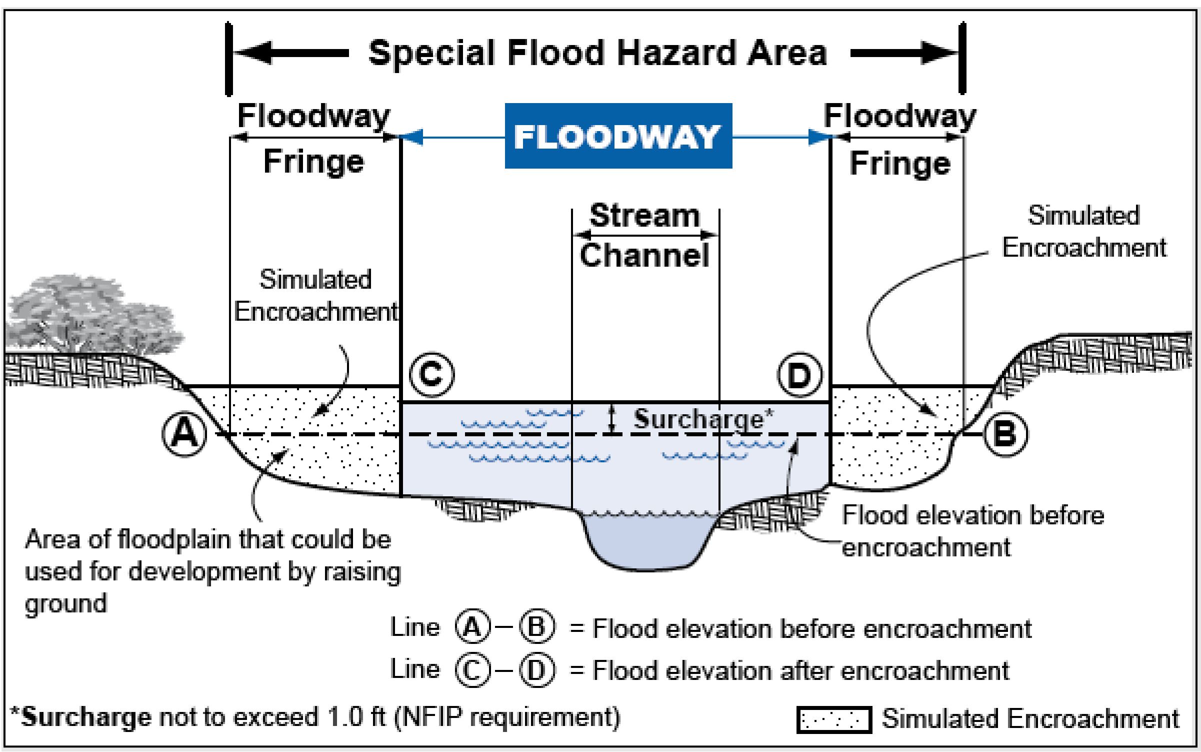


Figure 5.1-4: Floodway Definitions

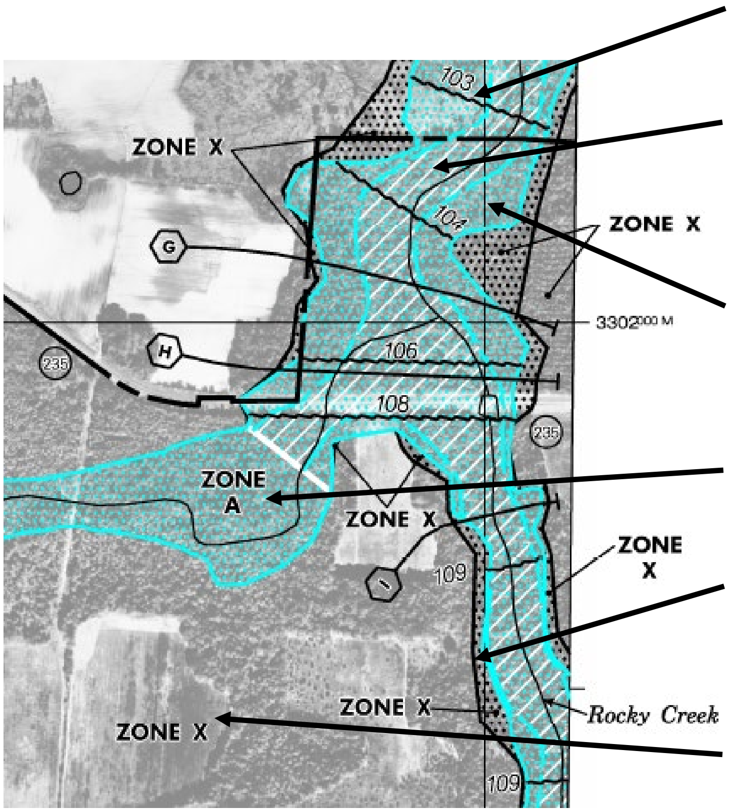


Figure 5.1-5: Example Flood Map

The simplest way to be consistent with the NFIP standards for an established floodway is to design the bridge and approach roadways so that you exclude their components 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 be considered 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. You will prepare a No-Rise Certification and support it by technical data. Base the data 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 5.1-6). In this case, the bridge and roadway approaches may be designed to allow no more than a one-foot increase in the base flood elevation depending on local regulations and if offsite land use values will not be significantly impacted (see Section 4.4 of the *Drainage Manual*). Use information from the FIS and the original hydraulic model to model the bridge and submit technical data to the local community and FEMA.

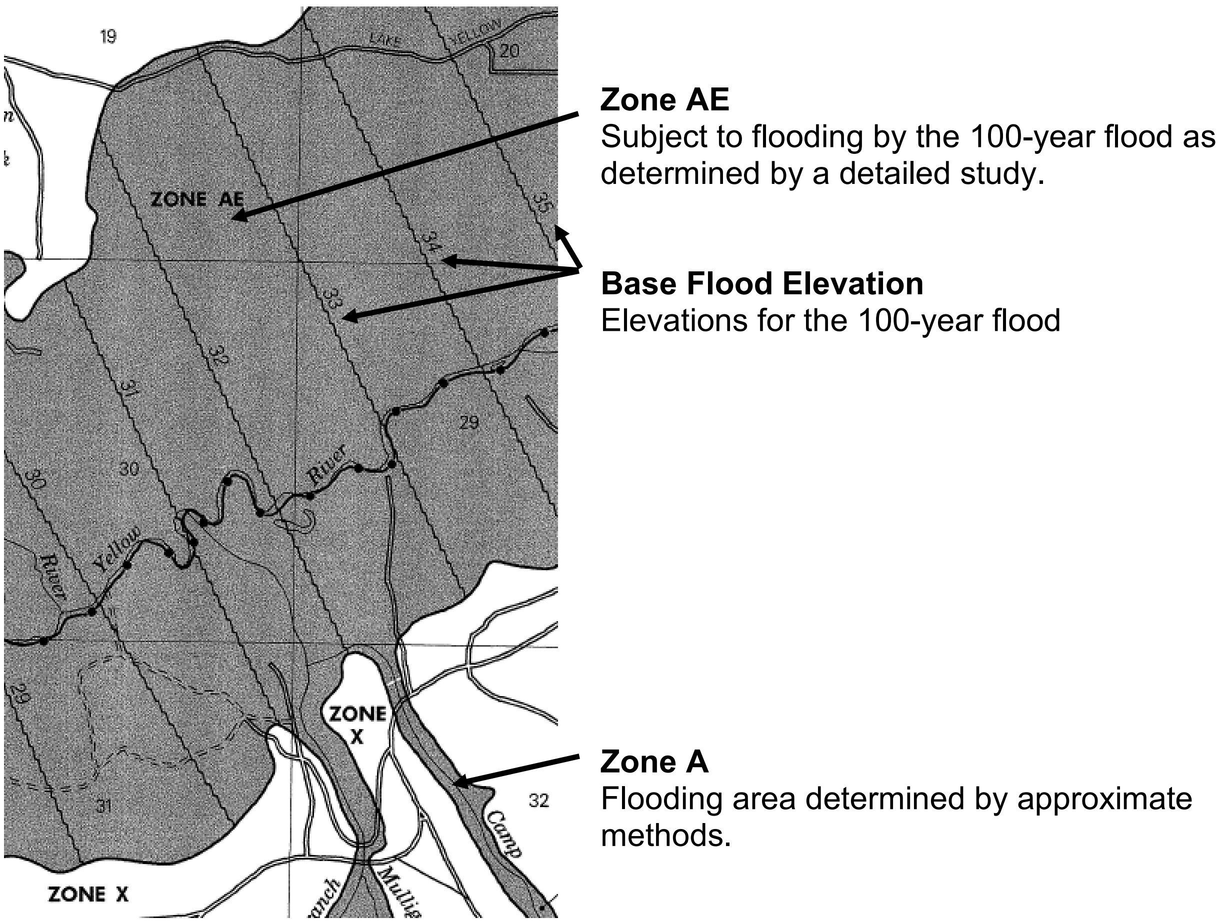


Figure 5.1-6: Example Flood Map

If the encroachment is in an area without a detailed study (Zone A on Figure 5.1-5 and Figure 5.1-6), then generate technical data for the project. You should give base flood information to the local community. Pursuant to NFIP regulations in CFR 60.3(c)(10), no more than a foot of increase in base flood elevation is allowed for cumulative development within the floodplain.

5.1.2.2 Other Government Agency Requirements

Many government agencies (cities, counties, Water Management Districts, etc.) will have additional limitations on backwater conditions in floodplains. The agency may designate the limitations at multiple distances upstream from the bridge. For example, backwater increase immediately upstream must be limited to one foot, and backwater increase 1,000 feet upstream must be limited to 0.1 foot.

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

5.1.3 Design Frequencies

Design frequency requirements are given in Section 4.3 of the *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:

- The bridge must convey the design frequency without damage (Section 4.2 of the *Drainage Manual*).
- Backwater for the design frequency must be at or below the travel lanes (Section 4.4 of the *Drainage Manual*).
- The bridge must have adequate debris clearance.

Figure 5.1-7 shows the relationship between these design frequency criteria and the geometric design. These criteria tend 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 G, Risk Evaluations. Discuss changing the design frequency with the District Drainage Engineer before making a final decision. In addition, incorporate or address in the design hydraulic design frequency standards of other agencies that have control or jurisdiction over the waterway or facility.

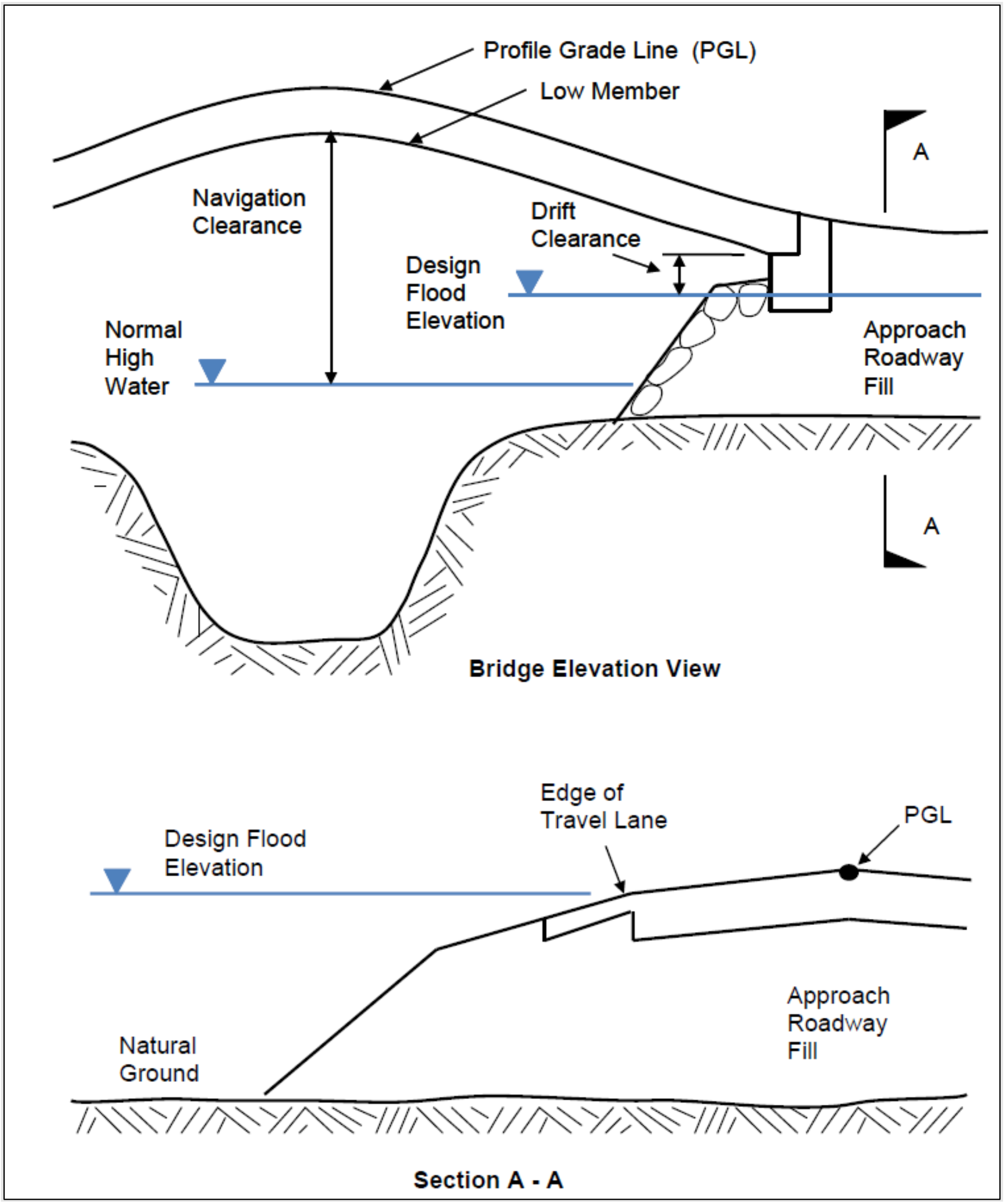


Figure 5.1-7: Bridge and Cross Drain Roadway Grade Controls

Scour analysis and design has a separate design frequency, discussed in Section 4.9 of the *Drainage Manual*. You will find national standards for scour design in FHWA HEC 18, *Evaluating Scour at Bridges*.

The worst-case condition for scour usually will occur at overtopping of the approach roadway or another basin boundary. Overtopping flow at the bridge often provides flow relief, and scour conditions will be a maximum at overtopping.

For more guidance on scour computation and design, refer to Section 5.5 of this document and the FDOT *Bridge Scour Manual*.

5.1.4 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 vertical and horizontal clearance requirements must be maintained to ensure the hydraulic crossing functions in conformance with the design criteria. Minimum clearances are addressed in the *FDM 260*.

5.1.4.1 Debris

Per *FDM 260*, a two-foot minimum debris drift clearance used by the Department traditionally has provided an acceptable level of service. Though this clearance usually is adequate for facilities of all types, review bridge maintenance records for the size and type of debris that may be expected. For example, if the watershed is a forested area subject to timbering activities, anticipate sizeable logs and trees among the debris. Meandering rivers also will tend to fell trees along the banks, carrying them toward downstream bridge crossings. On the other hand, bridges immediately downstream from a pump station may have little opportunity to encounter debris. Also, manmade canals tend to be stable laterally and will fell many fewer trees than sinuous, moving natural rivers. In such low debris cases, if a reduced vertical clearance is economically ideal, the hydraulic designer should approach the District Drainage Engineer for a variation to reduce the debris drift clearance.

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

5.1.4.2 Canal Right-Of-Way

When a bridge crosses over a local permitting agencies’ canal, additional permitting may be required. Early coordination with the local entity for any additional requirements for the crossing is recommended.

For example, any roadway going over a SFWMD Canal, the roadway is required to obtain a SFWMD Right-of-Way Occupancy Permit. SFWMD has a *ROW Permit Information Manual* on guidelines that specify vertical and horizontal clearances, among other criteria, required to obtain the permit. SFWMD R/W Occupancy Permit information can be reviewed at https://www.sfwmd.gov/doing-business-with-us/permits/right-of-way

5.1.4.3 Navigation

Per *FDM 260*, 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, you can obtain this information from the U.S. Geological Survey (USGS). Statistically, the mean annual flood is equivalent to the 2.33-year frequency interval (recurrence interval). Therefore, if you use a synthetic hydrologic method to determine the Normal High Water, use the 2.33-year event. In some cases, stain lines at the site indicating the normal flood levels can be used to estimate the Normal High Water.

Obtain control elevations from the regulating agency (Water Management Districts, water control districts, U.S. Army Corps of Engineers, etc.).

5.1.4.4 Waves

Elevate coastal bridges one foot above the design wave crest, as required in the *FDM 260*. If the clearance is less than one foot, which often occurs near the bridge approaches, you must design the bridge according to the *Guide Specifications for Bridges Vulnerable to Coastal Storms,* a publication from the American Association of State Highway and Transportation Officials (AASHTO).

5.1.5 Bridge Length Justification

It is typically unnecessary to span the entire width of a stream at flood stages. Where conditions permit, you can extend approach embankments 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 the water surface profile and scour conditions are evaluated properly.

The BHR should demonstrate clearly that the proposed structure length and configuration are justified for the crossing. Use historical records from the life of the bridge, along with hydrologic and hydraulic calculations, 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 to analyze another length may become apparent, and that may turn out to be the proposed structure length.

5.1.6 Berms and Spill-Through Abutment Bridges

Normally, you would not place spill-through abutments 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*, you must determine the horizontal limit of protection 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, make the horizontal 10-foot measurement 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, you should make the horizontal 10-foot measurement 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 that provides the minimum berm width often will be the shortest bridge length 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 7.7 fps, or wave heights not exceeding 2.4 feet. Determine the horizontal and vertical extent using HEC 23. A minimum of 10 feet is recommended as a horizontal extent even if HEC 23 shows that a horizontal extent less than 10 feet is acceptable. 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, you can do the following:

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

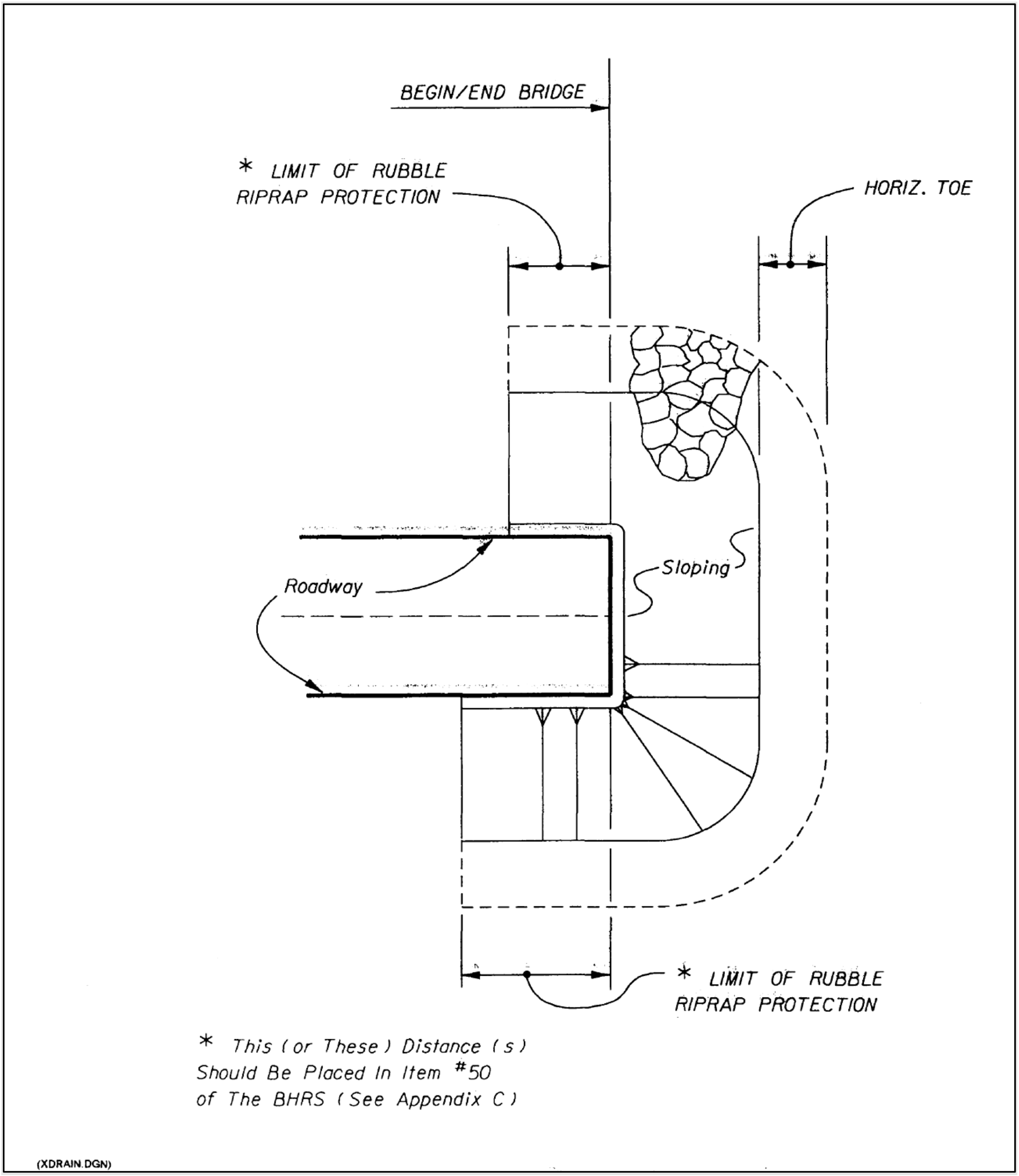


Figure 5.1-8: Limits of Rubble Riprap Protection

Figure 5.1-8 is a plan view that 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 the USACE *Shore Protection Manual* for design of coastal revetment.

Use bedding stone 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.) that are appropriate based on-site conditions. Write a technical specification 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 (CSIPs) during construction.

No matter what options are allowed, match the bedding (filter fabric and bedding stone) 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.

5.1.7 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, consider the potential for lateral migration of the stream, and plan that 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. You can develop a conservative configuration 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 5.1-9 shows some configurations.

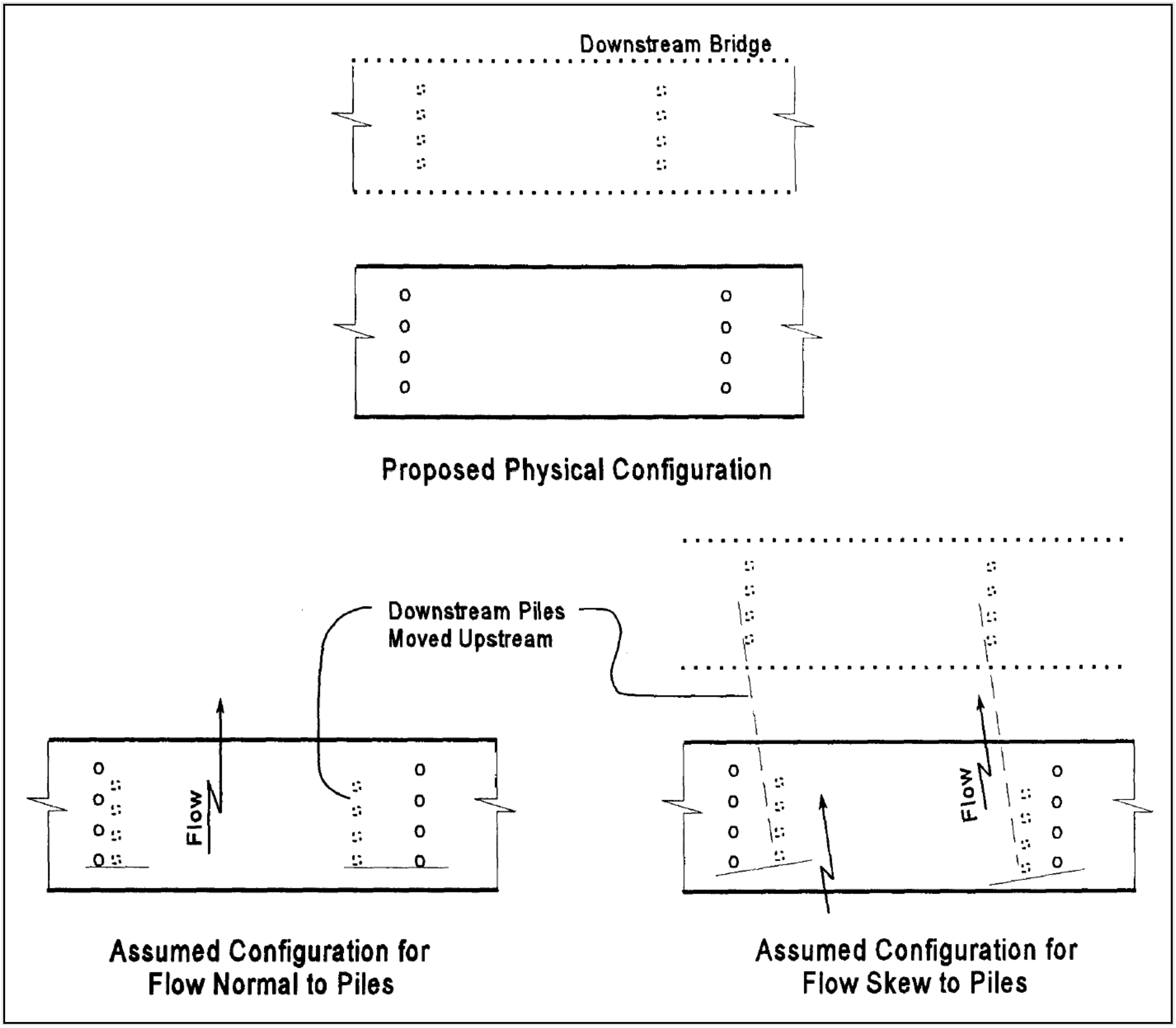


Figure 5.1-9: Configurations for Computing Scour of Dual Bridges

5.1.8 Design Considerations for Bridge Widenings

The new substructure or foundations under the widened portion of a bridge often are different than the existing substructure in 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 (defined in Chapter 6 of the FDOT *Structures Design Guidelines*) 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 often will 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 deviating from criteria.

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. Typically, they are located on both sides of the structure being protected, as shown in Figure 5.1-10. Fender system types are less variable, consisting usually of pile-supported wales, as shown in Figure 5.1-11. Fender systems typically wrap around the protected piers and run along the main navigation channel.

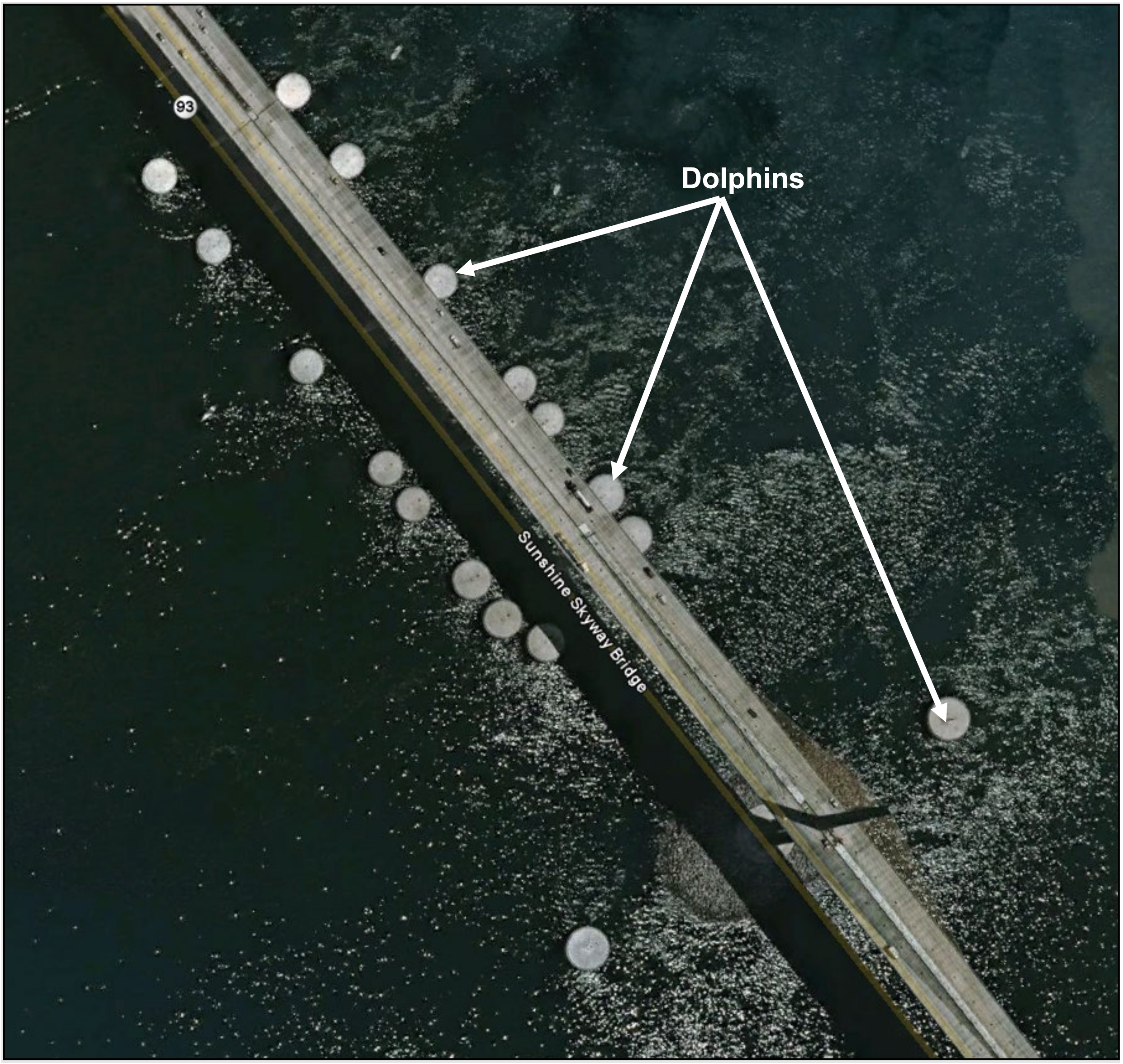


Figure 5.1-10: Dolphin Pier Protection at the Sunshine Skyway Bridge

Figure 5.1-11: Fender System at the Old Jewfish Creek Bridge

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

Scour at fender systems typically is 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 hydraulic shadow of the fender system. Piers and fender systems introduced into relatively narrow rivers may cause contraction scour between the fender systems. This scour usually is greatest near the downstream end of the system.

5.2 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 usually can be assumed.

5.2.1 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.

5.2.1.1 Geometric Data

Follow these steps 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 5.2.4 for guidance.
2. Locate available geometric data within the model domain. You can use liberally estimated boundaries of the domain 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 estimate the domain boundaries conservatively.

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 (DEMs)
    - DEMs are essentially x, y, z coordinate points on a 90-meter grid. They were derived from the Quadrangle Maps. DEMs also are available at LABINS.
  - LiDAR
    - Coverage in Florida is not yet complete. Available data can be downloaded at:https://www.usgs.gov/centers/eros/science/usgs-eros-archive-products-overview?qt-science_center_objects=0#qt-science_center_objects
- U.S. Army Corps of Engineers
  - USACE performs hydrographic surveys on navigable waterways, which can provide main channel information.
  - Mobile District: https://www.sam.usace.army.mil/Missions/Spatial-Data-Branch/

  - Jacksonville District: http://www.saj.usace.army.mil/Missions/CivilWorks/Navigation/HydroSurveys.aspx
- 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 5.2.1.1 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, you may use them to fill in gaps farther away from the site.

The remaining data sources usually will have a level of accuracy that was adequate for hydraulic modeling at the time of collection. However, consider the age of the data. If the terrain within the model domain has changed significantly, then you must find newer existing data sources or you will need to order survey.

You may need data from different sources 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.

Ordering Survey Data

The FDOT *Surveying Handbook* (dated October 31, 2003) states that bridge survey and channel survey requirements are project specific. You will need to provide site-specific instructions to the surveyors so that they do 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 you can use cross sections to develop two-dimensional models, a DTM format is preferable. Discuss the survey format with the surveyor to determine which format is most appropriate.

Always order survey 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.

Determine the location of the approach and exit cross sections for the model and extend survey information in the main channel to these locations. Additional survey information in the adjacent floodplain and farther upstream and downstream of these extents will depend upon the other available geometric data.

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. Also ask the surveyor 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.

5.2.1.2 Geotechnical Data

Geotechnical information is required at bridge foundations 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 Section 5.5, 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), include sieve analyses in the geotechnical data collection to characterize the size of the bed sediments. 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 cohesive sediments (rock or clay), establish the bed material's scour resistance. For rock, the FHWA provides guidelines for scourability of rock formations in HEC 18 (refer to Chapter 4).

Additionally, the Department has developed a Rock Scour Protocol, which you can find at:
http://www.fdot.gov/roadway/Drainage/Bridgescour/Bridge-Rock-Scour-Analysis-Protocol-Jan2008.pdf.
The referenced protocol recommends obtaining core borings at each pier for testing at the State Materials Office. It is your responsibility 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.

5.2.1.3 Historical Data

Historical data provide important information for many aspects of the bridge hydraulics and scour analysis. They provide numbers 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.

Gage Measurements
In bridge hydraulics analysis, you can use gage data in a number of ways:

- 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 (Hydrology) for more information.
- To provide starting water surface elevations, or boundary conditions, for the model if the gage is downstream of the bridge. Refer to Section 5.2.4.9 for more information.
- To calibrate the model. Refer to Section 5.2.5.1 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 still may be useful if the flow rates can be adjusted. Refer to Section 4.5, Peak Flow Transposition in FHWA Highway Hydrology, Hydraulic Design Series 2 (HDS-2), for more information.

USGS gage information can be found at this website:
https://waterdata.usgs.gov/fl/nwis/current/?type=flow

Gage data may also be available from the Water Management Districts and other local agencies.

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. You also can use current aerial photographs 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 archives aerials dating back to the 1940s. Ordering information is available at the following link:

https://www.fdot.gov/gis/aerialmain.shtm

The University of Florida also maintains a database of older aerial photographs:

http://ufdc.ufl.edu/aerials

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

http://www.labins.org/

Existing Bridge Inspection Reports

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

Channel profiles usually are 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 mid-span.

You may be able to find the Phase 1 Scour Evaluation Report 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 5.2-1 and 5.2-2 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.

You can use the channel profiles to determine long-term bed changes at the bridge site.

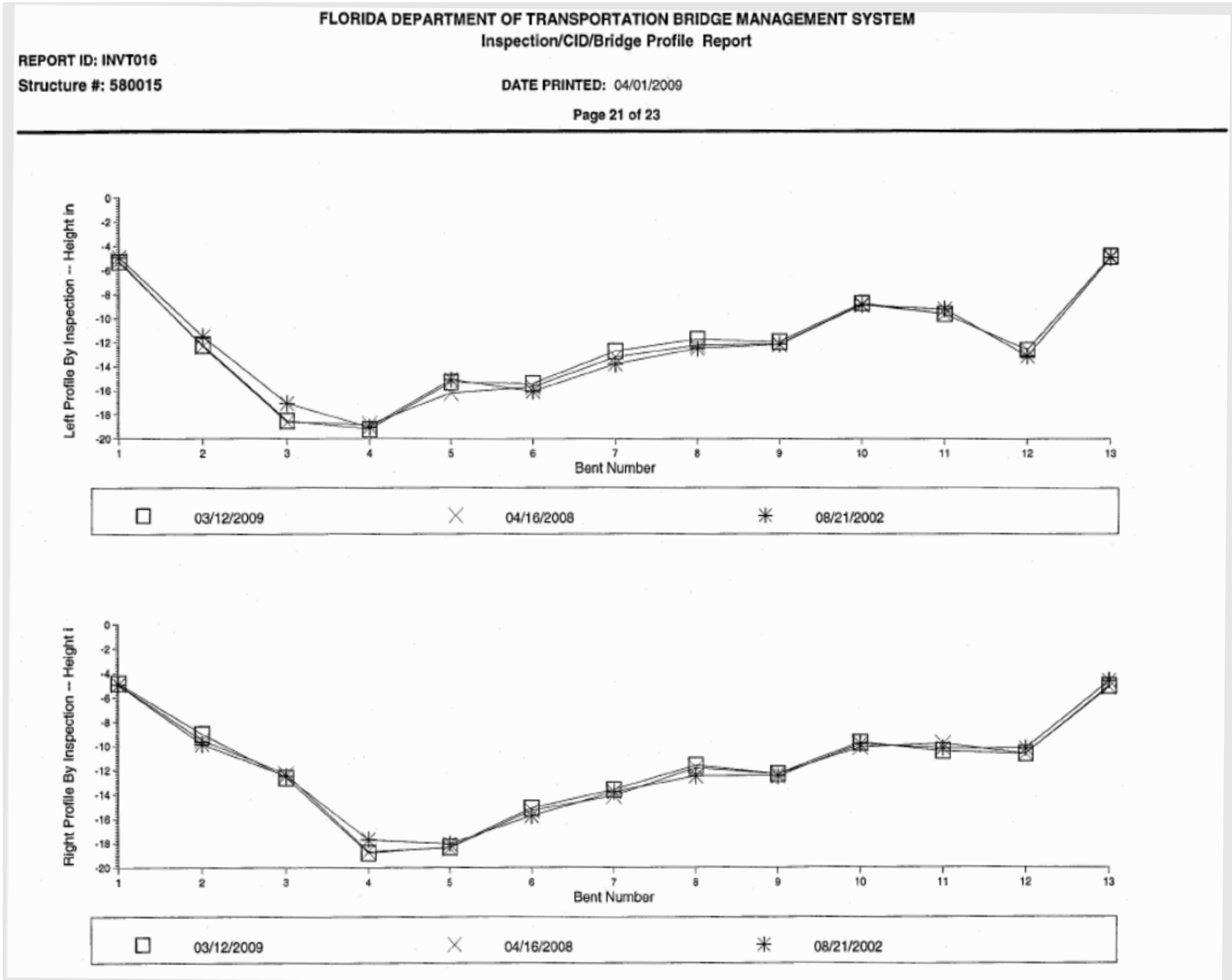


Figure 5.2-1: Example Bridge Profile from a Bridge Inspection Report

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

(All Heights Are In Feet)

Inspection Date and Key: 03/12/2009 CXYM

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 5.2-1: Example Bridge Profile from a Bridge Inspection Report (cont.)

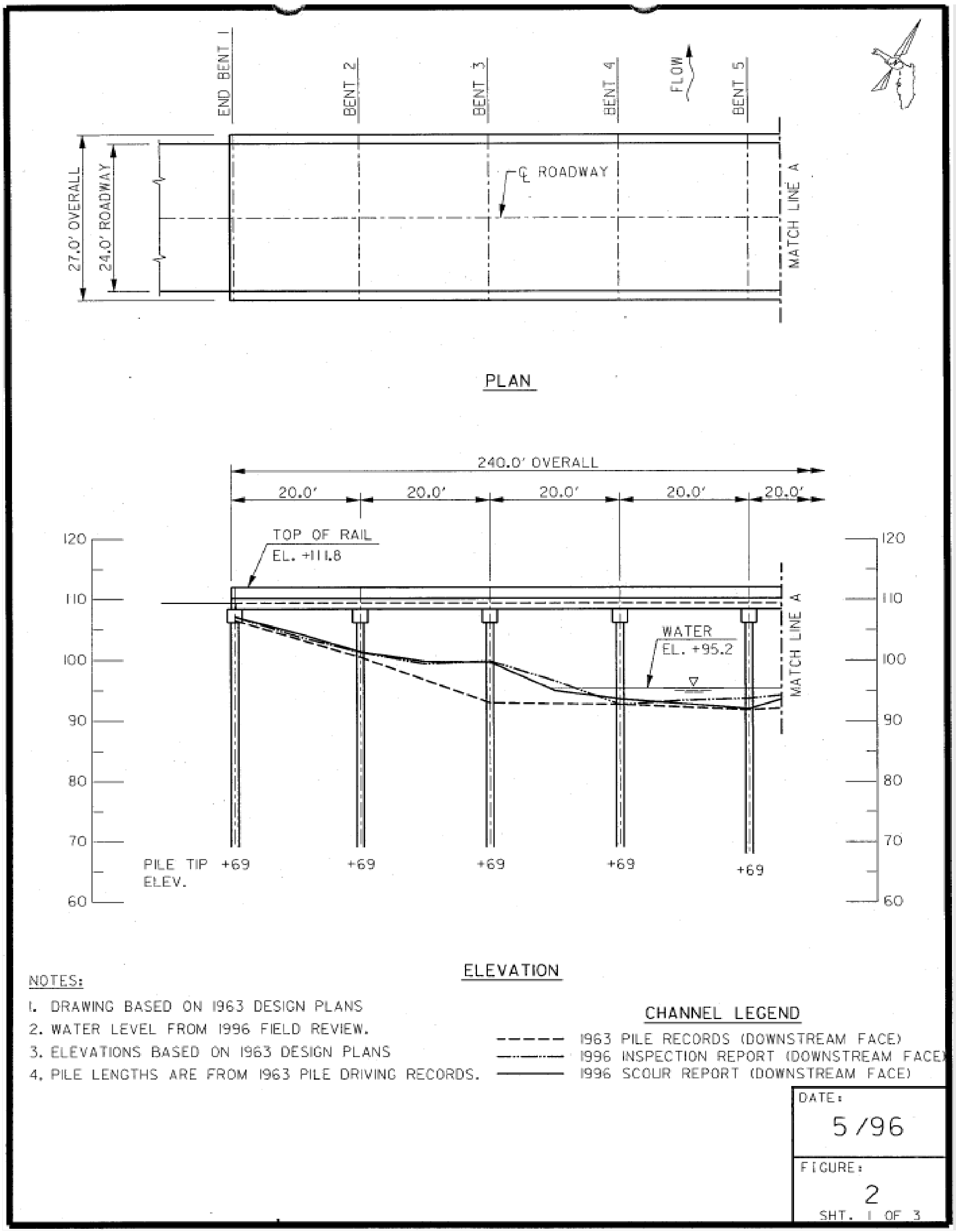


Figure 5.2-2: Excerpt from Scour Evaluation Report

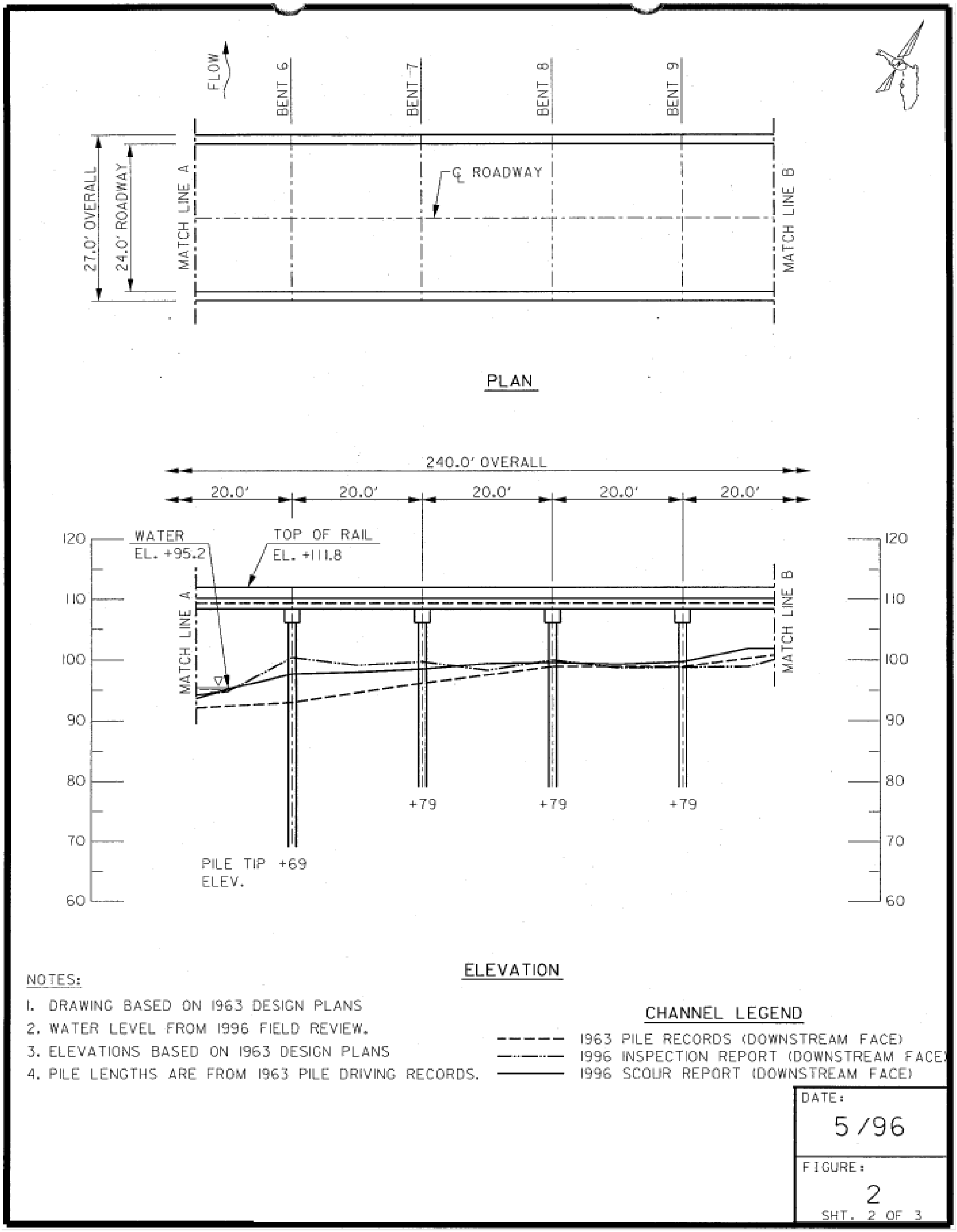


Figure 5.2-2: Excerpt from Scour Evaluation Report (continued)

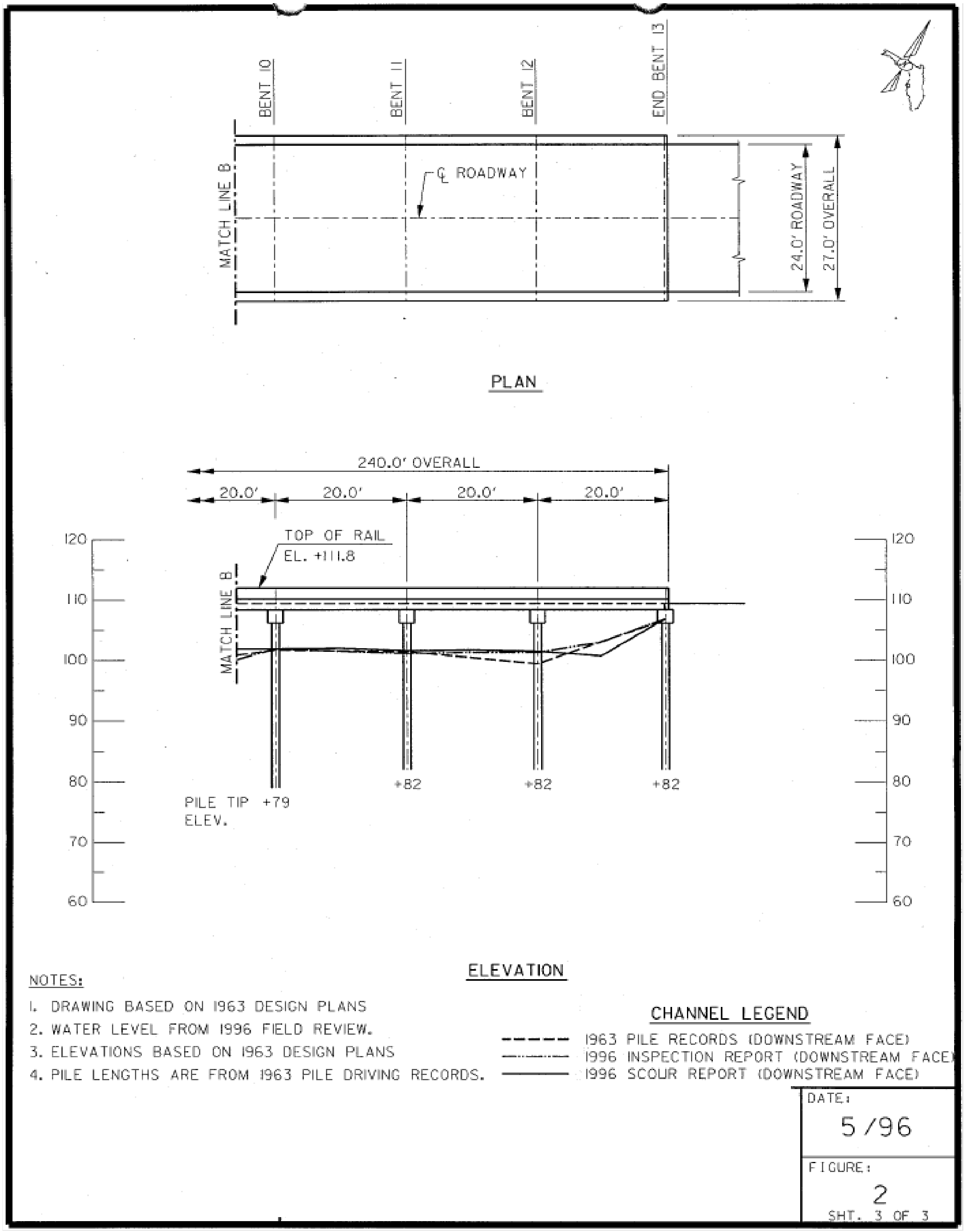


Figure 5.2-2: Excerpt from Scour Evaluation Report (continued)

Previous Studies

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

If a detailed study was performed by FEMA, then obtain the Flood Insurance Study, the NFIP Maps, and the original model (refer to Section 5.2.1.5).

Additional sources of existing studies can include the Water Management Districts, the Florida Department of Environmental Regulation, county offices, and the U.S. Army Corps of Engineers.

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.

5.2.1.4 Drainage Basin Information

Obtain drainage basin information for the hydrologic analysis. The type of information collected depends upon the hydrologic method used in the analysis. Refer to Section 5.2.2 and Chapter 2 (Hydrology) for guidance on the hydrologic analysis and data requirements.

Delineate the drainage basin boundaries 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.

Also gather information on other structures on the river upstream and downstream of the proposed bridge site, including the size and type of structure for comparison with the proposed structure.

5.2.1.5 FEMA Maps

Obtain the FEMA Flood Insurance Rate Map and the Flood Insurance Study for the site. You can order these maps or download them from the FEMA Map Service Center at the following link:

https://msc.fema.gov/portal/home

Backup and supporting data for a detailed study, if the area has a detailed study, also can be obtained from FEMA. A data request form must be completed and sent to FEMA. Contact the FEMA Map Service Center for ordering information.

https://www.fema.gov/sites/default/files/documents/fema_flood-insurance-study-data-request-form.pdf

5.2.1.6 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. Contact the agency exercising control over these structures 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.

5.2.1.7 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. Perform a field investigation 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.

5.2.2 Hydrology

In most riverine analyses, you can assume steady-state conditions and perform the hydraulic analysis 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.

Section 4.7.1 of the *Drainage Manual* gives criteria for selecting discharges used for riverine analysis. Further guidance is given in Chapter 2 (Hydrology).

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) usually will not significantly affect the amount of water going downstream. However, if you or the regulatory agency are significantly concerned about this effect, then you should conduct an analysis to verify the concern. You can calculate the pre- and post-water surface profiles and route them with an unsteady flow model.

5.2.3 Model Selection

Before selecting a specific model to use at a given bridge site, you must make two general decisions to isolate groups of appropriate models.

The two basic decisions are:

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

5.2.3.1 One-Dimensional versus Two-Dimensional

It is important for the hydraulic engineer to accurately represent the hydraulic condition. The engineer should understand the model assumptions because they form the limitations of that approach. The approach should be selected based primarily on its advantages and limitations, though also considering the importance of the structure, potential project impacts, cost, and schedule.

One-dimensional modeling requires that variables (velocity, depth, etc.) change predominantly in one defined direction, x, along the channel. Because channels are rarely straight, the computational direction is along the channel centerline. Two-dimensional models compute the horizontal velocity components (V_x and V_y) or, alternatively, velocity vector magnitude and direction throughout the model domain. Therefore, two-dimensional models avoid many assumptions required by one-dimensional models, especially for the natural, compound channels (free-surface bridge flow channel with floodplains) that make up the majority of bridge crossings over water.

Advantages of two-dimensional modeling include a significant improvement in calculating hydraulic variables at bridges. One-dimensional models are best suited for in-channel flows and when floodplain flows are minor. They are also frequently applicable to small streams. For extreme flood conditions, one-dimensional models generally provide accurate results for narrow to moderate floodplain widths. They can also be used for wide floodplains when the degree of bridge constriction is small and the floodplain vegetation is not highly variable. In general, where lateral velocities are small one-dimensional models provide reasonable results. Avoiding significant lateral velocities is the reason why cross section placement and orientation are so important for one-dimensional modeling.

Two-dimensional models generally provide more accurate representations of:

- Flow Distribution
- Velocity Distribution
- Water Surface Elevation
- Backwater
- Velocity Magnitude
- Velocity Direction
- Flow Depth
- Shear Stress

Although this list is general, these variables are essential information for new bridge design, evaluating existing bridges for scour potential, and countermeasure design.

Two-dimensional models should be used when flow patterns are complex and one-dimensional model assumptions are significantly violated. If the hydraulic engineer has great difficulty in visualizing the flow patterns and setting up a one-dimensional model that realistically represents the flow field, then two-dimensional modeling should be used.

One study that developed criteria for selecting one- versus two-dimensional models is "Criteria for Selecting Hydraulic Models" (NCHRP 2006). The recommendations from that study are summarized and expanded on below.

<u>Multiple Openings.</u> Multiple openings along an embankment are often used on rivers with wide floodplains. Rather than using a single bridge, additional floodplain bridges are included. Although one-dimensional models can be configured to analyze multiple openings, the judgment and assumptions that are made by the hydraulic engineer in combination with the assumptions and limitations of the software result in an extreme degree of uncertainty in the results. The proportion of flow going through a particular bridge and the corresponding flow depth and velocity are important for structure design and scour analysis. Because multiple opening bridges represent a large investment, two-dimensional analysis is always warranted.

Another type of multiple opening is multiple bridges in series. There are conditions when this bridge configuration should be analyzed using two-dimensional models. These include unmatched bridge openings or foundations that do not align. An upstream or downstream railroad or parallel road may significantly alter the flow conditions and warrant two-dimensional analysis.

<u>Wide Floodplains.</u> Floodplains often include features that significantly impact flow conveyance and flow distribution. Historic channel alignments and changes in land use or vegetation affect floodplain flow distribution. In a one-dimensional model, cross sections may have significantly different vegetation or topography due to land use activities. Using the cross sections exactly as they exist, the one-dimensional model will depict a sudden change in flow distribution that is not physically possible. The two-dimensional model avoids these difficulties because in the simulation all the flow is interconnected. Therefore, wide and complex floodplains benefit from two-dimensional analysis.

Skewed Roadway Alignment. Roadways should be aligned perpendicular to channel and floodplain flows. FHWA (1978) indicates that skewed crossings with angles of up to 20 degrees produced no objectionable flow patterns. Two-dimensional modeling is the recommended approach for higher skew angles or moderate amounts of skew combined with moderate to high flow contraction. Not only will the flow patterns and bridge conveyance be better defined, but potential problems with backwater will also be evident.

Bends, Confluences and Angle of Attack. Highly sinuous rivers are, by definition, not one-dimensional, especially during floods when water in the floodplain moves in and out of the channel. Two-dimensional models do not make any simplifying assumptions related to channel versus floodplain flow distance because the two-dimensional network directly incorporates flow paths. Flow conditions at confluences also vary depending on the proportion of flow in the main stem and tributary. Two-dimensional models provide improved estimates of angle of attack because velocity direction is computed directly.

Two-dimensional modeling may also be considered for design conditions including road overtopping, upstream obstructions, multiple channels, or countermeasure designs.

Table 4.1. Bridge Hydraulic Modeling Selection.

Bridge Hydraulic Condition	Hydraulic Analysis Method	
	One-Dimensional	Two-Dimensional
Small streams	●	◗
In-channel flows	●	◗
Narrow to moderate-width floodplains	●	◗
Wide floodplains	◗	●
Minor floodplain constriction	●	◗
Highly variable floodplain roughness	◗	●
Highly sinuous channels	◗	●
Multiple embankment openings	◗/○	●
Unmatched multiple openings in series	◗/○	●
Low skew roadway alignment (<20°)	●	◗
Moderately skewed roadway alignment (>20° and <30°)	◗	●
Highly skewed roadway alignment (>30°)	○	●
Detailed analysis of bends, confluences and angle of attack	○	●
Multiple channels	◗	●
Small tidal streams and rivers	●	◗
Large tidal waterways and wind-influenced conditions	○	●
Detailed flow distribution at bridges	◗	●
Significant roadway overtopping	◗	●
Upstream controls	○	●
Countermeasure design	◗	●

● well suited or primary use
◗ possible application or secondary use
○ unsuitable or rarely used
◗/○ possibly unsuitable depending on application

5.2.3.2 Steady versus Unsteady Flow

Use an unsteady flow model 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 versus 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).

You can find more information on these situations in USACE Manual EM 1110-2-1416 (15 October 1993), *River Hydraulics*.

5.2.3.3 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. Make sure you are using the latest version and document the version in the Bridge Hydraulics Report.

HEC-RAS and WSPRO both are suitable to analyze one-dimensional, gradually varied, steady flow in open channels, and you also can use them 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 you 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. You can use this information to estimate the backwater effects of the structure and provide 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 developed originally for FHWA and the 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.

The Resource Management Associates software RMA 2 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.

5.2.4 Model Setup

You will need the following data 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)

5.2.4.1 Defining the Model Domain

You will need upstream, downstream, and lateral study boundaries 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 causes the water surface calculations to be less accurate or requires additional survey at a higher cost than the inclusion in the initial survey. Overestimation results in greater survey, data processing, and analysis cost.

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 farther 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.

The following equation can be used to determine how far upstream data collection and analysis needs to be performed.

$$Lu = 10{,}000 * HD^{0.6} * HL^{0.5}/S$$

where:

Lu = Upstream study length (along main channel), in feet for normal depth starting conditions

HD = Average reach hydraulic depth (1-percent chance flow area divided by cross section top width), in feet

S = Average reach slope, in feet per mile

HL = Head loss, ranging between 0.5 feet and 5.0 feet at the channel crossing structure for the 1-percent 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).

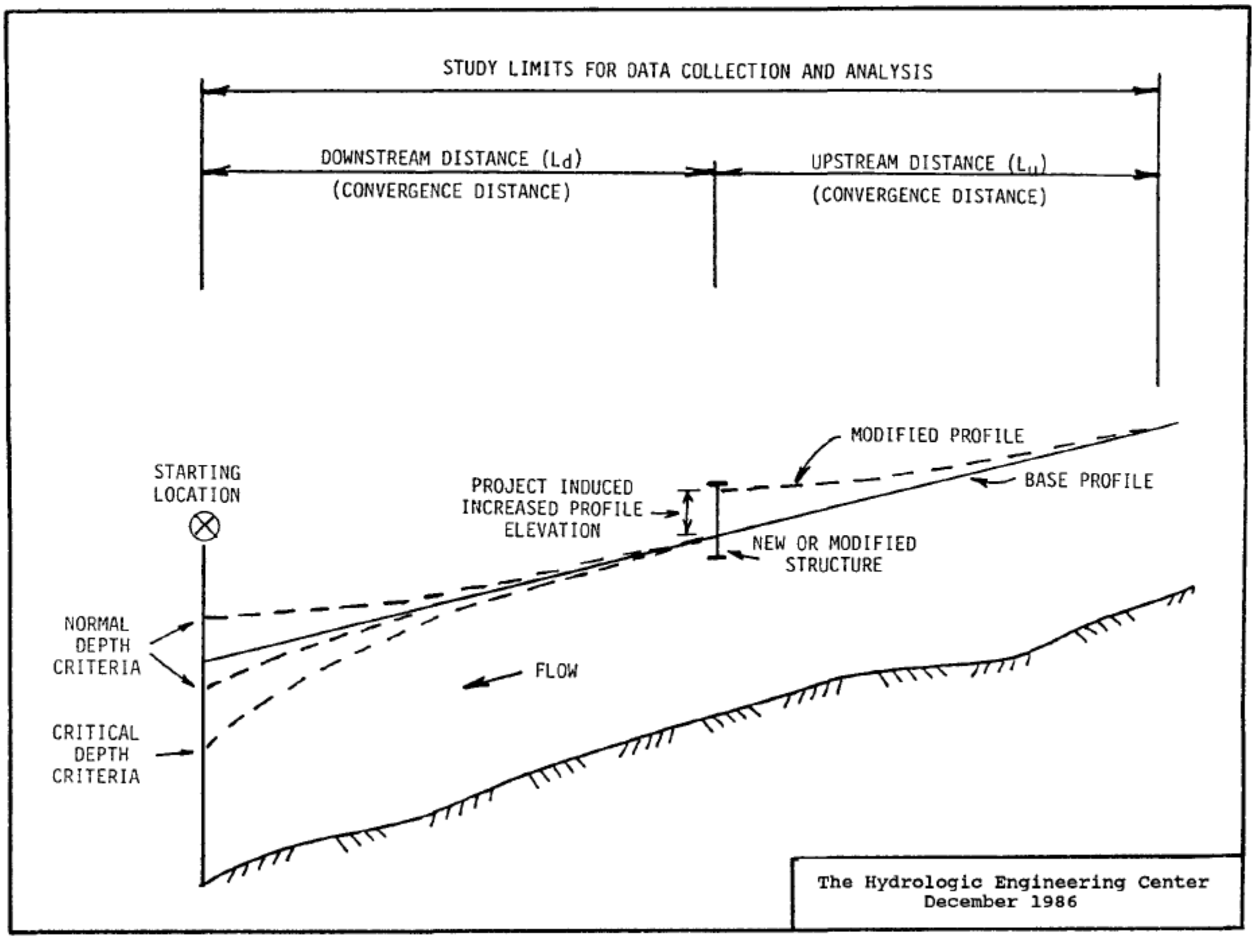


Figure 5.2-3: Open Channel Depth Profiles

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 to use, 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, extend the model 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 also can be points of known elevation. Other locations where you can calculate the water surface elevation from the discharge can include weirs, dams, and culverts if these locations are not significantly influenced by their tailwater.

When the downstream channel and overbank are nearly uniform, use the normal depth assumption to determine the starting water surface elevation, 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. This reach should not be subject to significant backwater from farther downstream. The following equation can be used to determine how far downstream data collection and analysis needs to be performed.

$$Ldn = 8{,}000 * HD^{0.8}/S$$

where:

Ldn = Downstream study length (along main channel), in feet for normal depth starting conditions

HD = Average reach hydraulic depth (1-percent chance flow area divided by cross section top width), in feet

S = Average reach slope, in feet per mile

Make some sound engineering judgment when determining the variables HD, S, and HL. Guidelines are presented below:

a. **Average reach hydraulic depth (HD):** If limited existing data are 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. Use survey data or other existing geometric data that are more accurate than the Quadrangle Maps 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 also is 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. 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.

Lateral Extents
Extend the model 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 be unknown until the model is complete. But you must collect data to complete the model. So, you must estimate the water surface elevation and lateral extent for the data-gathering effort. You can estimate the elevation or the lateral extent from FEMA maps and other historical studies of the site. In some cases, it is appropriate to set up a preliminary model based on limited data to estimate the water surface elevations. Whichever method you use 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.

5.2.4.2 Roughness Coefficient Selection

You can use a number of references 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*, USGS Water-Supply Paper 2339 (replaces Report Number FHWA-TS-84-204), which can be accessed at the following link: http://www.fhwa.dot.gov/bridge/wsp2339.pdf
2. *Estimating Manning's Roughness Coefficients for Natural and Man-Made Streams in Illinois,* USGS and Illinois Department of Natural Resources. http://il.water.usgs.gov/proj/nvalues/

Roughness values from previous models or studies can be useful. However, you should verify these roughness values 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.

5.2.4.3 Model Geometry

Model selection is discussed in Section 5.2.3. This section discusses the creation of one- and two-dimensional models.

One-Dimensional Models
One-dimensional models use cross sections to define the geometry of the channel and floodplain. There are several good references available for 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/twri3-a15/pdf/twri_3-A15_b.pdf.

Some of the guidelines presented below are from this reference.

a. Take cross sections where there is an appreciable change in slope.
b. Take cross sections where there is an appreciable change in cross sectional area (i.e., minimum and maximum flow areas).
c. Space cross sections 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 5.2-4 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.

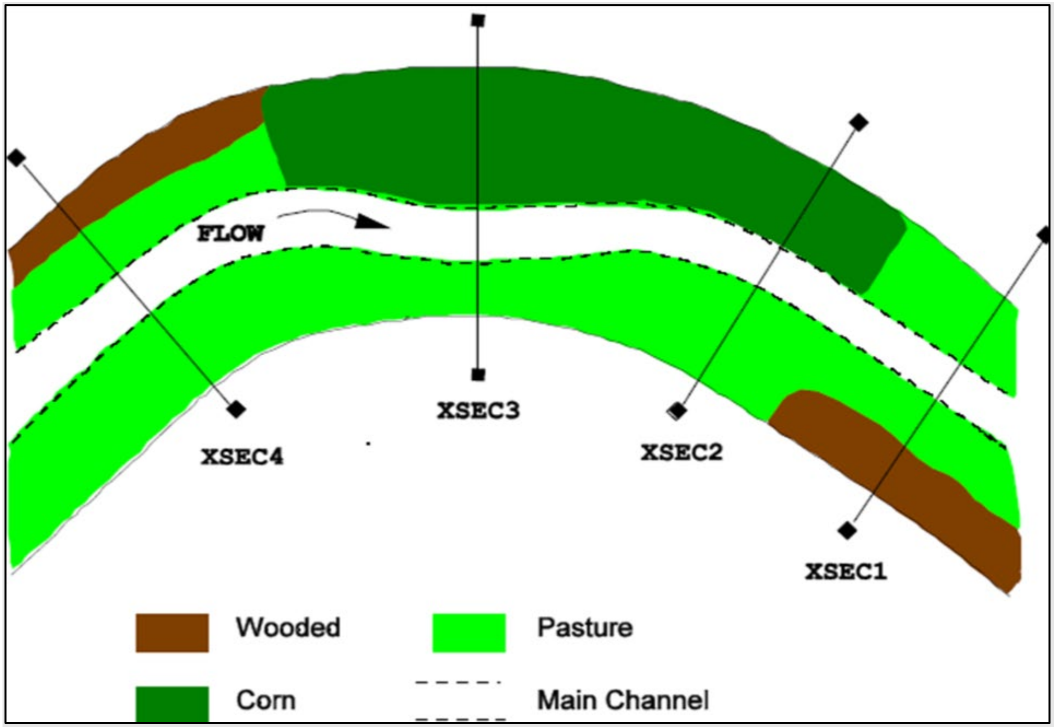


Figure 5.2-4: Example Cross Section Spacing

d. Take cross sections normal to the flood flow lines. In some cases, you may need to “dog leg” cross sections. Figure 5.2-5 illustrates this procedure.

e. Place cross sections 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 downstream conveyance, K2, should satisfy the criterion: $0.7<(K1/K2)<1.4$.

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

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

h. Place cross sections between sections that change radically in shape, even if the two areas and the two conveyances are nearly the same.

i. Place cross sections 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 generally will 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. Locate cross sections at or near control sections.

k. Locate cross sections 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.

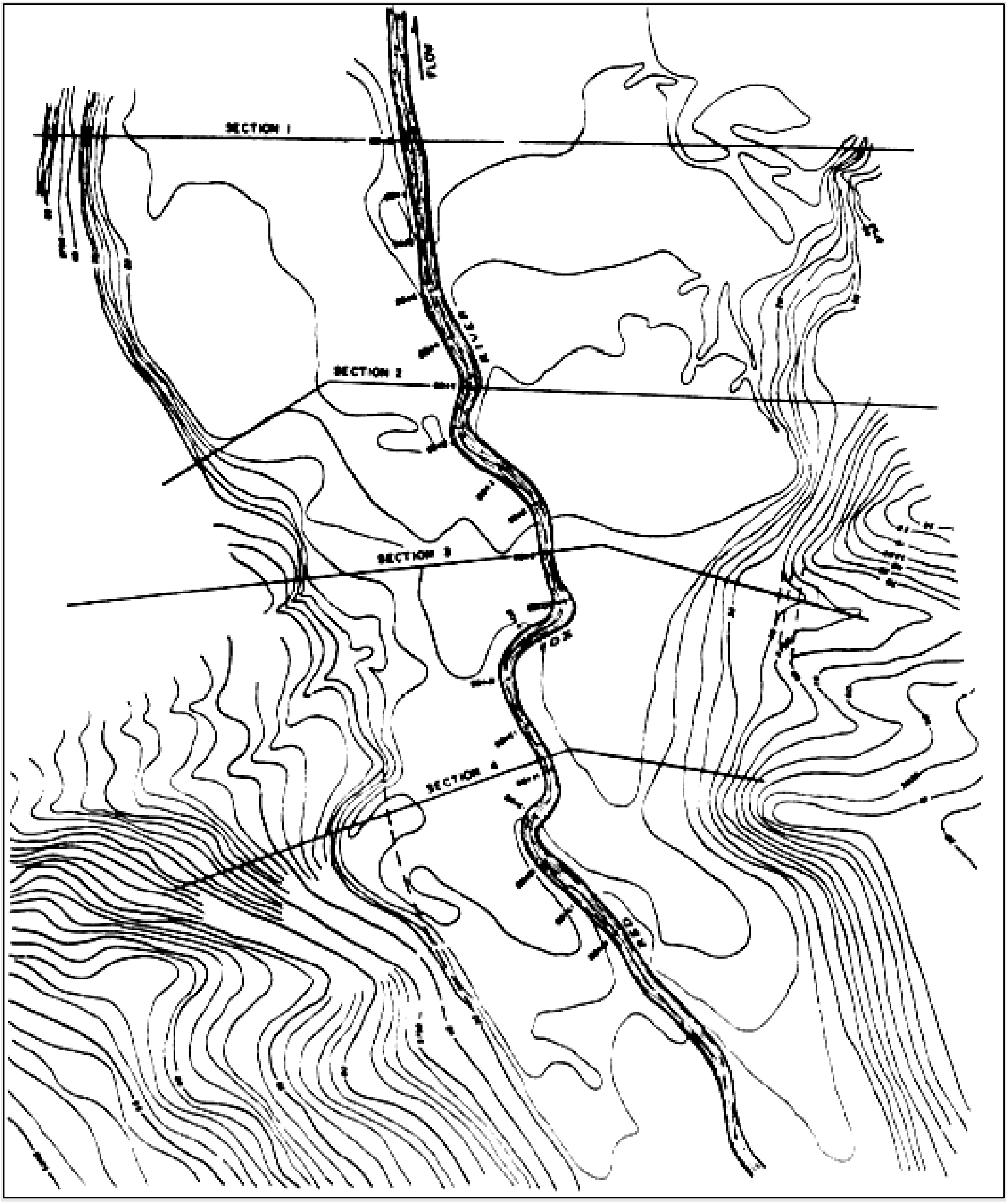


Figure 5.2-5: “Dog Legging” Cross Section

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

Figures 5.2-6 and 5.2-7 show guidelines on when to subdivide.

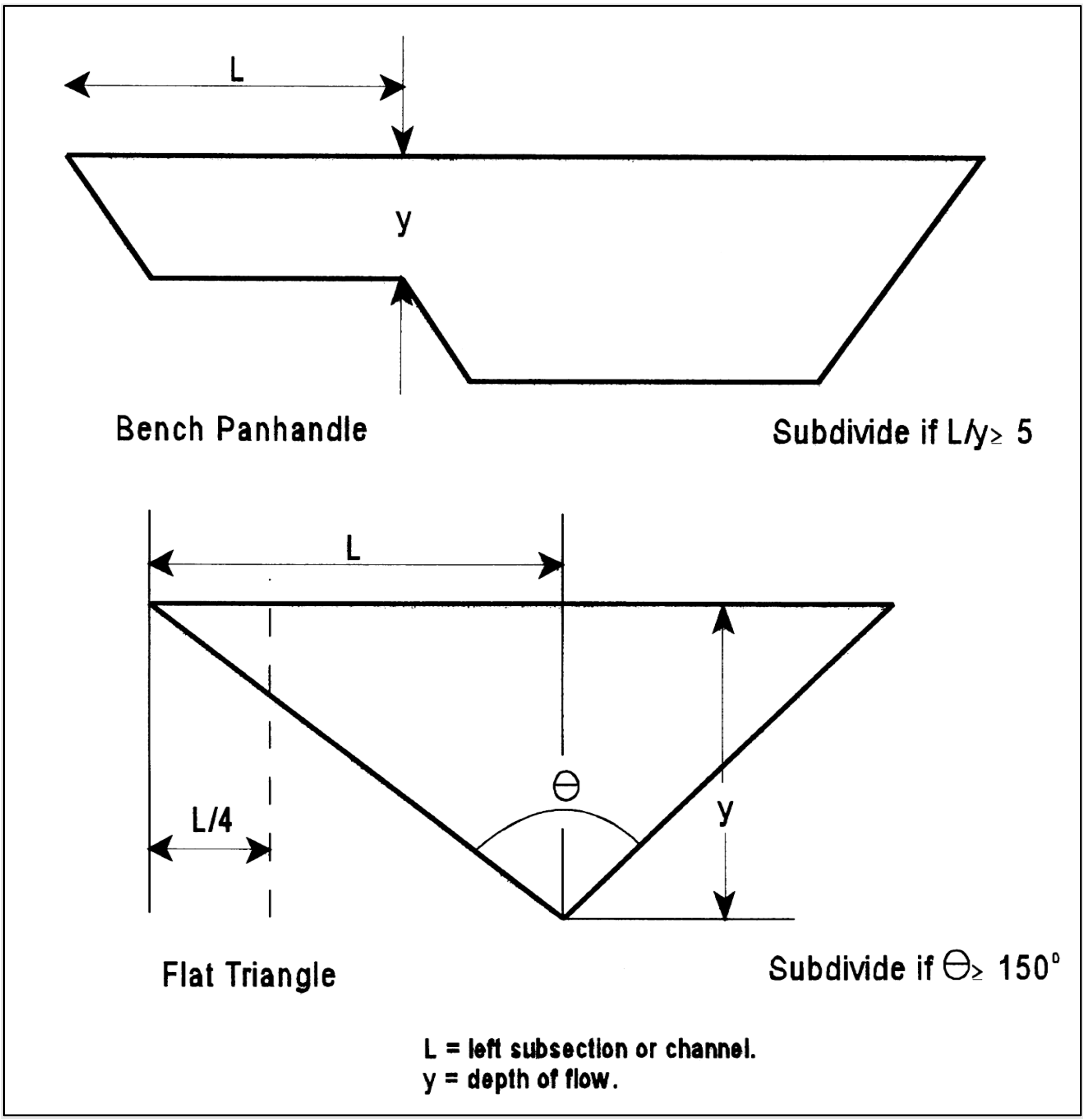


Figure 5.2-6: Subdivision Criteria of Tice (written communication, 1973)

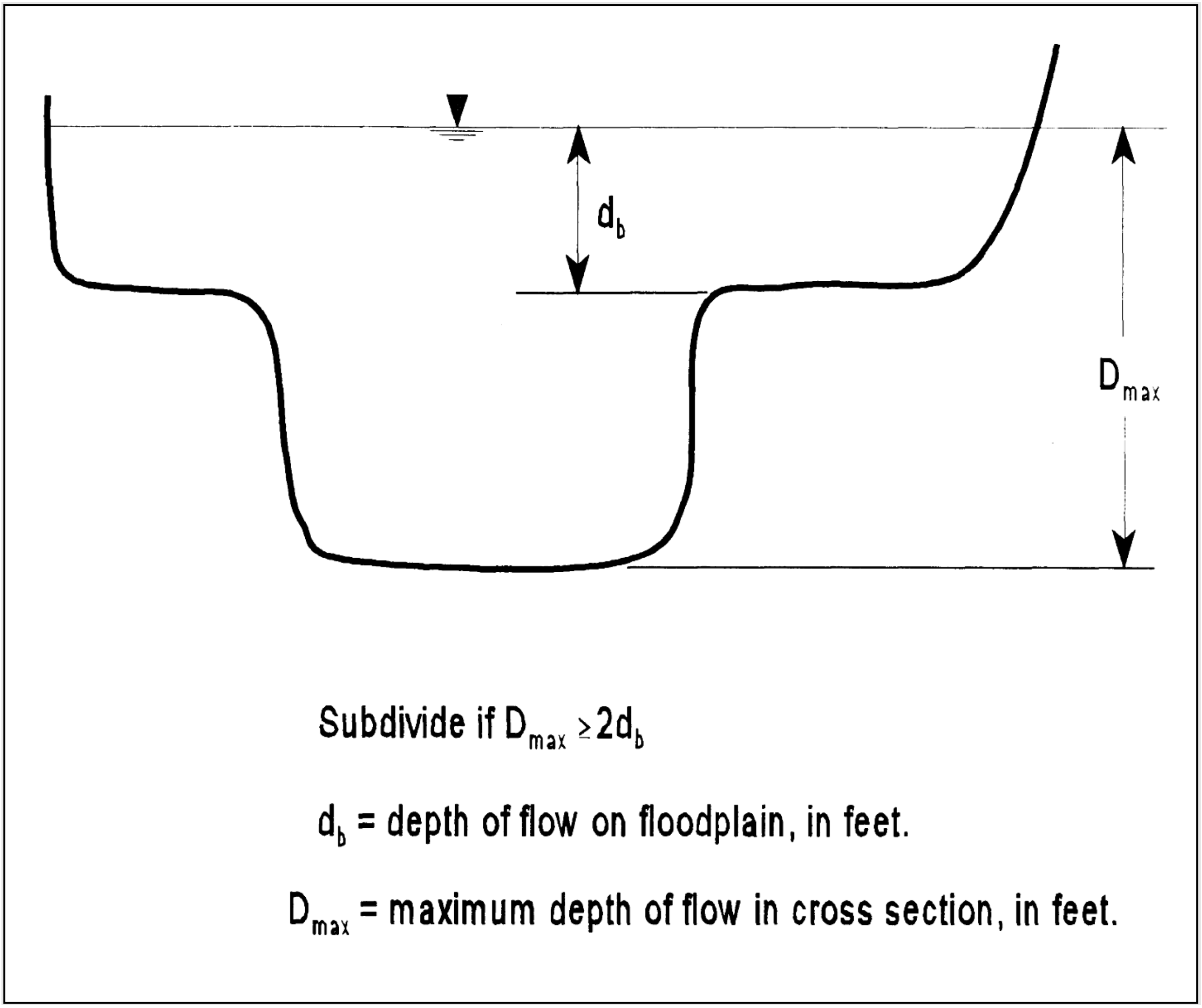


Figure 5.2-7: Subdivision Criteria of Tice (written communication, 1973)

(A) Conveyance

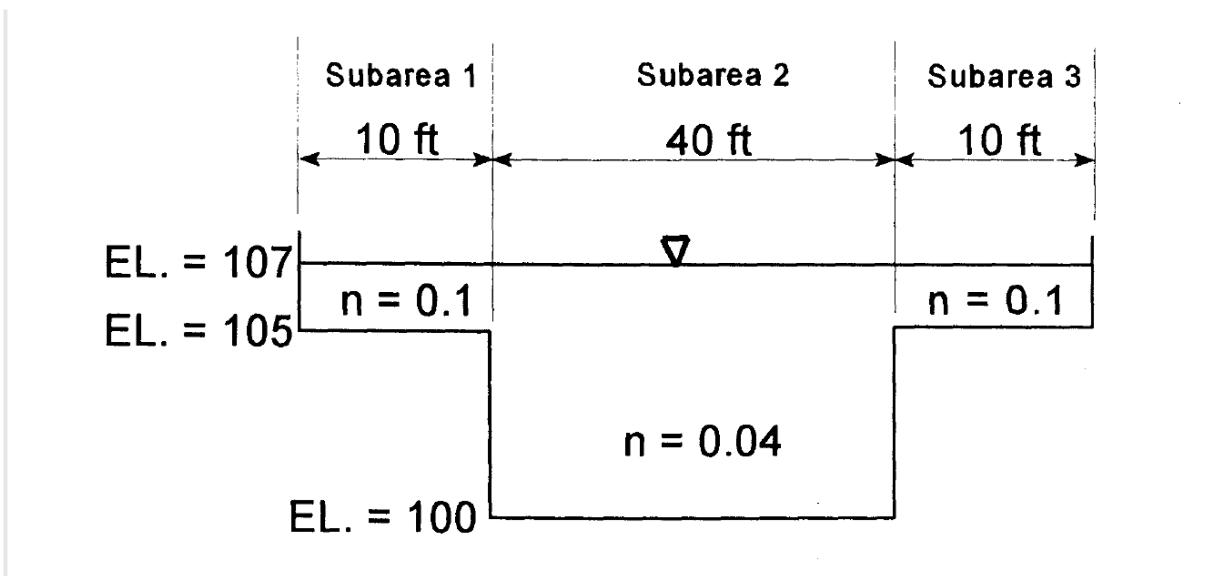
Conveyance is a measure of the ability of a channel to transport flow. In streams of irregular cross section, it is necessary to divide the water area into smaller but more or less regular subsections, assigning an appropriate roughness coefficient to each, and computing the discharge for each subsection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

$$k = \frac{q}{S^{1/2}} = \frac{1.49}{n} a r^{2/3}$$

where:
k = Channel subsection conveyance
q = Subsection discharge, in cubic feet per second
S = Channel bottom slope, in 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 5.2-1 illustrates a conveyance computation of a subdivided cross section.

Example 5.2-1—Computing Conveyance



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

<u>Solution:</u>

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 + 32,890.9 + 419.5
= 33,729.9

b. Assuming the total discharge for the water surface elevation of 107.0 feet in part (a) is 4,000 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:

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

(B) Velocity Head

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

$$h_v = \frac{\propto Q^2}{2g\, A^2}$$

where:

Q = Discharge at the section, in cubic feet per second
h_V = Velocity head, in 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 $(Q/A_1)^2/2g$, does not a give a true measure of the kinetic energy of the flow. You can obtain a weighted average value of the kinetic energy by multiplying the average velocity head above by a kinetic energy coefficient ($\propto_1$) defined as:

$$\propto_1 = \frac{\Sigma\,(qv^2)}{Qv_1^2}$$

where:

$\propto_1$ = Kinetic energy coefficient, before the bridge
q = Discharge in a subsection, in cubic feet per second
v = Average velocity in same 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 5.2-1:

Table 5.2-1: 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

Source: Chow, V.T., 1959, *Open-Channel Hydraulics:* New York, McGraw-Hill.

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.

The energy equation for flow along a channel includes a term for the kinetic energy or velocity head, $V^2/2g$. Use the average velocity, V, for the entire cross section 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, you should first square the differing velocities and then sum them, $v_1^2 + v_2^2 + \ldots + v_n^2$. Since the sum of the squares is greater than the square of the sum, you will need to use the kinetic energy correction factor. This factor usually is 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. Check the alpha values to be sure they are appropriate.

Alpha values typically should stay in the ranges shown in Table 5.2-1. 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. 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.

The following examples illustrate the importance of proper subdivision, as well as the effects of improper subdivision.

Example 5.2-2—Effects of Subdivision on a Panhandle Section
In Figure 5.2-8, 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.

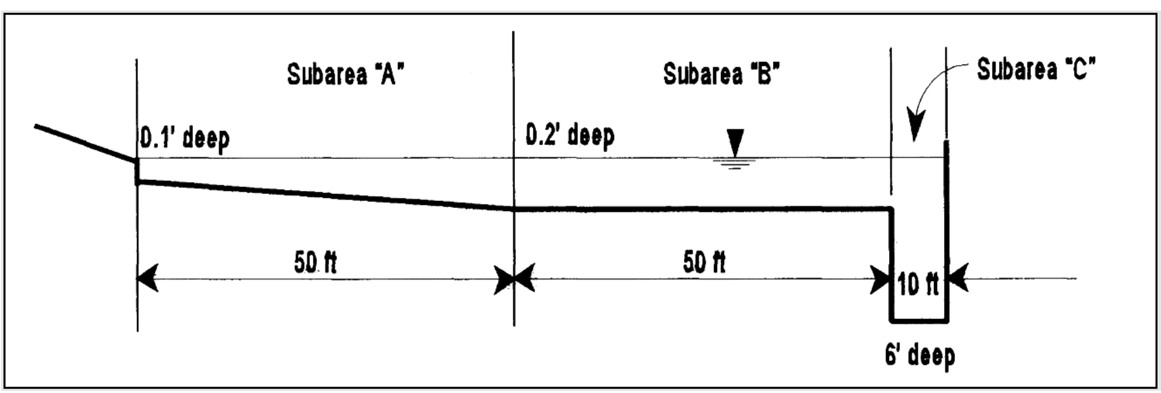


Figure 5.2-8: Effects of Subdivision on a Panhandle Section

Given:

$K = 1.49/n\ (AR^{2/3})$
n is constant over cross section
Factor out 1.49 and compare $AR^{2/3} = K$ feet

Note: K feet varies as to the number of sections selected as a function of R, or more specifically Wp.

Method 1: Consider K_1 feet 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 feet 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 feet as section "C" and ignore sections "A" and "B."

$$K_3' = (6\times10)\left(\frac{60}{5.8+10+6}\right)^{2/3} = \underline{117.8}$$

Method 4: Consider K_4 feet with "A," "B," and "C" treated as independent sections.

$$K_4' = [(6\times10)(60/21.8)^{2/3}] + [(50\times0.2)(10/50)^{2/3}] + [(50\times0.15)(7.5/50.1)^{2/3}]$$

$$K_4' = 117.8+3.4+2.1 = \underline{123.3}$$

Method 1 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 3, which shows conveyance in just the main channel as being greater. Two reasons why method 1 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.

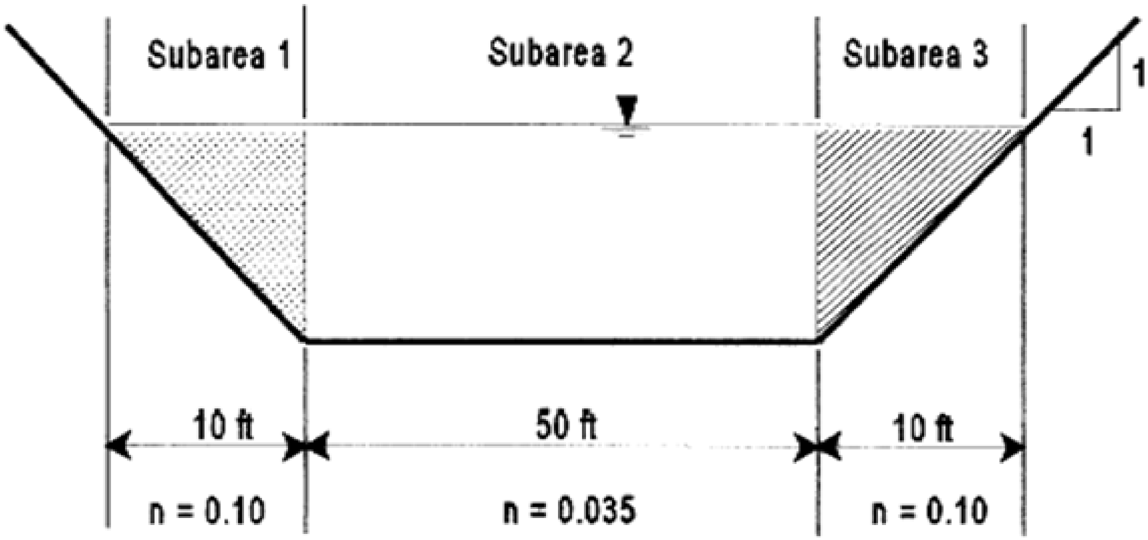
Method 2 is correct. It combines subareas of the channel cross section that have similar hydraulic properties to yield a reasonable answer of total conveyance. If n values between section “A” and “B” were significantly different, combining them to determine conveyance might not provide the desired accuracy.

Method 3 is incorrect but exemplifies how easily you 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 4 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 5.2-3 Effects of Subdivision on a Trapezoidal Section

In Figure 5.2-9, 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 5.2-9, 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 5.2-9: Effects of Subdivision on a Trapezoidal Section

(C) Friction Losses

Compute the friction loss as follows:

$$h_f = L\ S_f$$

where:
L = Flow length, in feet
S_f = Average friction slope, in feet/feet

You can calculate the average friction slope 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:

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

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

(D) Expansion/Contraction Losses

Expansion Losses
Compute the expansion loss as follows:

$$h_e = k_e\,(h_{v2} - h_{v1})$$

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

Compute the contraction loss as follows:

$$h_c = k_c \left(h_{v2} - h_{v1}\right)$$

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.

(E) 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 the total energy head at the downstream section plus any energy losses that occur between the two sections.

Energy Equation

Write the energy equation between two adjacent cross sections as follows:

$$h_1 + h_{v1} = h_2 + h_{v2} + h_f + h_e + h_c$$

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

It is not possible to find a direct solution of the above equation when either h_1 or h_2 is unknown, since the associated velocity head and the energy loss terms also are then 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:

$$\Delta H = (h_1 + h_{v1}) - (h_2 + h_{v2} + h_f + h_e + h_c)$$

Use successive estimates of unknown elevations 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.

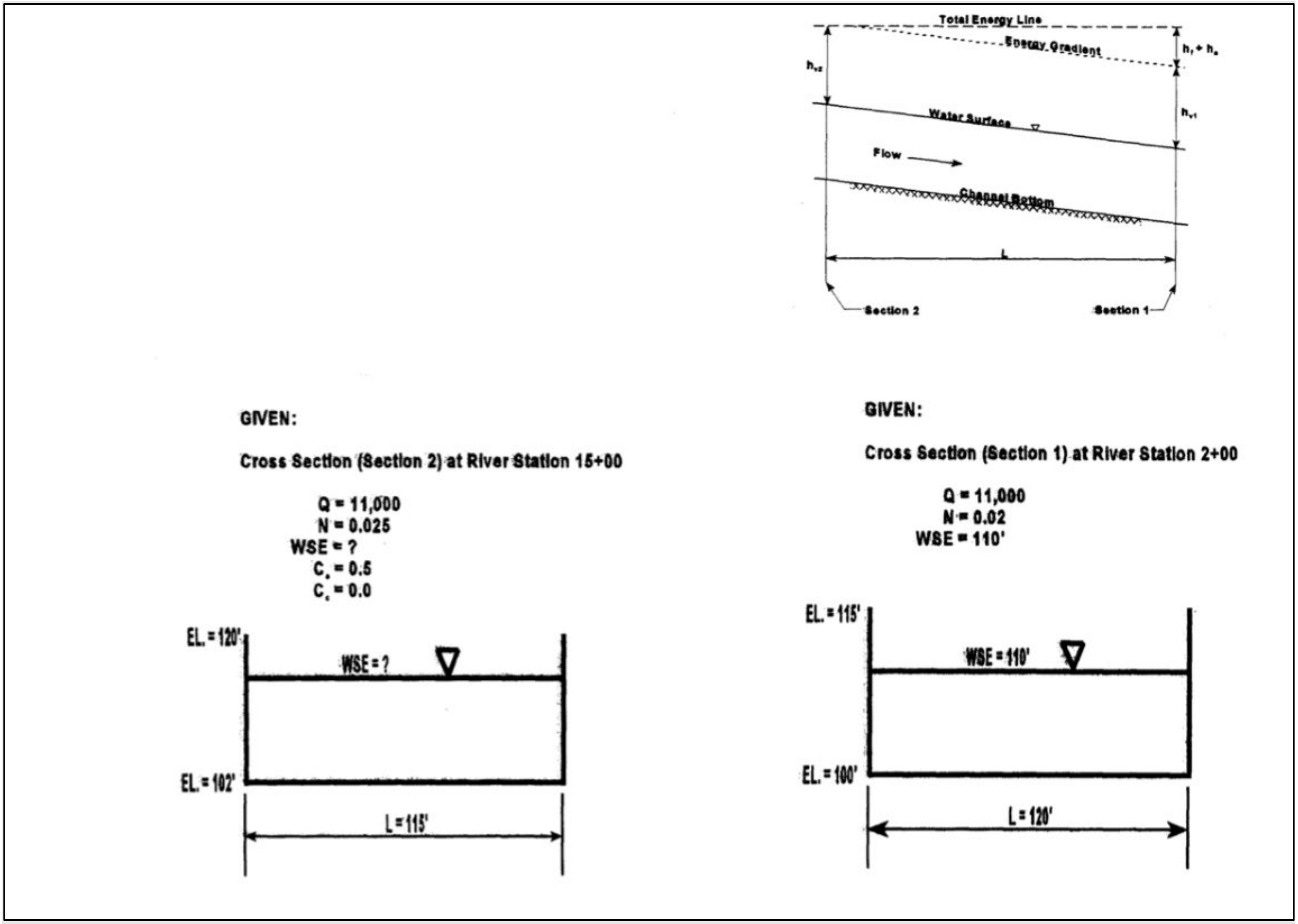
Step 4: Determine energy losses that correspond to estimated water surface elevation.

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.

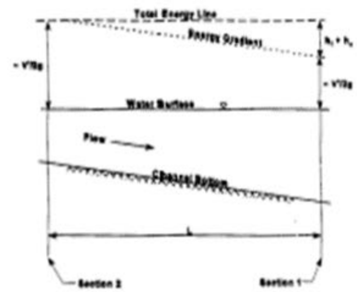
Example 5.2-4 illustrates a step backwater computation. Descriptions of conveyance, velocity head, friction loss computations, and expansion and contraction losses are provided after the example.



Example 5.2-4: Standard Step Backwater Computation

$Q = \underline{11{,}000\ \text{cfs}}$

$C_c = \underline{\ 0.0\ }\quad C_e = \underline{\ 0.5\ }$



Cross Section No.	Water Surface Elevation		Area (A)	Hydraulic Radius (R)	$R^{2/3}$	n	K	$\bar{S}_f$	L	h_f	K^3/A^2 (10^9)	ALPHA (α)	V	$\alpha \frac{V^2}{2g}$	$\Delta(\frac{\alpha V^2}{2g})$	h_e	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) $\bar{S}_f = \dfrac{[0.5\,(Q_1 + Q_2)]^2}{K_1 K_2}$

(11) $h_f = L\,\bar{S}_f$

(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) $h_e = C_e \left|\Delta(\alpha V^2/2g)\right|$ for $\Delta(\alpha V^2/2g) < 0$

(17b) $h_e = C_c \left|\Delta(\alpha V^2/2g)\right|$ for $\Delta(\alpha V^2/2g) > 0$

(18) Δ(water surface elevation) = $\Delta(\alpha V^2/2g) + h_f + h_e$ = Columns (16) + (11) + (17)

Example 5.2-4: Standard Step Backwater Computation (continued)

5.2.4.3.2 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 5.2-10 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 5.2-11 and Figure 5.2-12 display examples of finite difference and finite element model meshes.

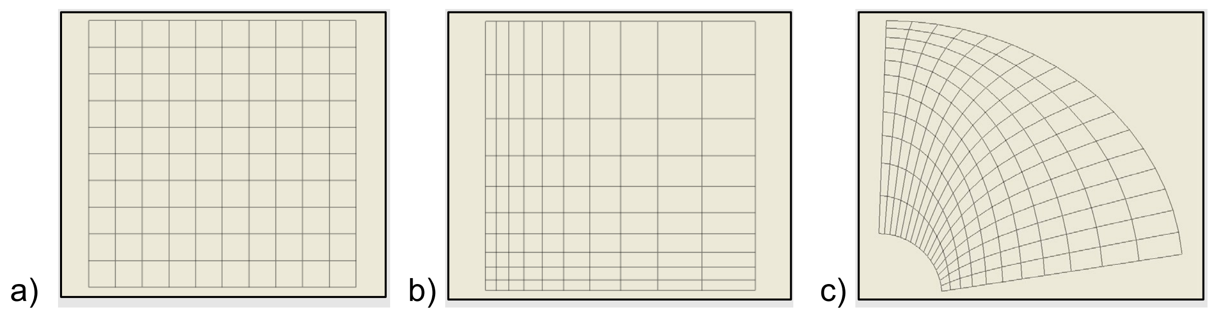


Figure 5.2-10: Example of (a) Cartesian, (b) Rectilinear, and (c) Curvilinear Grids

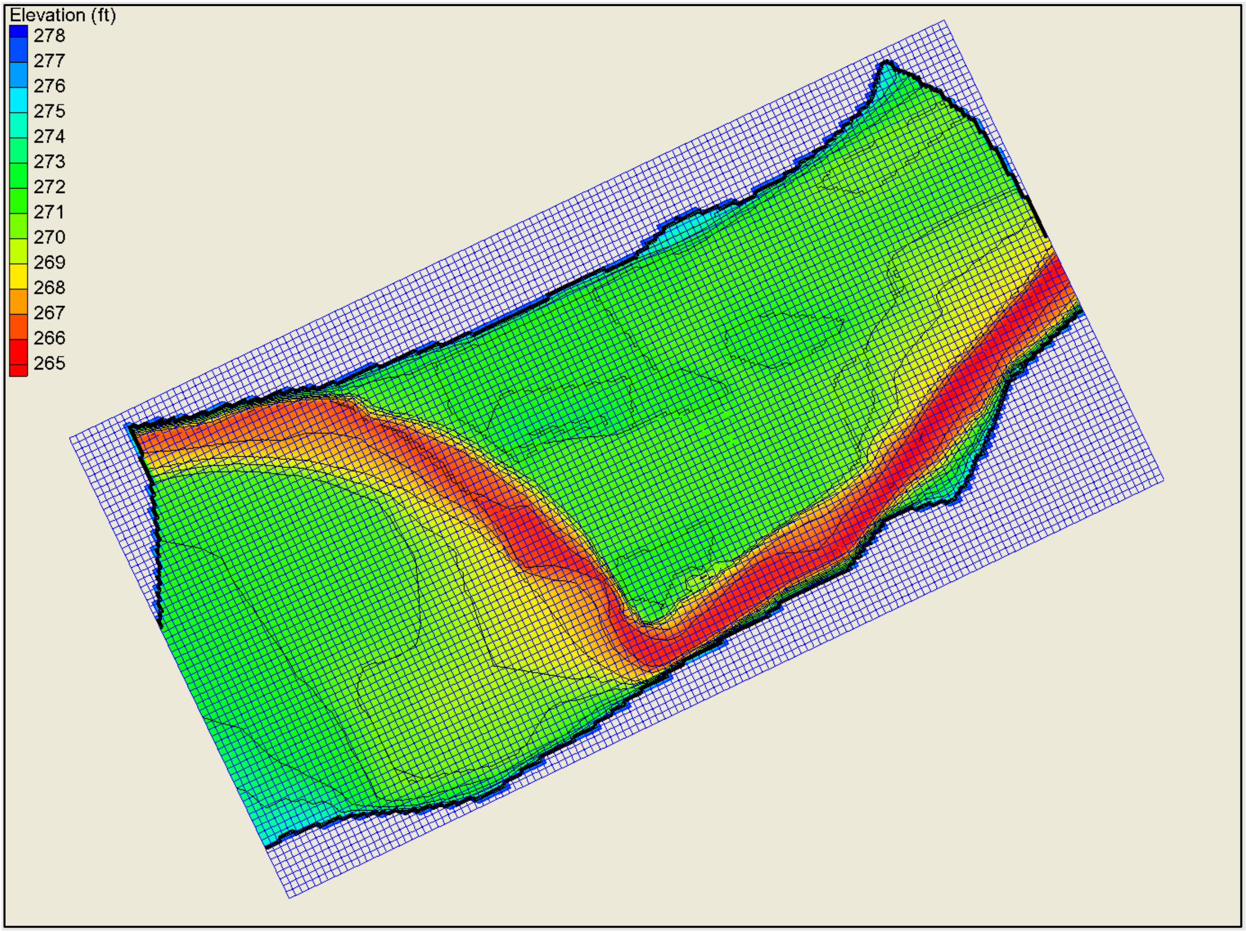


Figure 5.2-11: Example of a Finite Difference Model Mesh

After defining the model domain, the next step in the model geometry development is to specify element locations, sizes, and orientation. In other words, specify the resolution of the model. Finite element models typically will 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 often is 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 5.2-12 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 oftentimes is one of the model parameters that is modified to achieve both model stability and model calibration.

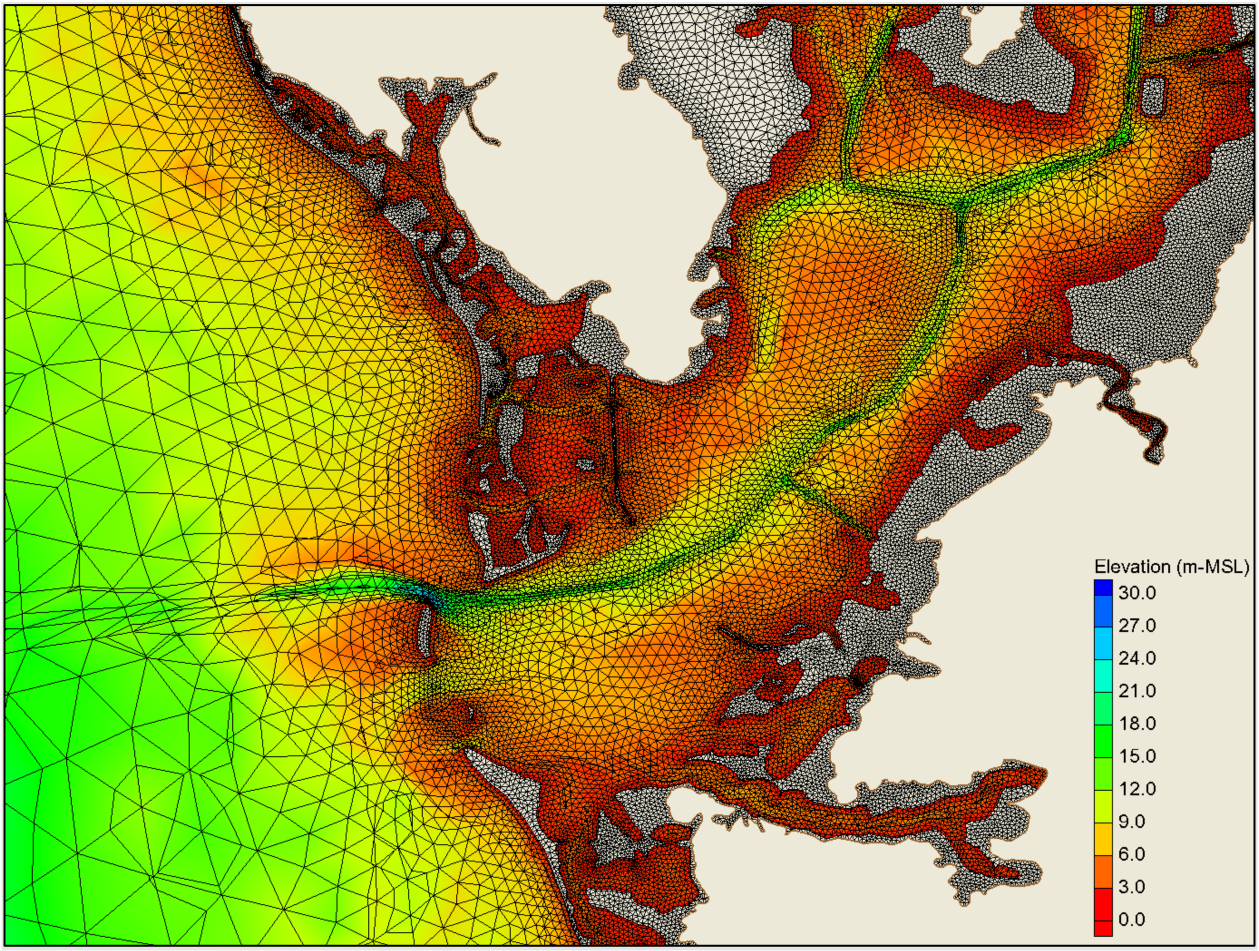


Figure 5.2-12: Example of a Finite Element Model Mesh

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.

5.2.4.4 Boundary Conditions

Upstream Flow
For a riverine analysis, give the flow at the upstream boundary. For a steady-state analysis, specify the peak discharge for each frequency at the upstream boundary. For an unsteady flow analysis, specify a flow hydrograph at the upstream boundary.

Downstream Stage
Specify the stage at the downstream cross section. 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.

You can use normal depth in many cases when the stream channel is nearly uniform for a fairly long reach. You can use HEC-RAS or WSPRO to compute the normal depth by providing an energy slope equal to the channel slope. This method also is known as "slope conveyance." Determine the channel slope using a USGS Quadrangle Map. Determine the slope below the last downstream cross section where contour lines cross the stream channel. You can use other estimates of energy slope; 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, you should use "convergence."

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. Estimate the distance as the downstream study length described in Section 5.2.4.1.

Determine convergence as follows:

a. Make trial-and-error calculations assuming a range of water surface elevations. This assumed range of water surface elevations should bracket your best guess of the water surface elevation at the farthest 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 farthest 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 between which the water surface should be, and the other two

should represent the range outside of which the water surface is unlikely to be. Refer to Figure 5.2-13.

c. The computed profiles will converge toward the true profile. The profiles should converge within an acceptable tolerance by the first section of interest in the reach (see Figure 5.2-13). If the profiles do not adequately converge, then you should obtain additional geometric data downstream.

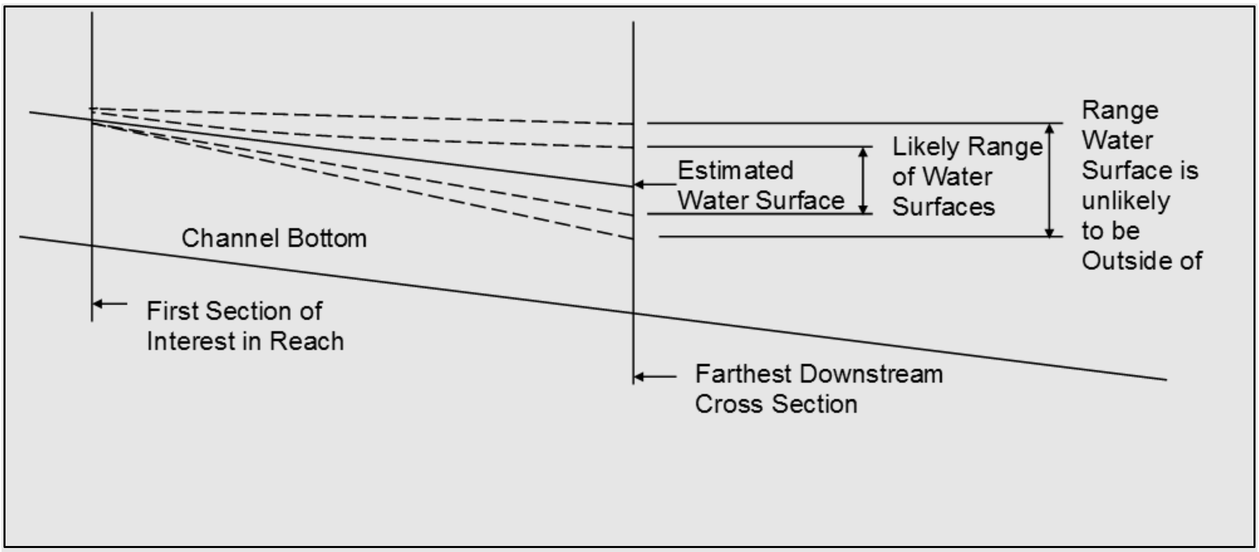
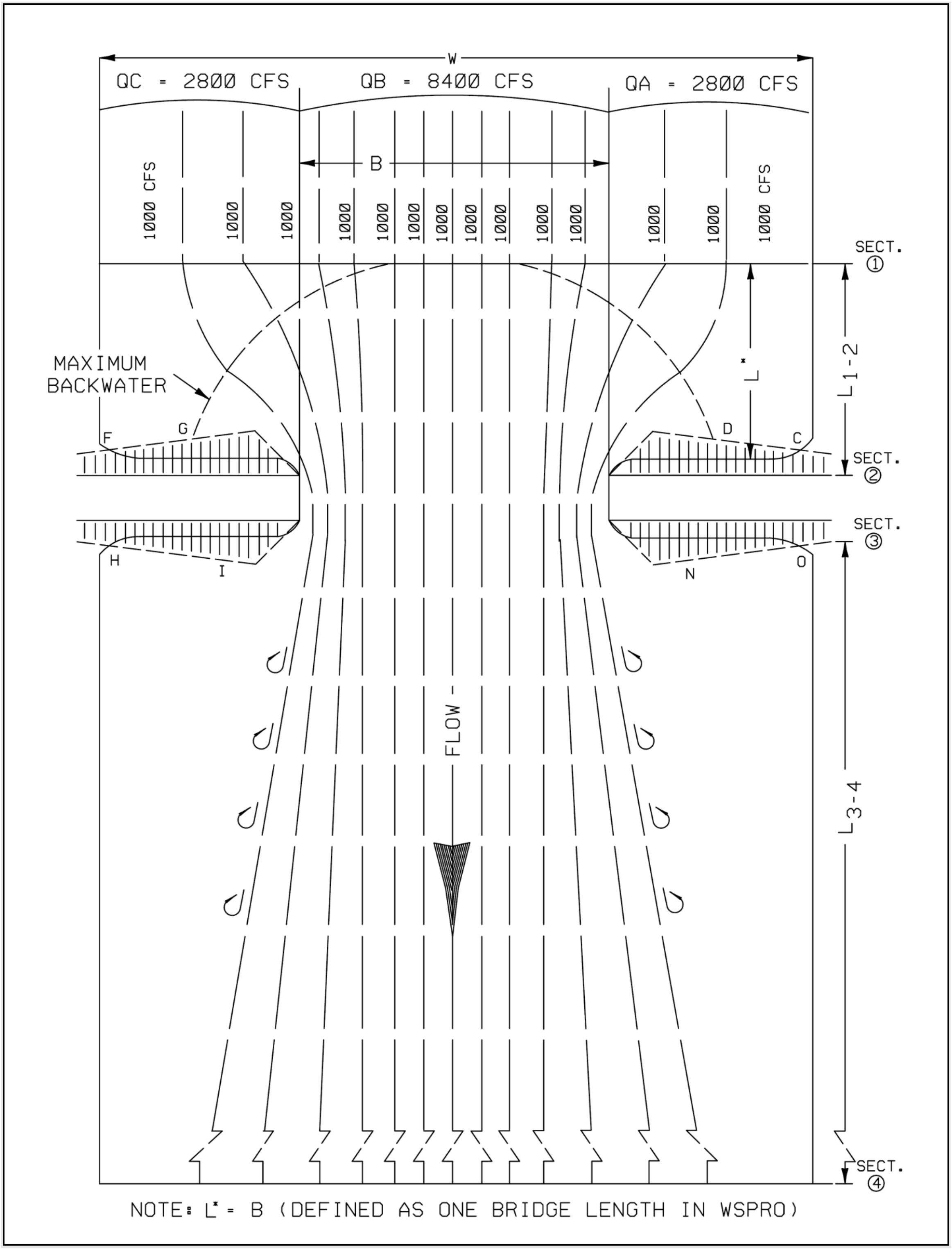


Figure 5.2-13: Convergence Profiles

5.2.4.5 Bridge Model

Flow Characteristics at Bridges
Figure 5.2-14 illustrates the manner in which flow contracts in passing through the channel constriction. The flow bounded by each adjacent pair of streamlines is the same (1,000 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 re-established.

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 5.2-15 (Part A). The dashed line labeled "normal water surface" represents the normal stage of the stream for a given discharge before constricting the channel. The solid line labeled "actual water surface" represents the nature of the water surface after constriction of the channel. 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 bridge backwater.



Reference: USDOT, FHWA HDS-1 (1978)

Figure 5.2-14: Flow Lines for Typical Bridge Crossing

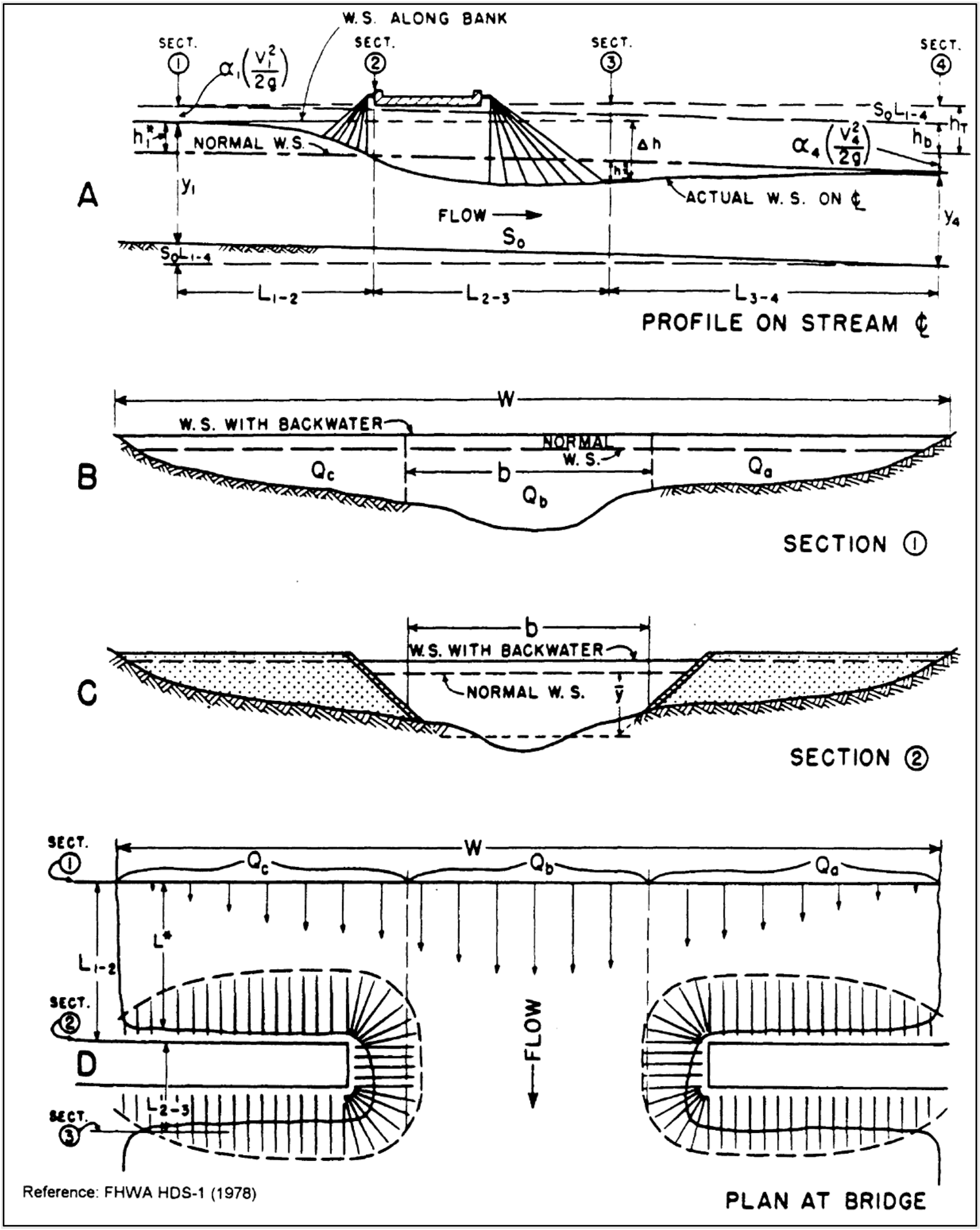


Figure 5.2-15: Normal Crossings: Spill-through Abutments

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 often is the same through the bridge from upstream to downstream. The most common reason that the roughness will change is if there is a significant extent of rubble riprap protecting the piers or channel banks.

Many Florida floodplains are heavily vegetated. Many riverine bridges span a significant length across the floodplain. The area beneath the bridge often is 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 5.2.4.3) usually would recommend against subdividing at the toe of the abutment, so a weighted roughness should be determined.

Be careful to model abrupt changes in roughness 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 good 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.

Bridge Routine
Refer to HEC-RAS documentation for cross section location information. However, if you are using the WSPRO bridge routine when modeling in HEC-RAS, don’t follow the documentation; instead, use the following recommendations.

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 section 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.

<u>Exit Section:</u>
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 5.2-16.

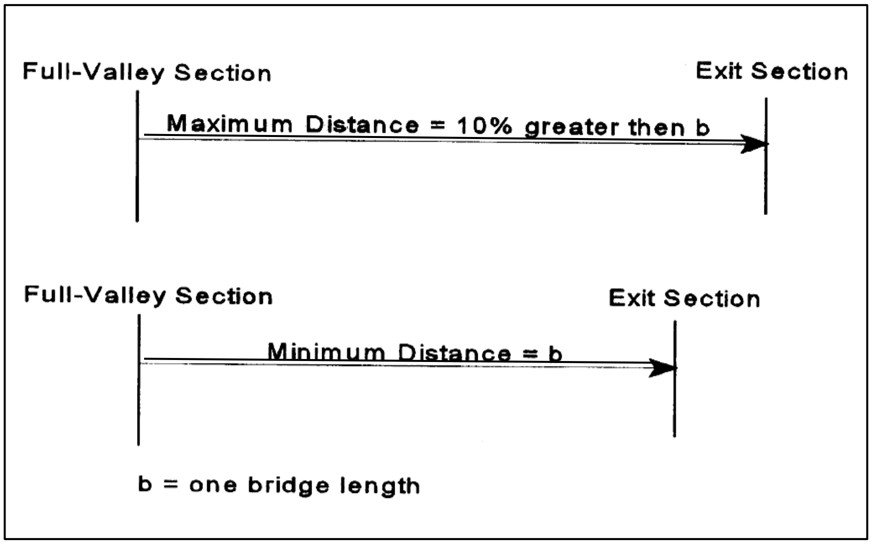


Figure 5.2-16: Location of Exit Section

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 5.2-17.

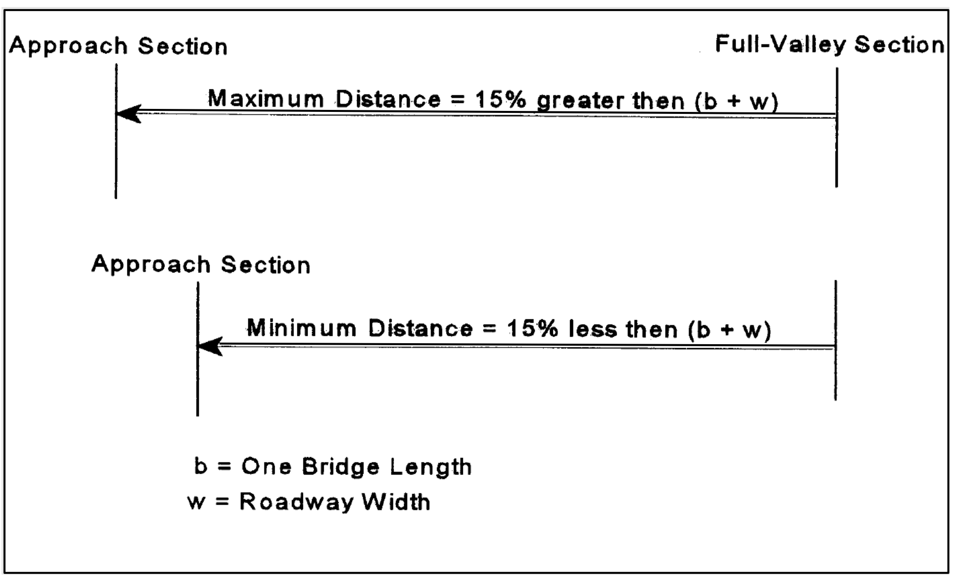


Figure 5.2-17: Location of Approach Section

If, for some reason, it is impossible to follow the cross section requirements, you may need to analyze the site without using the bridge routine.

Piers
You can model single-row pile-bent bridges 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.

5.2.5 Simulations

5.2.5.1 Calibration

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

Observed field data for a flood event can include:

- Water surface elevations
- Discharge measurements
- Velocity measurements

Obtain data from multiple flood events, 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, the most reliable source of information is gage data. Most gages used in riverine situations measure the water surface elevation. Figure 5.2-18 shows a simple staff gage that you must observe and record manually. More complex gaging stations will record stages automatically and either store the records for later download or transmit the data using telemetry.

Figure 5.2-18: Staff Gage on the Suwannee River

You can determine discharges indirectly from the water surface elevations. Traditionally, you would use a velocity meter to take measurements at intervals across the stream and then determine the discharge, as shown in Figure 5.2-19. When you have determined the discharge at enough different water surface elevations, you can establish a stage versus discharge relationship for the gage.

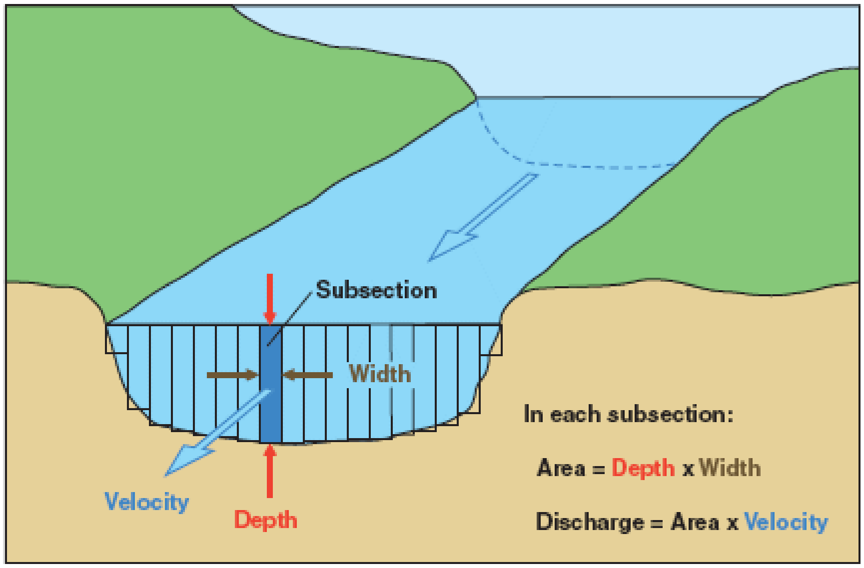


Figure 5.2-19: Discharge Determination with a Velocity Meter (from USGS Streamgaging Fact Sheet 2005-3131, March 2007)

More recently, discharges have been measured on some larger rivers with an Acoustic Doppler Current Profiler mounted on a boat (see Figure 5.2-20).

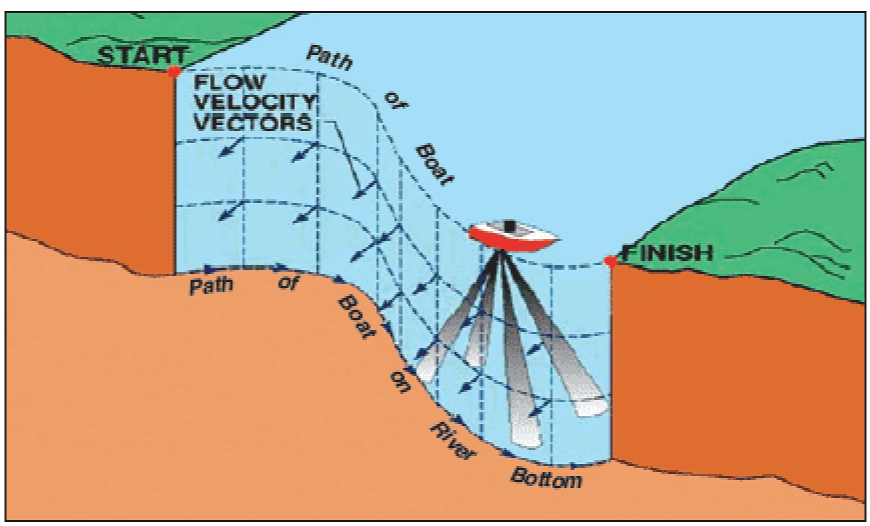


Figure 5.2-20: Discharge Determination with an Acoustic Doppler Current Profiler (from USGS Streamgaging Fact Sheet 2005-3131, March 2007)

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

If gage data are unavailable, consider sending survey out to measure:

- High water marks associated with known floods (Figure 5.2-21)
- Local resident or official high water permanent markers/signs (Figure 5.2-22)
- 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 5.2-21: 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 are 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 5.2-22: Local Resident indicating Flood Level on the Caloosahatchee River near LaBelle in 1913

After you obtain available gage data and/or high water mark elevations, the next step is to develop the hydraulic model for the existing site conditions. In some situations, this might entail creating multiple existing-condition models if the site conditions have changed since some of the calibration floods. Develop the model using standard guidance for the coefficients used in the model. Then compare the initial model results to the high water marks and adjust the coefficients. The common coefficients to adjust are:

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

Manning's roughness coefficient is the basic adjustment tool for unobstructed reaches. Considerable uncertainty exists when estimating roughness values. Estimates by experienced hydraulics engineers often vary by ± 20 percent (from USACE EM 1110-2-1416). If you hold the channel roughness constant and vary the overbank roughness, you should be well served.

Also, remember that Manning's roughness varies with depth, which can affect calibration as follows:

- As the depth over the roughness elements increases, 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.

Do not adjust the calibration coefficients outside of their normal ranges. If the calibration attempts are not acceptable, re-examine 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
- Accuracy of survey data or other geometric data
- Datums of geometric data
- Flow lengths
- Warning messages

Note that calibration problems can be caused by different issues. Use your best judgment in the calibration process. There is no universally accepted procedure or criterion 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 essentially can 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 you must convert the measured velocity to a depth-averaged velocity for comparison. Set the value high first, and then lower it until you obtain the ideal velocity distribution. 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.

5.2.5.2 Existing Conditions

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

5.2.5.3 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. Identify adverse hydraulic conditions in the Location Hydraulics Study for consideration when planning the roadway crossing. The final location will not depend solely on hydraulic aspects, but consider them 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 still may be possible.

Usually, you will evaluate and select the length of the bridge and the location of the abutments 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 that will be 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 usually can 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 10 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.

5.3 TIDAL ANALYSIS

A qualified coastal engineer should perform hydraulic and scour analyses of tidal and tidally influenced bridges. Section 5.1.1 defines the requirements and credentials of coastal engineers qualified to perform tidal analyses for the Department.

5.3.1 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.

5.3.1.1 Survey Data

You will need survey data to perform several aspects of a bridge hydraulics and scour analysis. Survey data not only provide the elevation data to construct hydraulic and wave models, but also provide 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 for riverine studies. Since new survey acquisition of the required data over the entire domain is rarely cost-effective, you can supplement survey data acquired around the bridge with publicly 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 5.3-1
- 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 (https://coast.noaa.gov/digitalcoast/data/coastallidar.html)

Be careful when combining data from several sources. There can be wide ranges in accuracy due to differing measurement techniques and survey dates. Pay close attention to conversion between different horizontal and vertical coordinate systems. Examine boundaries between survey data sets for inconsistencies and corrections.

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

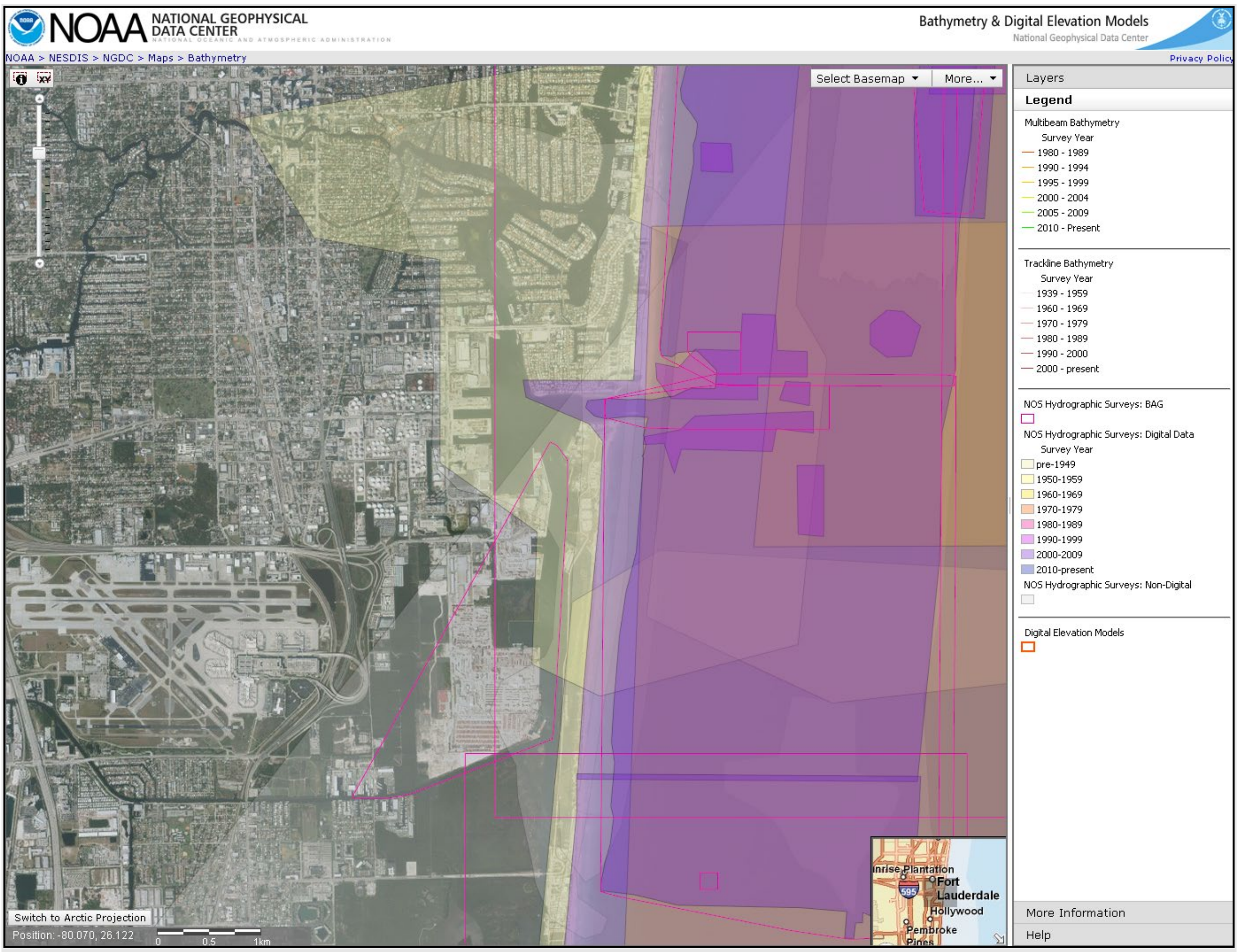


Figure 5.3-1: NOAA National Geophysical Data Center Website

5.3.1.2 Geotechnical Data

To calculate scour at bridge foundations, you will need geotechnical information 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 5.2.1.2 for a discussion of geotechnical data requirements.

5.3.1.3 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 data for characterization of the hurricane vulnerability through the hurricane history.

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 (MHW), 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. To be accessible when needed, these datums are referenced to fixed points known as bench marks. NOAA maintains numerous tidal bench marks throughout the state of Florida that are available from the Center for Operational Oceanographic Products and Services (CO-OPS) website (https://tidesandcurrents.noaa.gov/products). The Florida Department of Environmental Protection (FDEP) is an additional source for 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 you should document them 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 5.3-2 and Table 5.3-1 display an example of tidal bench mark information and gage data (with tidal datums) for Key West, Florida.

Table 5.3-1: 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

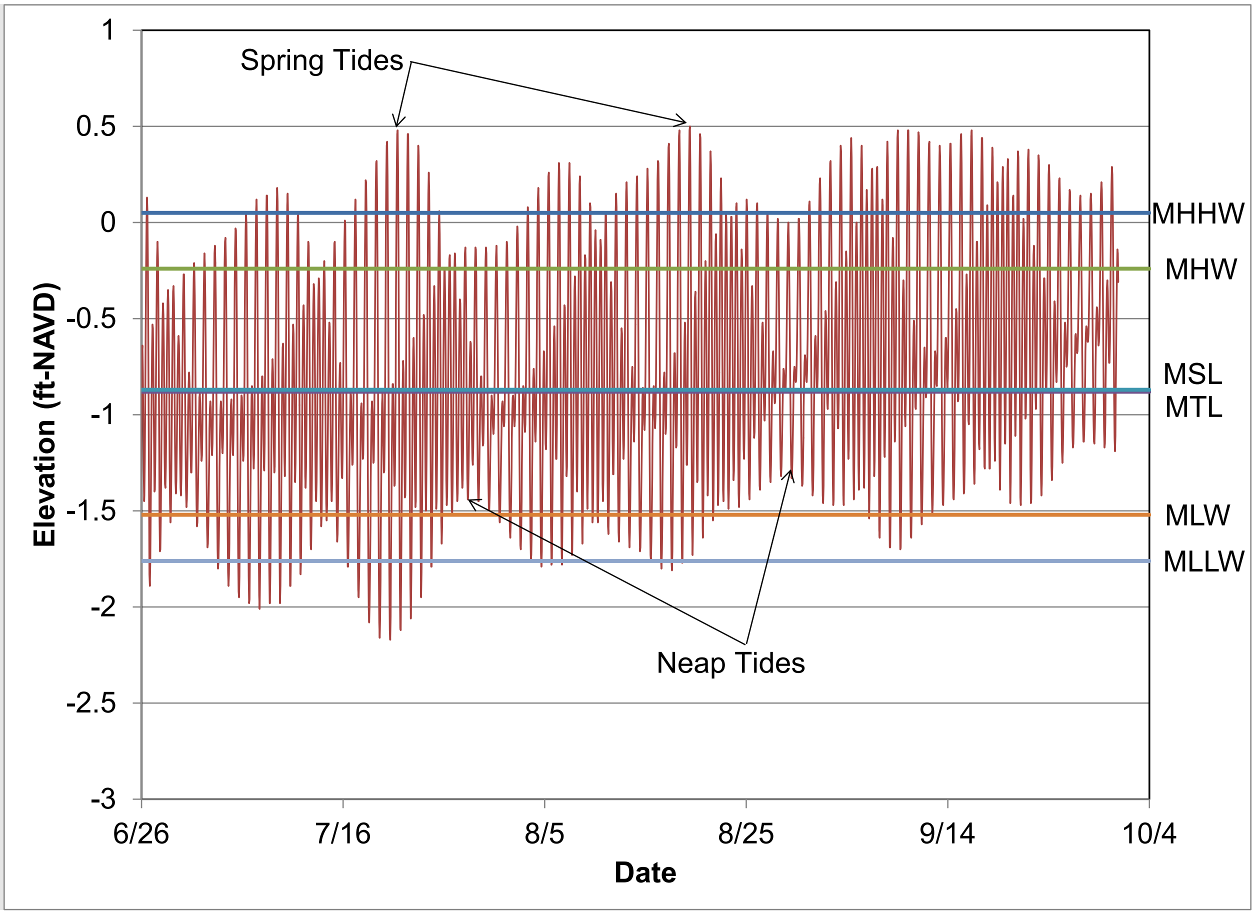


Figure 5.3-2: Measured Tides at Key West and Tidal Datums

Gage Measurements
Gage measurements provide information both for model calibration and model boundary conditions. Several sources of gage data are available to the public. 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.

You also can employ tide gage data for development of model boundary conditions, as well as for model calibration. Tide gages record stages at a fixed location in tidally influenced areas. NOAA maintains gages throughout the state. You can access both recent and historic data online (https://tidesandcurrents.noaa.gov/water_level_info.html). In Florida, the site provides data at 29 active stations (Figure 5.3-3) and historic data at 722 locations.

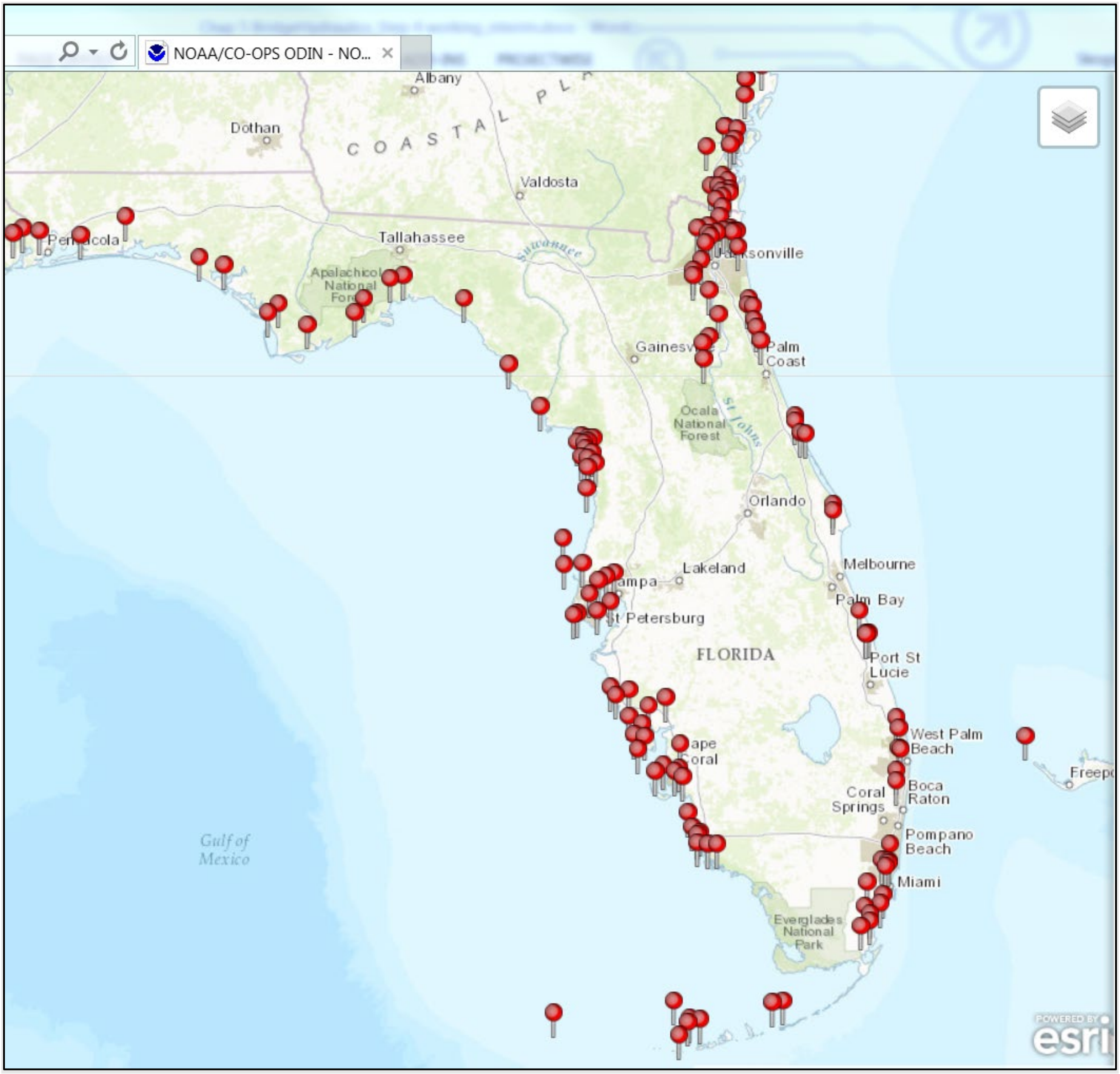


Figure 5.3-3: Location of Florida's Active Tide Stations Maintained by NOAA (Source: https://tidesandcurrents.noaa.gov/map/index.html?region=Florida)

Used to calibrate data for wave models, wave gage data typically is much more rare 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 measure 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 5.3-4). Figure 5.3-5 provides an example of these data as time series of significant wave heights. 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.

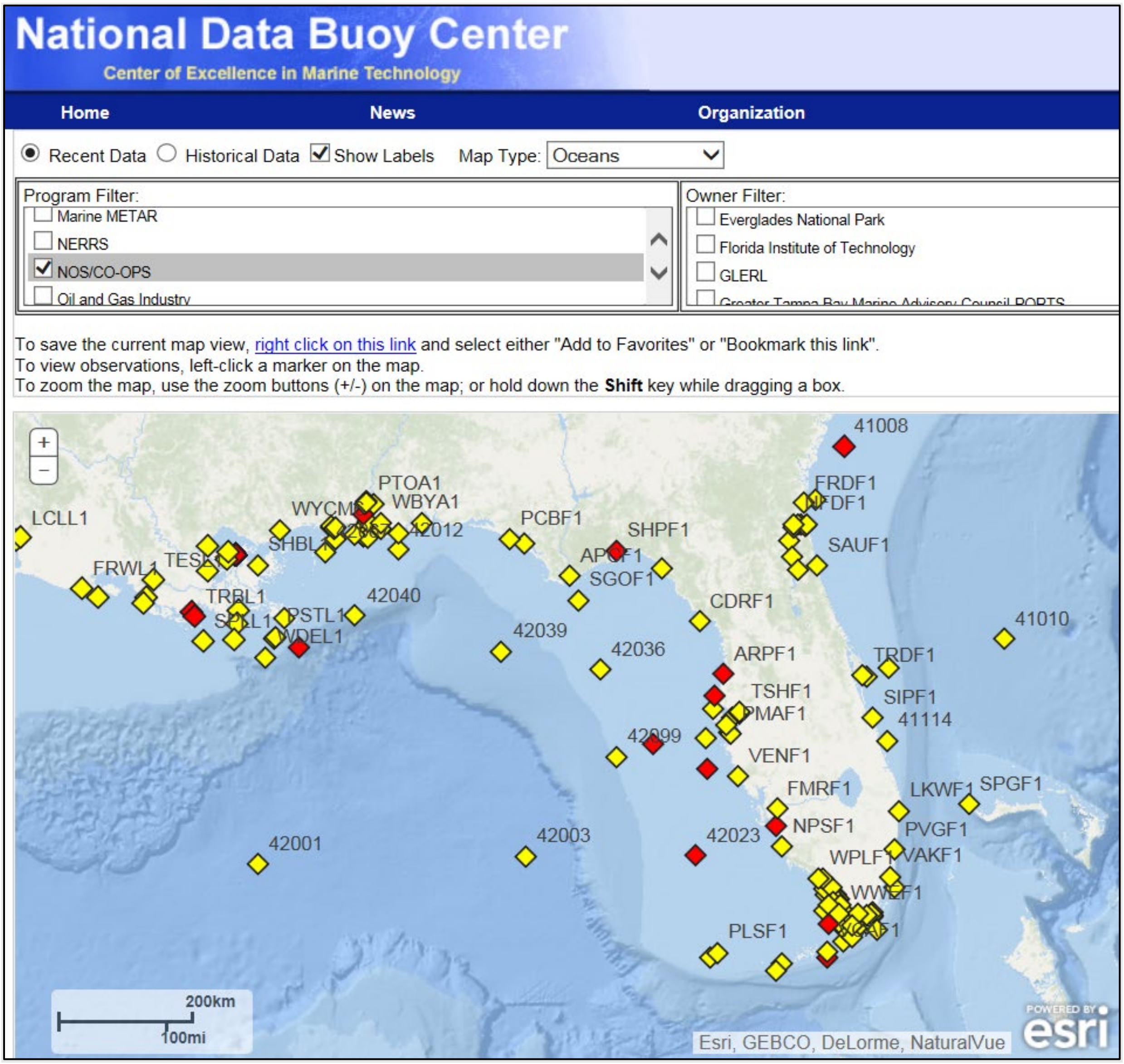


Figure 5.3-4: Locations of NDBC Stations around Florida
(Source: http://www.ndbc.noaa.gov/)

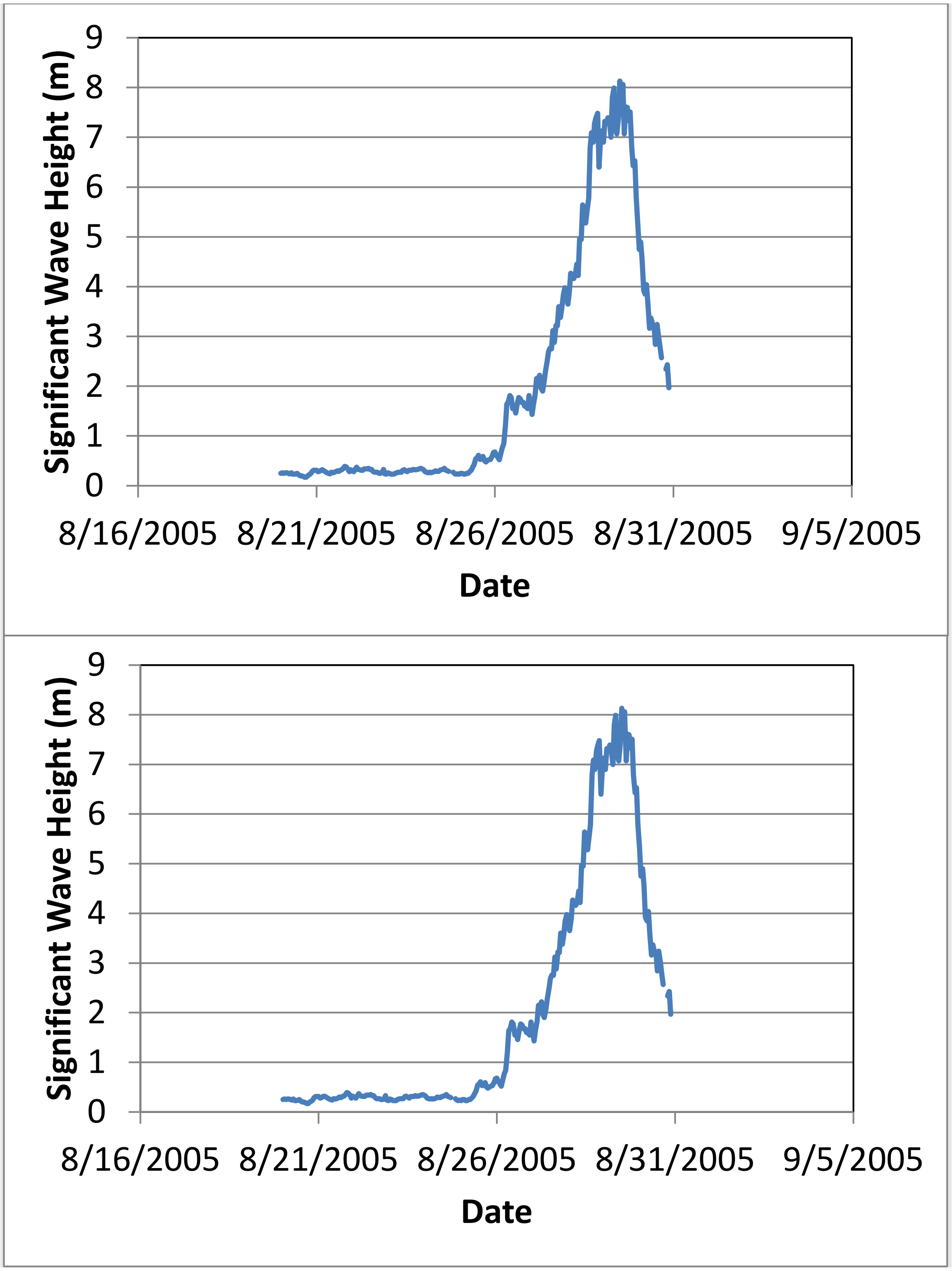


Figure 5.3-5: Example of Wave Gage Data at NDBC Station 42039 during Passage of Hurricane Katrina

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. FEMA typically performs post-storm damage assessments. Although the survey accuracy has significantly increased in recent years, be cautious when using these data. Coastal high water marks typically are 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 run-up—represents the height of water rise above the stillwater level due to water running up from a breaking wave

You often can find high water marks near each other and they 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. When the crest of the wave rides on the surge, this creates coastal wave height flooding. Thus, differences will occur between high water marks measured on the interior and exterior walls of a structure. Finally, wave run-up 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 run-up. Wave run-up 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 run-up).

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 a 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. The BHR should 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 5.3-6 and Table 5.3-2 (from https://coast.noaa.gov/hurricanes/).

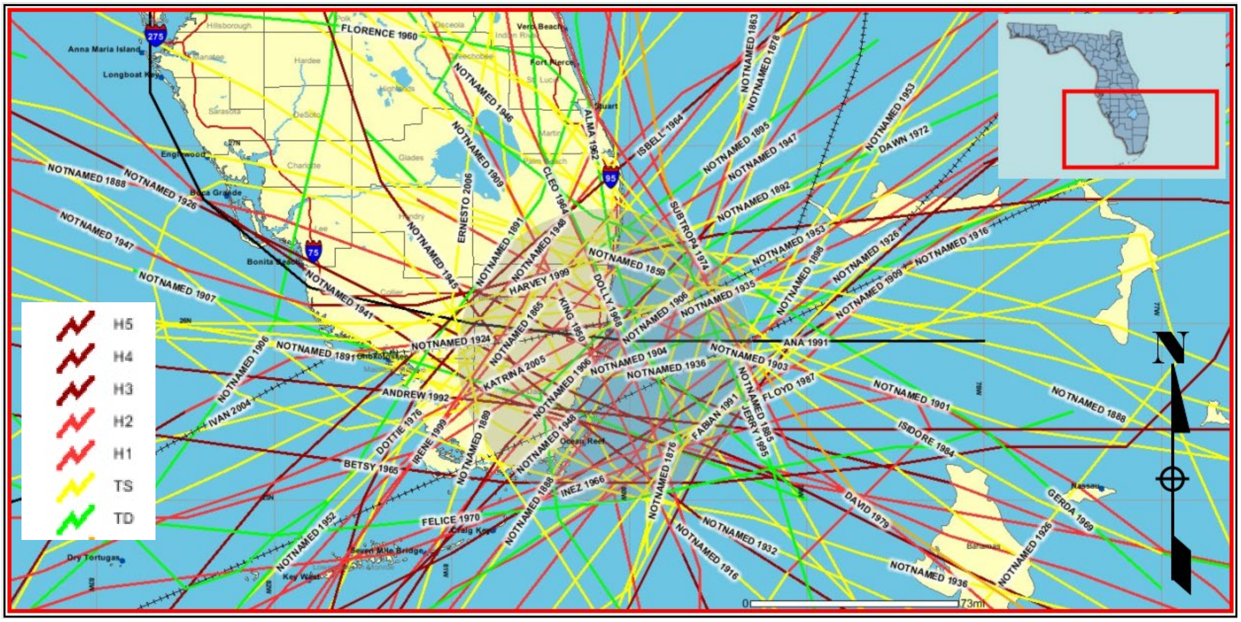


Figure 5.3-6: Hurricane and Tropical Storm Tracks Passing within 50 Nautical Miles (nmi) of Miami (Source: NHC)

Table 5.3-2: 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

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 5.3-7 and Figure 5.3-8. From the figures, changes in shoreline location occur south of the east abutment as well as to the spit south of the inlet. Section 5.5.1.1 further discusses calculation of long-term trends. Sources of historical aerial photography are the same as those discussed in Section 5.2.1.3.

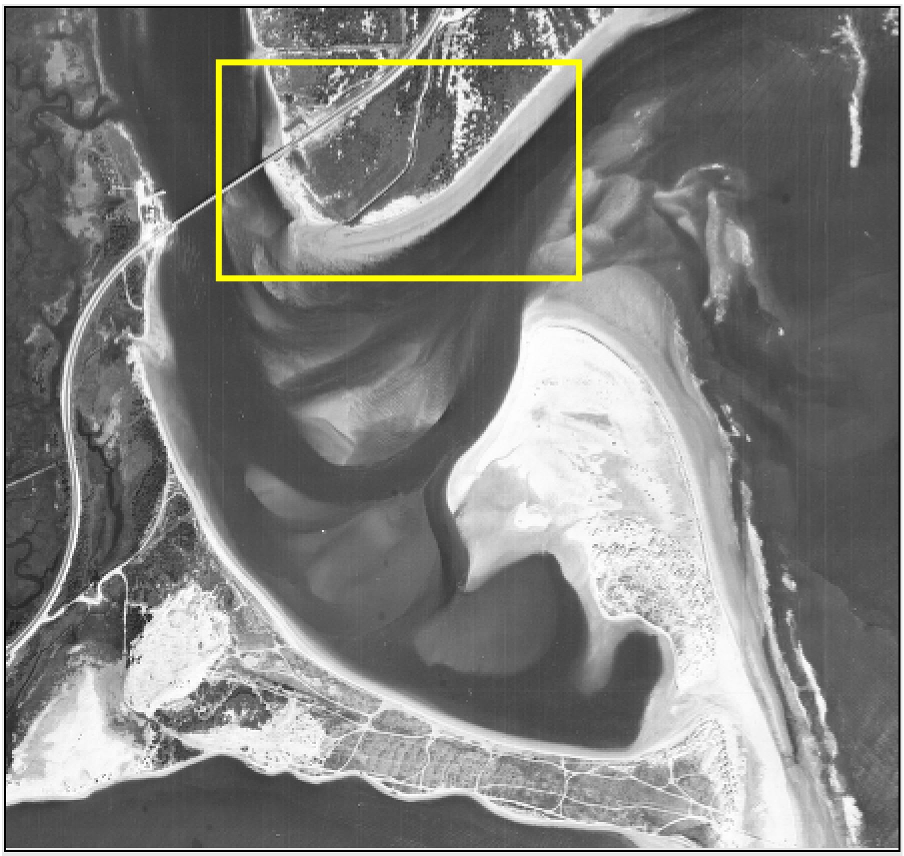


Figure 5.3-7: Heckscher Drive (SR-A1A) near Ft. George Inlet in 1969

Figure 5.3-8: Heckscher Drive (SR-A1A) near Ft. George Inlet in 2000

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 5.2.1.3 for additional discussion on obtaining and using these reports in hydraulic analyses.

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 provide a source of decades-long wave data that can provide boundary conditions or calibration data for nearshore wave modeling. The data include hourly wave parameters of significant wave height, peak period, mean period, mean wave direction, and wind speed and direction (Figure 5.3-9). The database includes both nearshore and offshore gages along both Florida's Atlantic Ocean and Gulf of Mexico shorelines. The data are available via the following link: https://cirp.usace.army.mil/products/cms.php

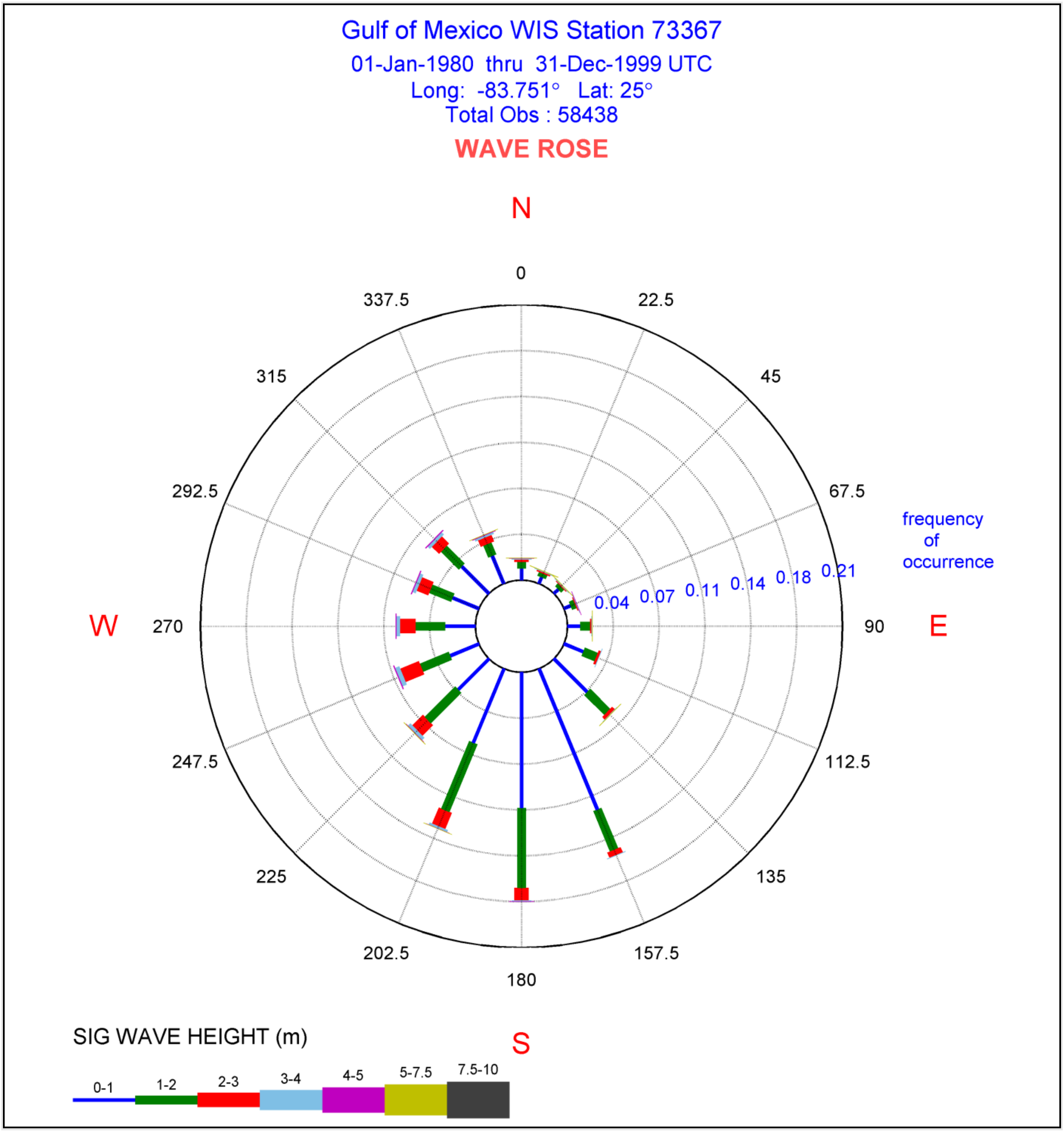


Figure 5.3-9: Example of Available WIS Data from:
https://cirp.usace.army.mil/products/cms.php

Previous Studies
Previously performed studies of a waterway can provide additional sources of data. Refer to Section 5.2.1.3 for sources and discussion of previous studies.

5.3.1.4 FEMA Maps

FEMA Flood Insurance Rate Maps (FIRM) are the official maps of communities that display the floodplains—specifically, special hazard areas and risk premium zones—as delineated by FEMA. They are 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 5.1.2). 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 model 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 1980s. Thus, you can attribute deviation in 100-year flood elevations from the published FEMA values to differences in the numerical models, boundary conditions, inclusion of wave setup, as well as in the post-simulation analysis. More recently, FEMA has begun to perform 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.

5.3.1.5 Inland Controls

Data collection for inland controls follows the same recommendations as for the upstream controls of riverine analysis (see Section 5.2.1.6).

5.3.1.6 Site Investigation

You should plan to do a field investigation for all new bridge construction. Refer to Section 5.2.1.7 for a detailed list outlining key items you should collect during site investigations. In addition to this list, data collection at tidal bridges also should 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.

5.3.2 Hydrology (Hurricane Rainfall)

During hurricane events associated with heavy rainfall, you can experience significant surface runoff from land. 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 USACE reference, *Engineering and Design Storm Surge Analysis EM 1110-2-1412* (1986), provides a methodology for estimating rainfall associated with landfalling hurricanes. The methodology applies to 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 varies uniformly along the coast for any given storm. Also, the rainfall depths are uniform along any line parallel to the storm track extending across the 25-mile-wide zone. The reference provides point rainfall graphs (Figure 5.3-10) for selected frequency levels at either 6-hour 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, you may assume a steady 10-year discharge over the duration of the surge. This is likely to be conservative in light of a recent examination of hurricane rainfall in North Carolina that suggests that a two-year rainfall well represented historical storms in that state (OEA, 2011). Bridges over streams with short times of concentration (< four 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, you should perform a sensitivity study to characterize the influence of the runoff magnitude on the flow properties at a subject bridge during a surge event.

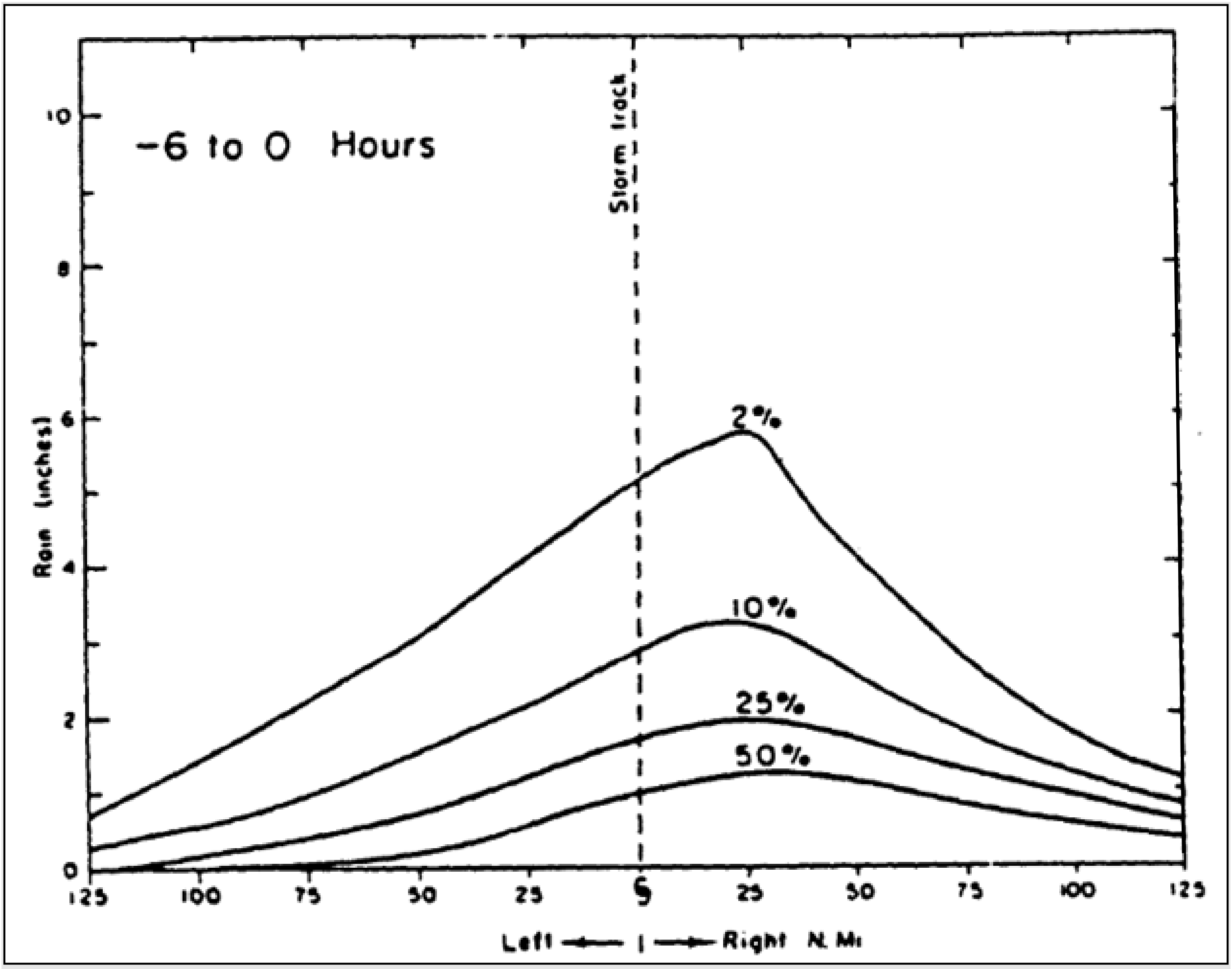


Figure 5.3-10: Rainfall for Selected Frequency Levels for Six Hours before Landfall (Source: USACE 1986)

5.3.3 Model Selection

If you perform hydraulic studies, you 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

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, including site conditions, design elements, available resources, and project constraints. The utility of the decision tool is that it presents a formal procedure to apply for the selection of the appropriate model rather than an intuitive process.

Figure 5.3-11 presents an example where an 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 5.3-11: Example of Model Selection Worksheet from NCHRP Web-Only Document 106

For tidal analyses, in general, one-dimensional modeling works well 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, there is not currently a similar selection procedure available. Selecting the appropriate model is left to your experience and discretion after carefully weighing the required design criteria and model features. Confirm your final model selection with the District Drainage Engineer.

5.3.3.1 Storm Surge Model

Developing design hydraulic parameters at a bridge location requires the model 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-dimensional 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

5.3.3.2 Wave Model

You can use either numerical models or deterministic methods in developing design wave climate parameters. The USACE references *Coastal Engineering Manual* (2002) and *Shore Protection Manual* (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)

5.3.3.3 Model Coupling

Model coupling refers to the interaction between the wave model and the surge model when simulating hurricanes. With no coupling, the surge and wave models run 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 5.3-12, taken from Sheppard et al., *Design Hurricane Storm Surge Pilot Study, FDOT Contract No. BD 545 #42* (2006), 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 four-meter increase in the predicted significant wave height.

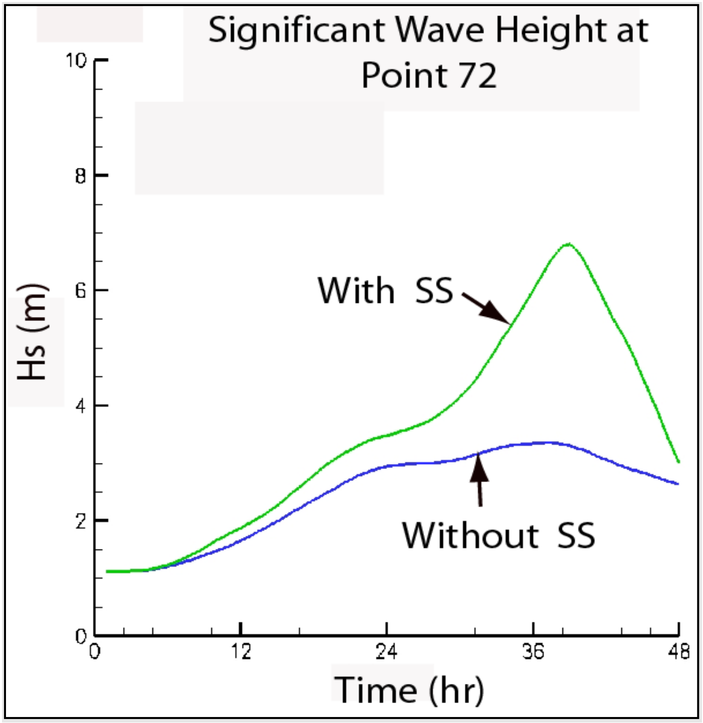


Figure 5.3-12: Wave Height Simulation during Hurricane Katrina with No Coupling (Without SS Curve) and with One-Way Coupling (With SS Curve) (Source: Sheppard et al., 2006)

With one-way coupling, input results (water elevations and currents) from the surge model into the wave model. This leads to more accurate prediction of the wave climate. With two-way coupling, transmit results from each model 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, two-way coupling provides the most accurate predictions.

5.3.4 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 developing boundary conditions.

5.3.4.1 Defining the Model Domain

The model domain is the spatial coverage of the model upstream of 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.

5.3.4.2 Roughness Selection

Specification of the roughness parameters for tidal analyses follows the same procedures as for riverine conditions (Section 5.2.4.2). 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 recommended 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 are helpful in targeting the proper roughness value.

5.3.4.3 Model Geometry

Model geometry refers to the spatial resolution incorporated into the model to describe the waterway bathymetry and overbank 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.

One-Dimensional Models
Specification of one-dimensional model geometry for tidal analyses follows the same recommendations as for riverine analyses (Section 5.2.4.3.1). In general, the only difference is the size of the model domain, which is discussed in Section 5.3.4.1.

Two-Dimensional Models
Specification of two-dimensional model geometry for tidal analyses follows the same recommendations as for riverine analyses (Section 5.2.4.3.2). 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.

5.3.4.4 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
- Specifying the same boundary conditions as above, with an additional wind boundary condition specified over the entire model domain
- Specifying 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.

Upstream Flow Boundary Conditions

Specification of upstream flow boundary conditions follows the same recommendations as those for riverine flow boundary conditions (Section 5.2.4.1), with some exceptions. In tidal analyses in Florida, inland boundaries typically are 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. In general, bridges over low-elevation, wide floodplains inland will experience more flow during a surge than bridges over high-elevation, narrow floodplains inland. This is because low-elevation, wide floodplains have substantial storage compared to high-elevation, narrow floodplains. The greater storage makes the floodplain less responsive to incoming flood flow of a storm surge, so the stage of low-elevation, wide floodplains rises more slowly than for floodplains with less storage. This creates a greater difference in water surface across the bridge, which increases the flow rate through the bridge.

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 5.3.2.

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 Department, 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 Department 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 shortcoming 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 5.3-13 presents the locations of the FDEP-developed elevations, as well as the locations of the interpolated elevations (in italics).

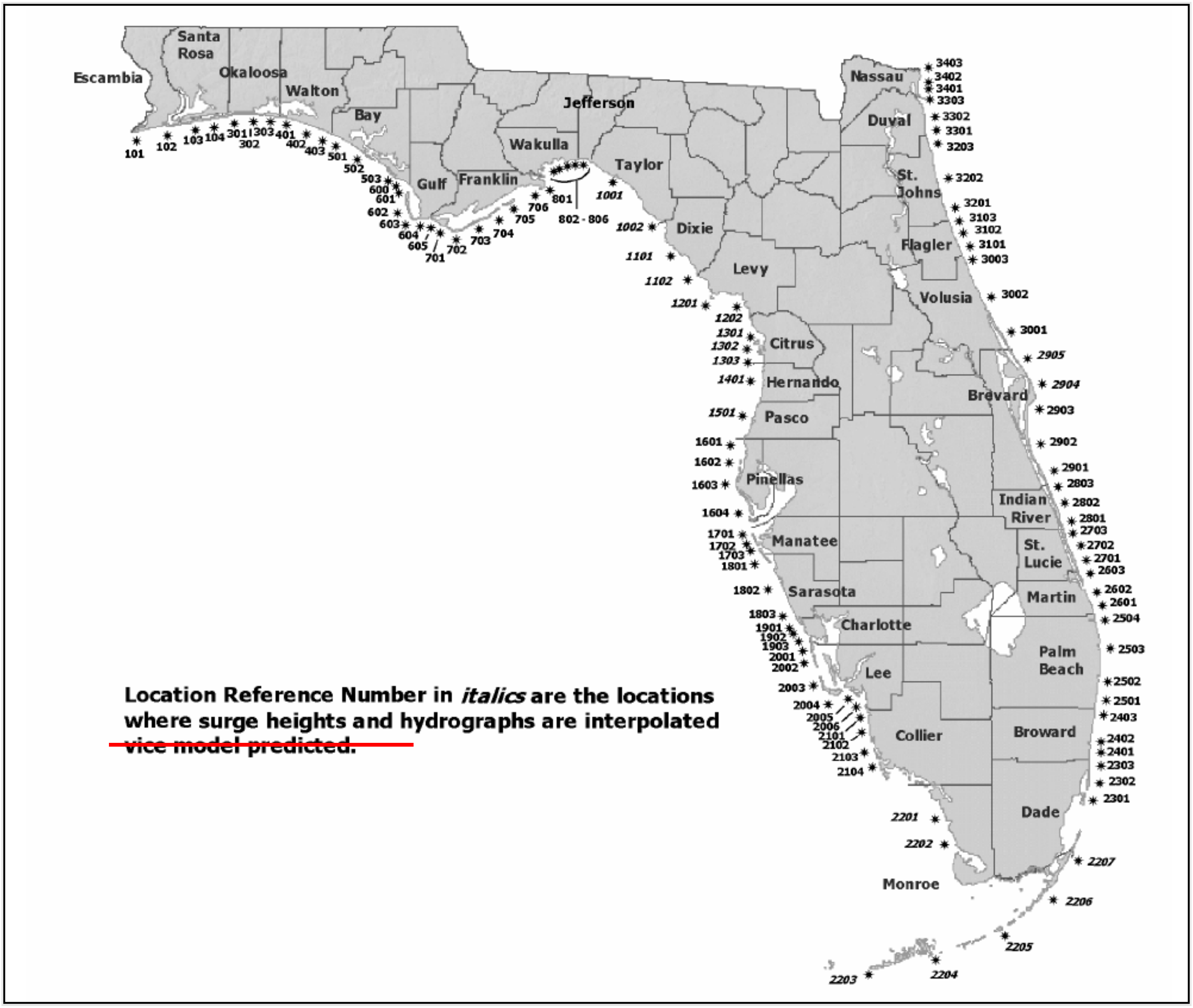


Figure 5.3-13: Storm Surge Peak Elevation and Hydrograph Locations

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

https://www.fdot.gov/roadway/Drainage/DHSH.shtm

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 5.3-14 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, Florida, 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.

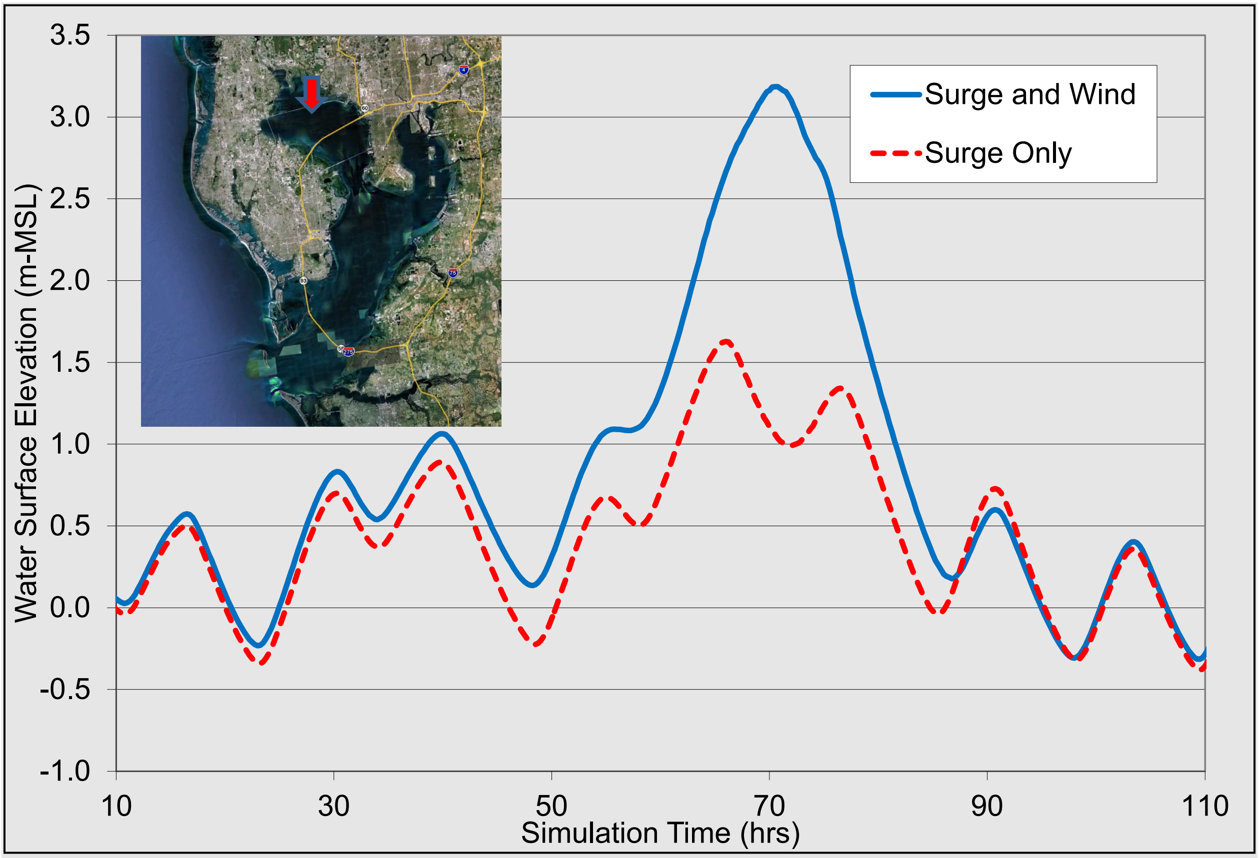


Figure 5.3-14: Surge Elevations at the Courtney Campbell Bridge Location during the 1852 Unnamed Hurricane both with and without Local Wind Effects

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, Florida. Figure 5.3-15 displays the calculated storm surge elevation time series at the Interstate 10 (I-10) Bridge over Escambia Bay (red line) and at the Pensacola Bay Bridge (blue line). Located near the back of 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.

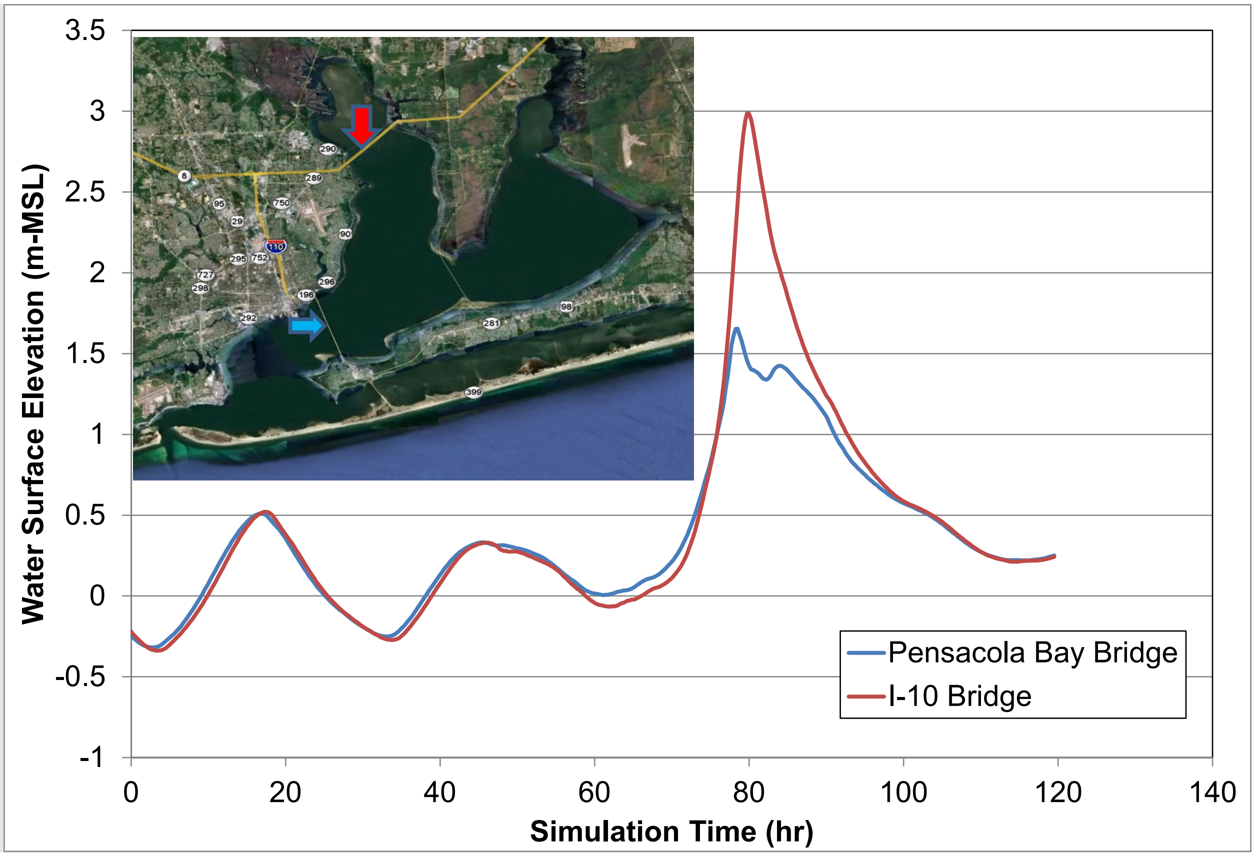


Figure 5.3-15: Hindcasted Surge Elevations at the I-10 over Escambia Bay Bridge and Pensacola Bay Bridge during Hurricane Ivan 2004.

Figure 5.3-15 shows that 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) that landed on or passed nearby the Florida coast from the NOAA hurricane database HURDAT. He divided the Florida coast into 15 districts (Figure 5.3-16), and developed expected extreme values for different return periods. Table 5.3-3 gives the expected maximum sustained (1-min average) wind speed for landfalling hurricanes calculated from Ochi's methodology.

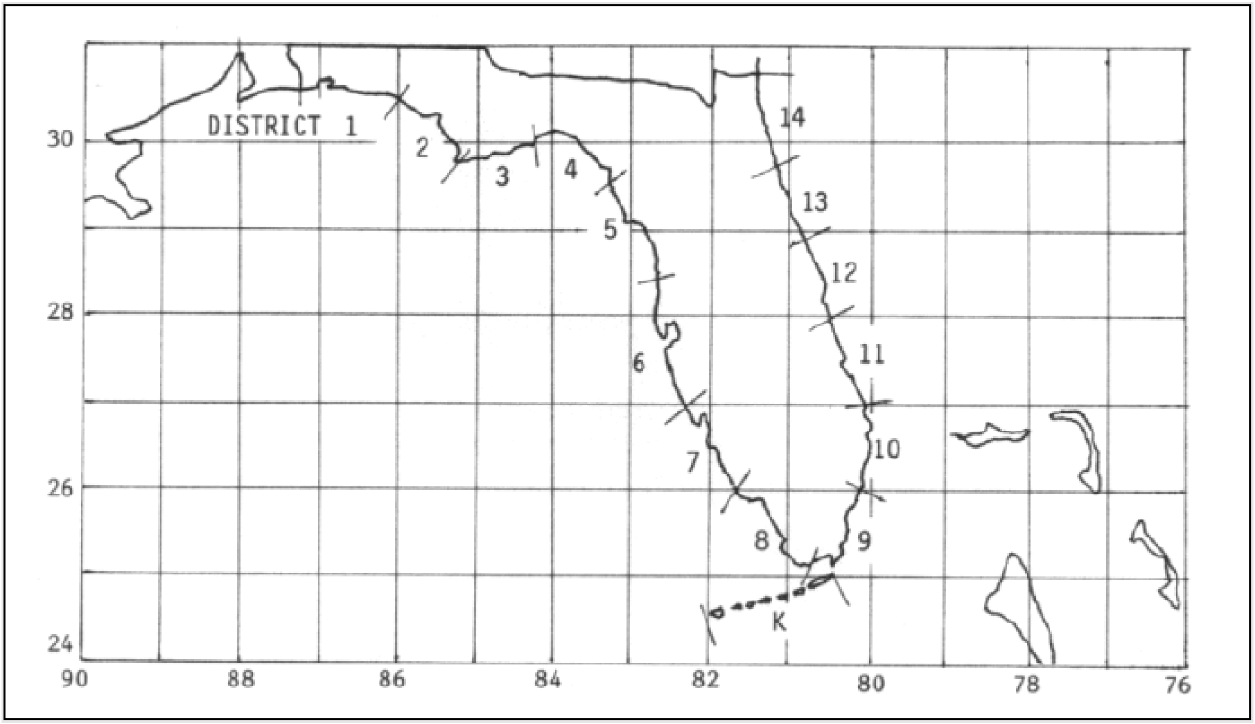


Figure 5.3-16: Locations of Coastline Division Employed in Wind Speed Analysis by Ochi (2004) (Source: Ochi, 2004)

Table 5.3-3: Example of Extreme Landfall Wind Speeds for Florida using 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.

Hurricane Hindcasts

Hurricane hindcasts simulate the wave and surge climate associated with a unique historical hurricane (Section 5.3.5). These types of simulations are performed primarily 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 (https://tidesandcurrents.noaa.gov/products) 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—including FEMA, NOAA, and USACE—have performed hindcasts of specific storms. These hindcasts are available sometimes upon request. Additionally, several commercially available sources of hindcast data also exist.

5.3.4.5 Bridge

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

Roughness
Roughness specification at bridge cross sections for tidal analyses follows the same recommendations as for riverine analysis (Section 5.2.4.2).

Bridge Routine
Selection of the appropriate bridge routines for tidal analyses follows the same recommendations as for riverine analysis (Section 5.2.4.5).

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 5.2.4.5). For two-dimensional modeling, typically, you would not model piers directly because their planform areas are significantly smaller than the areas of elements that resolve the bridge openings. However, there are several options 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 percent to 35 percent) 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 using a slip boundary condition along the model edges. Rather, you can attribute losses from the piers to the momentum losses associated with the creation of the secondary flows around the piers and in the wake region.

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

5.3.5 Simulations

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

5.3.5.1 Model Calibration

Before performing design simulations, you should calibrate the surge and wave model properly. Typically, you evaluate model performance through calibration and verification 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 wave models, calibration is achieved by comparing tidal simulations for a period of record to either measured data collected at specific locations or to widely available NOAA predictions at several locations. FEMA (2007) recommends that your calibration results for amplitude variation throughout the domain and phase variation be within 10 percent. In general, you typically do not perform flow rate or velocity calibration because of lack of reliable data. Flow calibration is more difficult to achieve than for water surface elevation data. However, if these data are 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 Table 5.3-4:

Table 5.3-4: 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 ideal. 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 also is difficult because calibration data are rarely available. If you can acquire the data, 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.

5.3.5.2 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 5.1.3 (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. Extract simulation results not only at the bridge cross section, but at locations upstream of the bridge piers (for local pier scour calculation). The length of the bridge dictates the number of locations. 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, you should extract results at a greater number of locations to resolve the variation. Extract flow rates and water depths upstream of the bridge constriction for contraction scour calculations.

Figure 5.3-17 displays an example of water surface elevation and velocity time series during the 100-year return period hurricane through Wiggins Pass near Naples, Florida. 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.

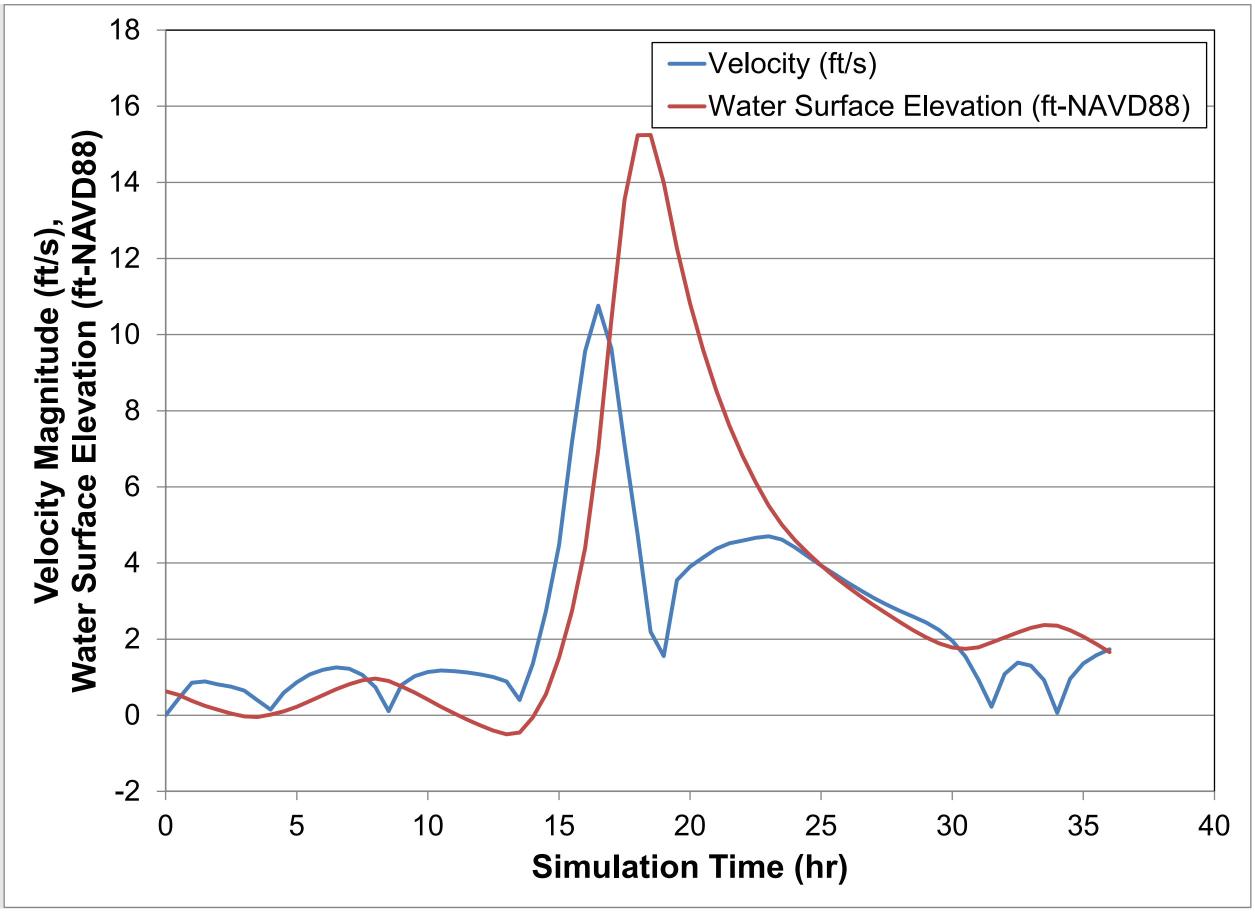


Figure 5.3-17: Example of Water Surface Elevation and Velocity Time Series during the 100-year Return Period Hurricane through Wiggins Pass near Naples, FL

5.3.5.3 Design Considerations

Typically, coastal bridges are not located in FEMA floodways and are not examined for their effects on backwater. As the designer, you would select the bridge location and profile for reasons related to right of way, environmental impacts, navigation, corrosion, etc., rather than for bridge hydraulics (backwater impacts). Review the recommendations contained in Section 5.2.5.3 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.

5.3.5.4 Wave Simulations

Wave parameters are necessary both for calculation of wave forces on bridge superstructures and for design of abutment protection. According to AASHTO (2008), calculate wave forces (discussed in Section 5.3.5.6) from 100-year return period wave conditions only. Similarly, design abutment protection 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 5.3-18).

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 $H_{max} = 1.80\ H_{significant}$.

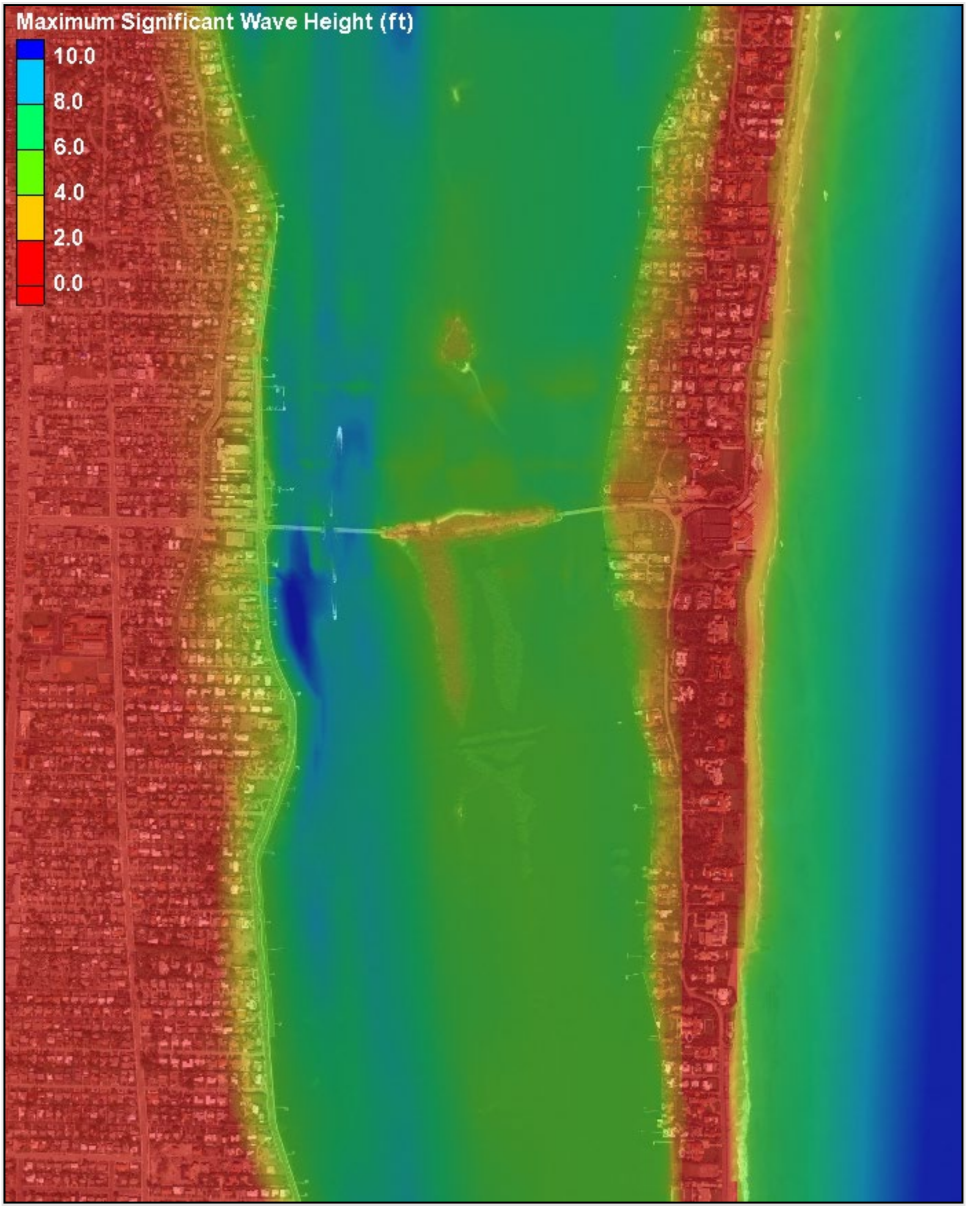


Figure 5.3-18: Example of Significant Wave Height Contours from Wave Modeling

5.3.6 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 5.3-19). Section 4.9.5 of the *Drainage Manual* and Section 2.5 of the *Structures Design Guidelines* address wave forces on bridge superstructures. The bulletin provides guidance on applying the specifications in the *AASHTO Guide Specifications for Bridges Vulnerable to Coastal Storms* to Department bridges. For bridges spanning waters subject to coastal storms, it states that 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/criticalness of the bridge when considering the consequences of bridge damage caused by wave forces. If you judge a bridge to be extremely critical, you would design it to resist wave forces. Bridges that you might judge to be non-critical do not require evaluation for wave forces.

Figure 5.3-19: Damage to the I-10 Bridge over Escambia Bay during Hurricane Ivan (2004)

Figure 5.3-20 defines the parameters involved in estimating wave forces and moments on bridge superstructures from the *AASHTO Specifications*. 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 Specifications* provide methods to determine both.

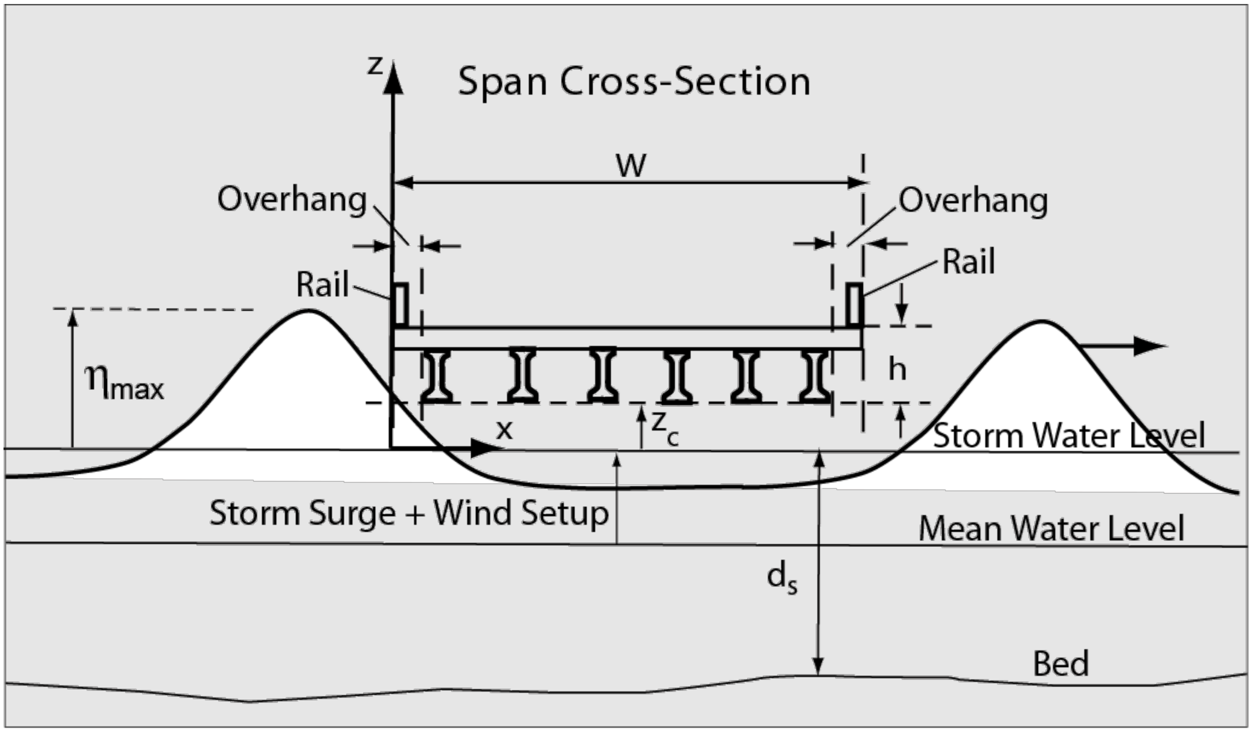


Figure 5.3-20: Definition Sketch for Wave Forces

The *AASHTO Specifications* provide a series of parametric equations for calculating the wave forces. There are two sets of equations—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.

5.4 MANMADE CONTROLLED CANALS

Manmade controlled canals have the following typical characteristics:

- They 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.
- They will not normally flood out of bank, even in a 100-year storm.
- They have low design velocities—typically 1 fps to 3 fps—and often are subject to aggradation requiring periodic dredging to maintain the needed cross section
- Their abutments typically do not 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 there are piles 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 typically is small.
- Usually, the canal owner can provide the hydraulic design discharge and stage.

Given the typically innocuous hydraulic and scour conditions at controlled canal bridges, you will find that the prudent level of effort required to perform the bridge hydraulics analysis is considerably less than for the typical bridge. In fact, you can abbreviate the traditional Bridge Hydraulics Report. Use the following outline for topics that should be included for controlled canals:

5.4.1 Introduction

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

5.4.2 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 frequency information is less than frequency requirements in the *Drainage Manual* for hydraulic or scour design, consult 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.

5.4.3 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.

5.4.4 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 percent of the waterway width
- Typically, pier scour on controlled canals is less than five feet; with no additional general or contraction scour, the CSU equations may be used

5.4.5 Abutment Protection

- Refer to Minimum Abutment Protection in Section 4.9.1 of the *Drainage Manual*
- Boat wakes and wave impact may dictate more robust abutment protection than would be needed to protect for the flood flow velocities; consider this and document as needed
- Owner may have specific requirements for abutment protection

5.4.6 Bridge Deck Drainage

Refer to Section 3.9 of the *Drainage Manual*, and Appendix H and Section 5.6 of this document.

5.4.7 Appendix

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

5.5 BRIDGE SCOUR

Lowering 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-predicting design scour depths can result in costly bridge failures and possibly in the loss of lives; while over-predicting can result in significant construction 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 Section 4.9.2 of the *Drainage Manual*.

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

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 5.5.2 for further discussion

Include the components discussed in the following sections in your scour estimates.

5.5.1 Scour Components

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

1. Long-term channel processes (channel migration and aggradation/degradation)
2. Contraction scour
3. Local scour

5.5.1.1 Long-Term Channel Processes

Scour associated with long-term channel processes is the change in bed elevation associated with naturally occurring or manmade 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).

Changes upstream and downstream affect stability at the bridge crossing. Natural and manmade disturbances may change 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, second, because bank migration, thalweg shifting, and degradation may cause foundation undermining regardless of whether the bridge experiences the design event.

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).

There are techniques to address channel migration 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.

Chapter 6 of HEC 20 (Legasse et al., 2001) provides procedures for predicting and evaluating lateral channel migration through aerial photograph analysis. See Section 5.2.1.3 for sources of aerial photographs.

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 are not—particularly along the southwest coast. 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)

A coastal engineer should perform the analysis of coastal hydraulics for the design and evaluation of bridges over tidal inlets. 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).

Aggradation/Degradation
Aggradation and degradation relate 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 in the watershed on stability at a bridge location, you must not only evaluate the current stability of the stream and watershed, but also the 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 you perform the necessary data collection (including contacting local agencies) to become aware of future 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*.

For existing bridge locations, 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 also can 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 photography is another method of channel stability analysis.

Estimate long-term vertical stability trends 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, add the estimate at the end of the project life to the total scour. If the result is aggradation, then document the estimate in the BHR. However, do not include this estimate in the estimate of total scour. Rather, the 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. Examining 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. A qualified coastal engineer should perform these analyses. 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.

5.5.1.2 Contraction Scour

Contraction scour occurs when a channel's cross section is reduced by natural or manmade 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, the encroachment of abutments, and the presence of headlands (examples in Figure 5.5-1 and Figure 5.5-2). 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 is 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, refer to HEC 18.

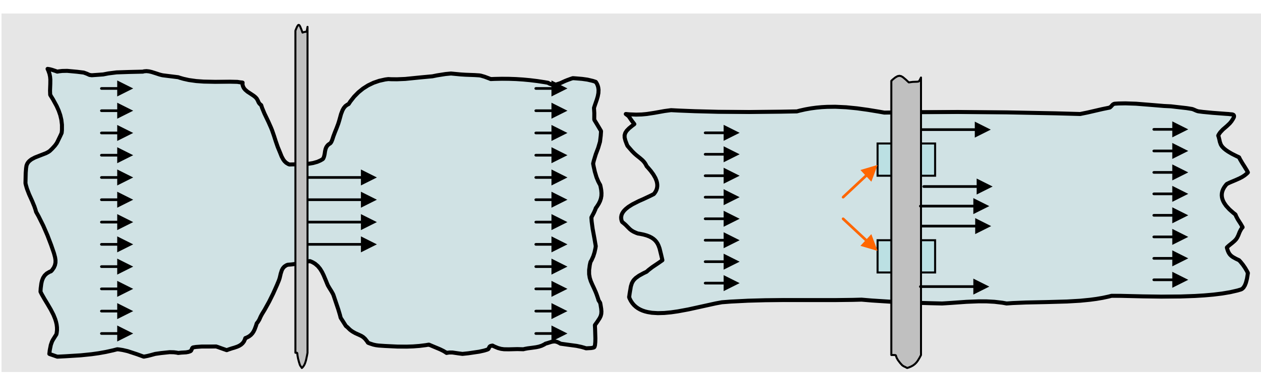


Figure 5.5-1: Examples of Contractions at Bridge Crossings

Figure 5.5-2: Example of Manmade Causeway Islands Creating a Channel Contraction

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), you may pursue alternative analyses, including sediment transport modeling. 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 below. Refer to HEC 18 for more information.

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:

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

$y_s = y_2 - y_o$ = average contraction scour

where:

y_1 = Average depth in the upstream channel, ft
y_2 = Average depth in the contracted section after scour, ft
y_0 = Average depth in the contracted section before scour, ft
Q_1 = Discharge in the upstream channel transporting sediment, ft^3/sec
Q_2 = Discharge in the contracted channel, ft^3/sec
W_1 = Bottom width of the main upstream channel that is transporting bed material, ft
W_2 = Bottom width of the main channel in the contracted section less pier widths, ft
K_1 = Exponent listed in Table 5.5-1

Table 5.5-1: 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/sec
- ω = Fall velocity of bed material based on the D_{50}, ft/sec (Figure 5.5-3)
- g = Acceleration of gravity, 32.17 ft/sec^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 (1,000 kg/m^3)

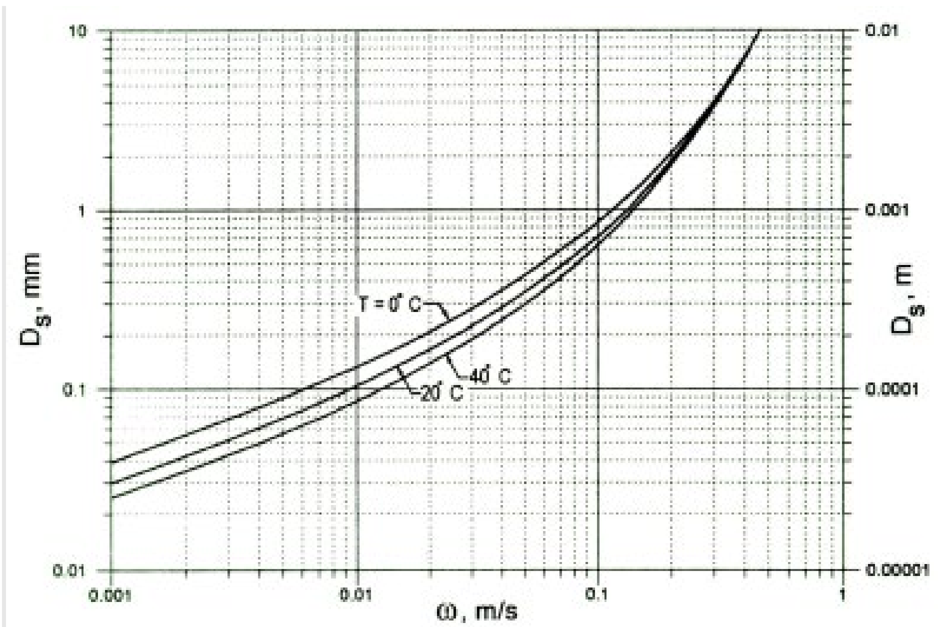


Figure 5.5-3: Fall Velocity of Sediment Particles with Diameter Ds and Specific Gravity of 2.65 (Source: HEC 18, 2001)

HEC 18 provides guidance for selecting upstream cross section locations, 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 sections to the bridge.

As stated previously, application of this methodology may result in overly conservative estimates. See the subsection "Unsteady, Complex Flows" in this section for an alternative methodology for calculating contraction scour.

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:

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

$y_s = y_2 - y_o$ = average contraction scour

where:

y_2 = Average equilibrium depth in the contracted section after contraction scour, ft

Q = Discharge through the bridge or on the set-back overbank area at the bridge associated with the width W, ft^3/sec

D_m = Diameter of the smallest non-transportable particle in the bed material (1.25 D_{50}) in the contracted section, ft

D_{50} = Median diameter of bed material, ft

W = Bottom width of the contracted section less pier widths, ft

y_o = Average existing depth in the contracted section, ft

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.

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.

5.5.1.3 Local Scour (Pier and Abutment)

You can divide local scour into pier and abutment scour. 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) 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.

You can divide local scour 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, 2005) rather than the CSU Pier Scour Equation when the total scour (long-term channel conditions, contraction scour, and pier scour) is greater than five feet. The Florida Complex Pier Scour Procedure is described in HEC 18, Fifth Edition. The Florida Complex Pier Scour Calculator and Procedure can be downloaded at:

http://www.fdot.gov/roadway/Drainage/Bridge-Scour-Policy-Guidance.shtm .

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

<u>Sheppard's Pier Scour Equations</u>

Sheppard's Pier Scour Equations target three dimensionless hydraulic and sediment transport parameter groups to predict scour at simple piers. You can apply the equation to both riverine and tidal flows and to sediment sizes typical within the continental U.S. The equations give good results for both narrow and wide piers. The FDOT *Bridge Scour Manual* includes a detailed discussion. The pier scour equations are summarized below:

In the clear-water scour range:

$$\left(0.4 \le \frac{V}{V_c} \le 1.0\right)$$

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

In the live-bed scour range:

$$\left(1.0 < \frac{V}{V_c} \leq \frac{V_{lp}}{V_c}\right)$$

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

and in the live-bed scour range above five feet:

$$\left(\frac{V}{V_c} > \frac{V_{lp}}{V_c}\right)$$

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

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

D^* = Effective diameter of the pier, ft

y_o = Water depth adjusted for general scour, aggradation/degradation, and contraction scour, ft

V = Mean depth-averaged velocity, ft/sec

V_c = Critical depth-averaged velocity, ft/sec

V_{lp} = Depth-averaged velocity at the live-bed peak scour depth, ft/sec

D_{50} = Median sediment diameter, ft

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 5.5-4.

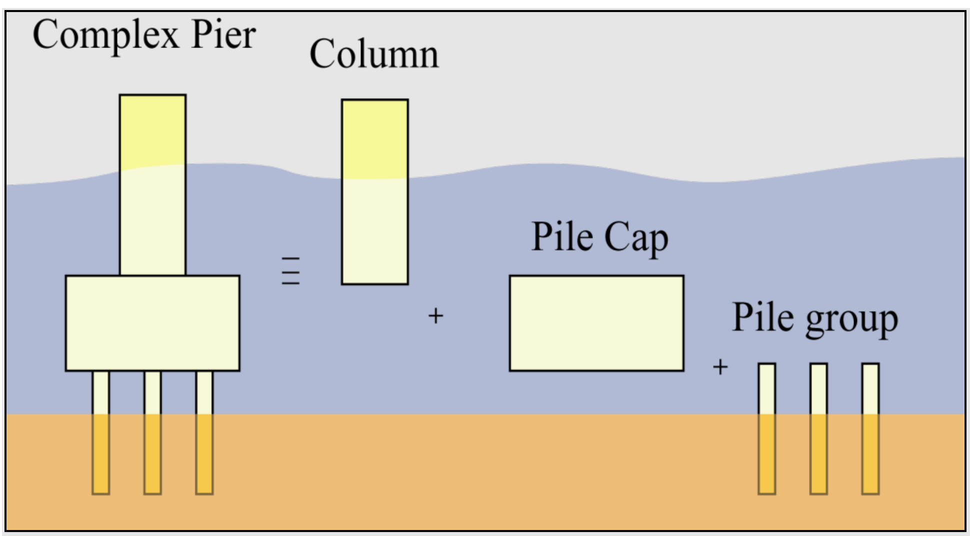


Figure 5.5-4: Complex Pier Components

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 to determine 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 Figure 5.5-5 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

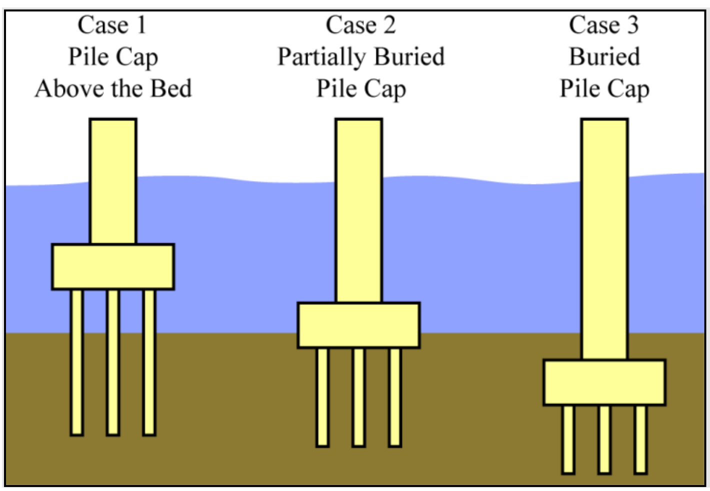


Figure 5.5-5: Three Cases of Local Scour Depth for Complex Pier Computations

Refer to the FDOT *Bridge Scour Manual* for a more detailed discussion on the procedure and the application of the equations.

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

5.5.1.4 Scour Considerations for Waves

Waves are an important factor that you must address when designing bridges exposed to long fetches. This is particularly true at bridge abutments and approach roadways. Figure 5.5-6 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 after 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 5.5.4.

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 5.5.4.

Figure 5.5-6: East Approach to the I-10 WB Bridge over Escambia Bay After 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.

5.5.2 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. The previous information 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 piers, the normal, everyday scour elevation should be reflected in the inspection reports and the design survey (Figures 5.5-7 and 5.5-8). 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. Investigate this and address it on a case-by-case basis.

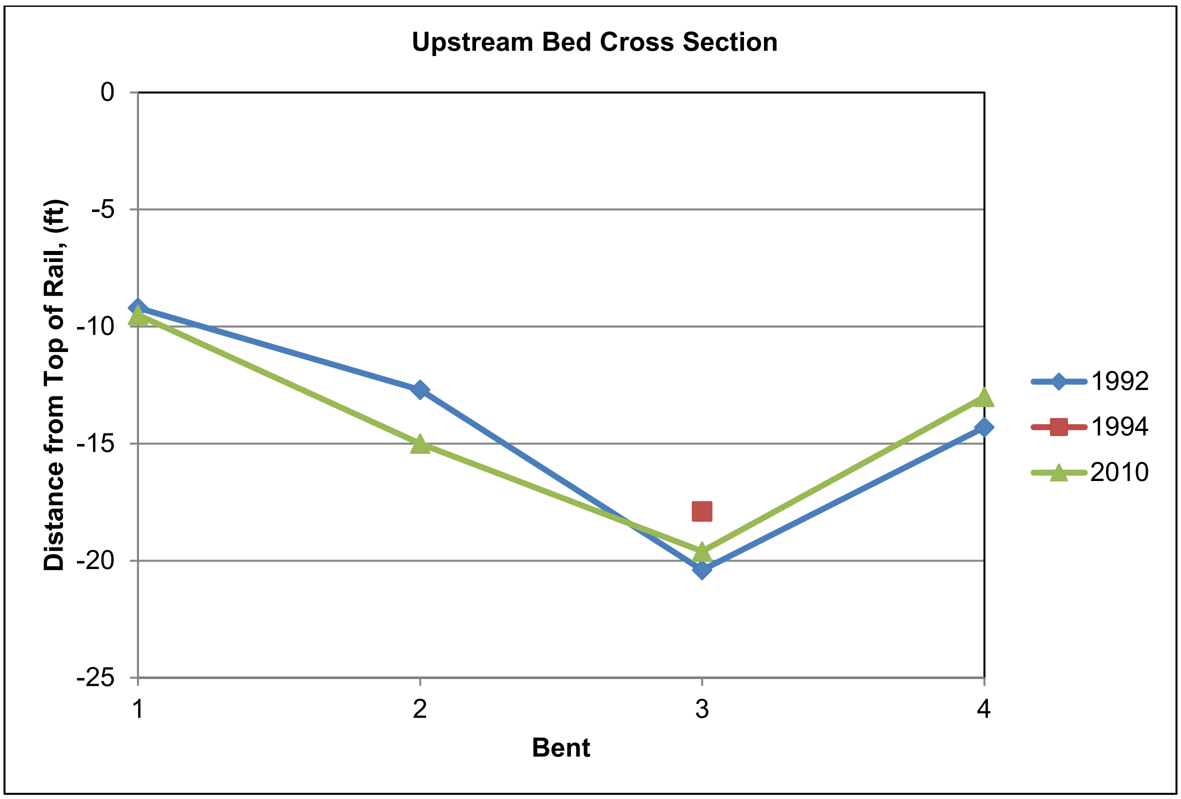


Figure 5.5-7: Example of Normal, Everyday Scour Holes from Bridge Inspection Data

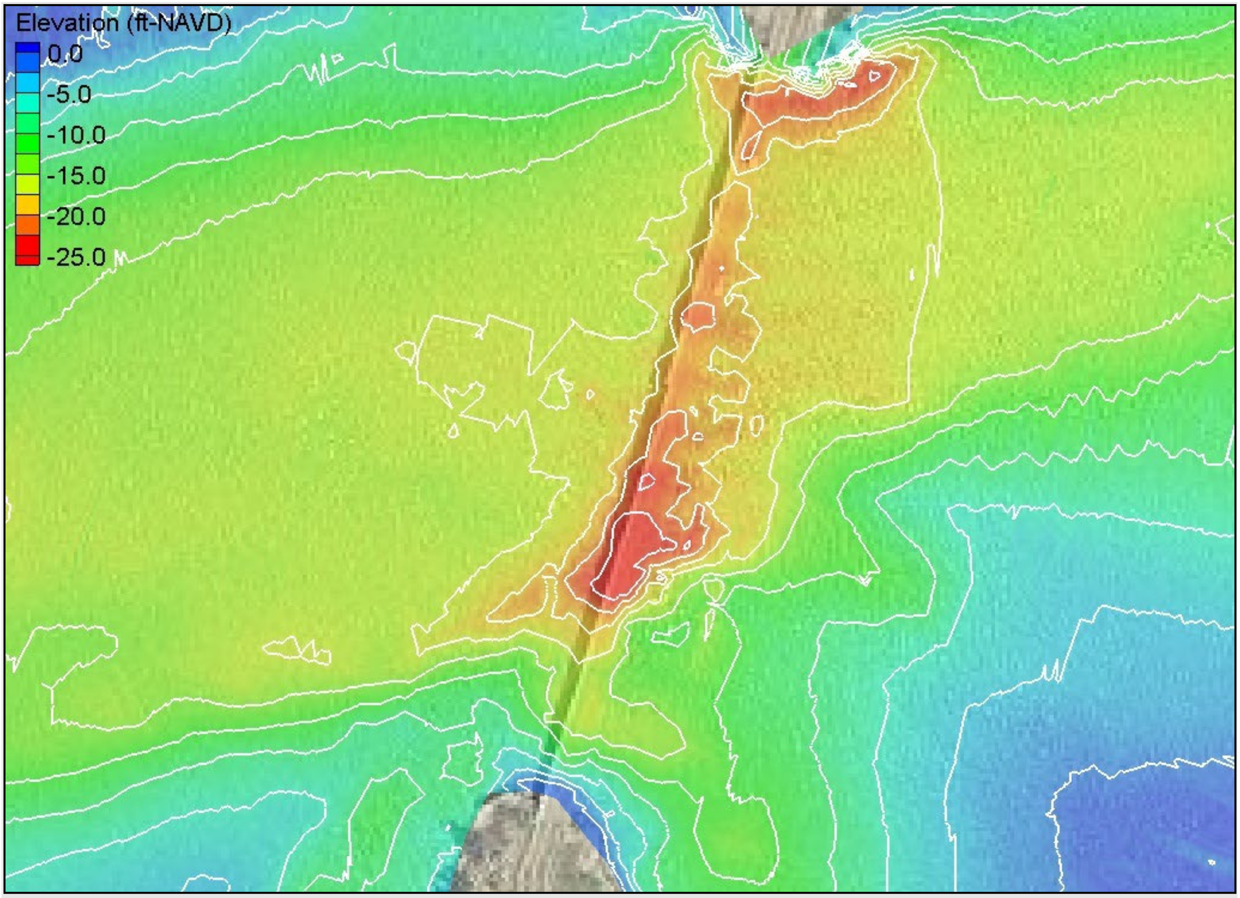


Figure 5.5-8: Example of Normal, Everyday Scour Holes from Survey Data

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, base estimates of the normal everyday scour 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.

5.5.3 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 for cohesive bed materials considered “scourable” according to FHWA guidelines. Refer to HEC 18, Fifth Edition, Chapter 4 for an explanation of rock characteristics that relate to strength and scour potential. Consult the District Drainage Engineer and the District Geotechnical Engineer before initiating the Rock/Clay Scour Procedure.

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 5.5-9 and 5.5-10, 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.

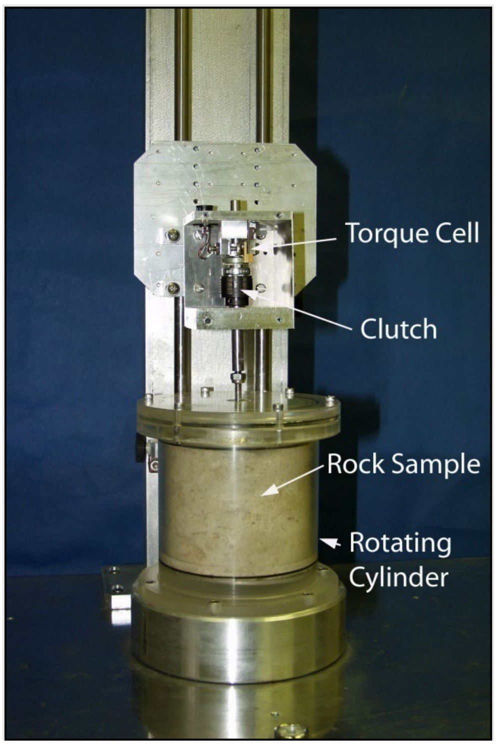


Figure 5.5-9: Rotating Erosion Test Apparatus (RETA, above)

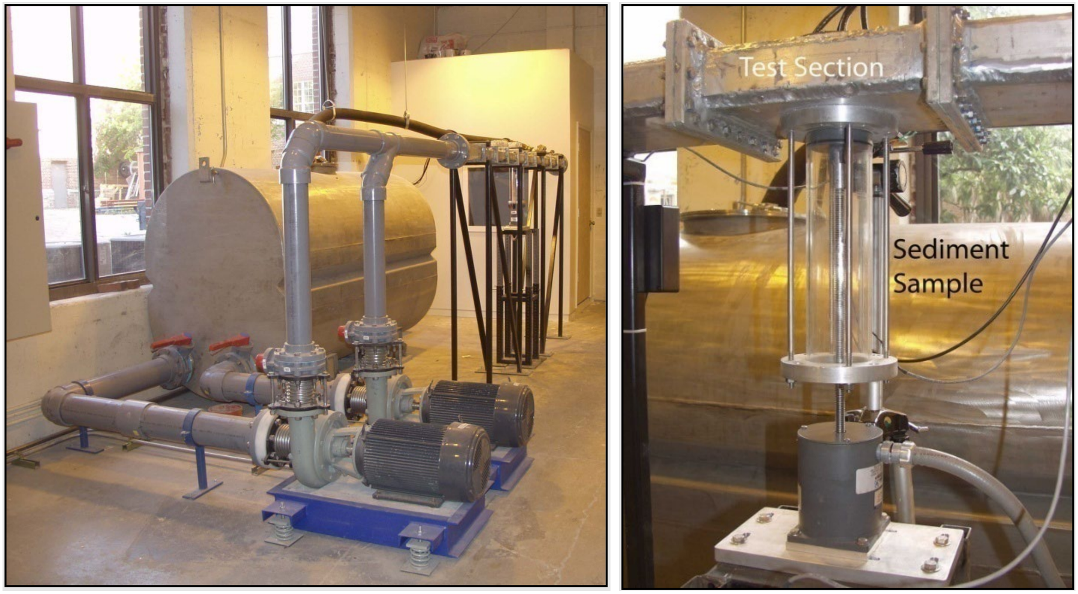


Figure 5.5-10: Sediment Erosion Recirculating Flume (SERF)

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.fdot.gov/roadway/Drainage/Fla-Rockclay-Proc.shtm

5.5.3.1 Pressure Scour

See HEC 18 for detailed information on pressure scour.

5.5.3.2 Debris Scour

See HEC 18 for detailed information on debris scour.

5.5.4 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.

5.5.4.1 Abutment Protection

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

Spill-Through Abutments
Where flow velocities do not exceed 7.7 fps, and/or wave heights do not exceed 2.4 feet, minimum protection consists of one of the following protection methods 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)—Articulating concrete block also is defined in Section 530
- Fabric formed revetment, also called grout-filled mattress (articulating with cabling throughout the fabric forms)

Create site-specific designs when using articulated concrete block or fabric formed revetment abutment protection. As of June 2020, the Department has Developmental Specification 531 for Fabric Formed Revetment Systems. The FDOT *Structures Detailing Manual* provides typical details for standard revetment protection of abutments and extent of coverage. Determine the horizontal limits of protection using HEC 23. Provide a minimum distance of 10 feet 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 5.5-11). 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 5.5-11: Damage to Sand-Cement Grouted Riprap Abutment Protection

Determine the horizontal and vertical extents, regardless of protection type, using the design guidelines contained in HEC 23. If the results from the HEC 23 calculations show that a horizontal extent less than 10 feet is acceptable, you should still provide a minimum of 10 feet. 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 way limits, do the following:

a. Recommend additional right of way.
b. Provide an apron at the toe of the abutment slope that extends an equal distance out around the entire length of the abutment toe. In doing so, consider specifying a greater rubble riprap thickness to account for reduced horizontal extent (Figure 5.5-12).

Make additional considerations regarding extents 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 5.5-6) as well as the approach roadways. Consider extending the limits of protection to include the approach spans in wave-vulnerable areas.

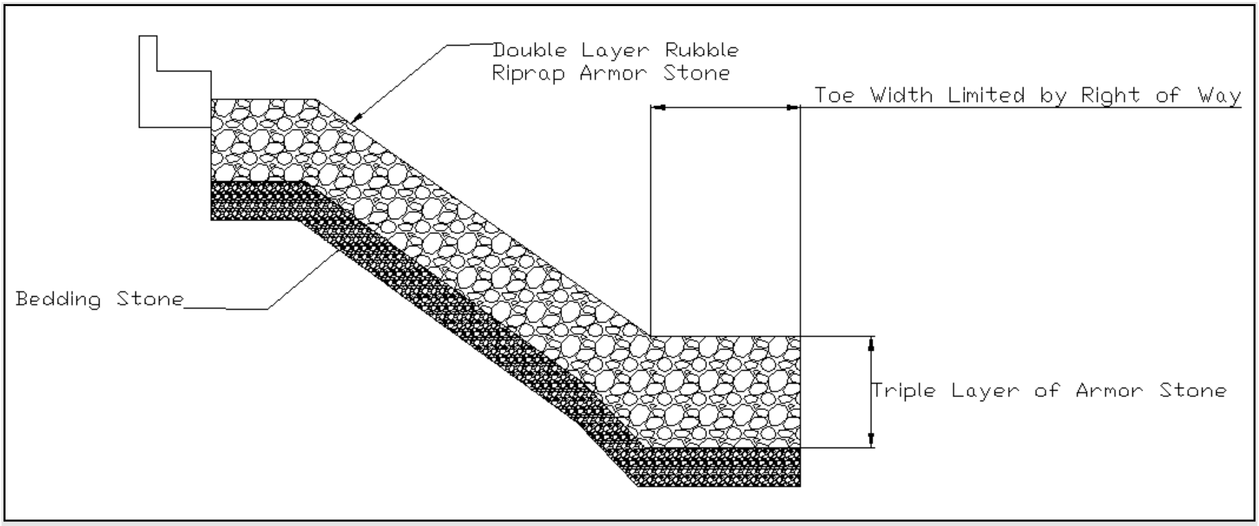


Figure 5.5-12: Example of Increased Toe Thickness to Offset Decrease in Toe Width

When bridges are to be widened, you may not be able to simply recommend using 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, you can do the following:

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

Bulkhead/Vertical Wall Abutments
You must protect abutments by sheet piling with rubble toe protection below the bulkhead, and with revetment protection above the bulkhead when appropriate. Design the size and extent of the protection for the individual site conditions.

Allow abutment protection to extend beyond the bridge along embankments that may be vulnerable during a hurricane surge. You need to consider wave attack above the peak design surge elevation and wave-induced toe scour at the foot of bulkheads. In such cases, consult a qualified coastal engineer 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. Avoid using steel cabling 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 often will grow through the rocks, adding aesthetic 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. For an illustration of bridge abutment slope protection adjacent to streams, refer to the FDOT *Structures Detailing Manual* at the following link:

https://www.fdot.gov/structures/structuresmanual/currentrelease/structuresmanual.shtm

Where velocities do not exceed 7.7 fps and waves do not exceed 2.4 feet on a 1V:2H slope, protection should consist of a 2.5-foot-thick armor layer comprised of FDOT Standard Bank and Shore Rubble Riprap over a one-foot thick layer of bedding stone over filter fabric. Size the filter fabric 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 to prevent the armor stone from damaging the filter fabric during construction; and to inhibit movement during design events. Ensure the riprap has a well-graded distribution to promote interlocking between the individual units, which improves performance of the protection. For riverine applications, 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 often was a point of failure. Shifting of the stones during a minor event would cause a gap to open at the top of the slope, allowing erosion to take place. 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 2.4 feet, you 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, use the more conservative (larger stone size) design. For these designs, develop a modified special provision 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. 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 USACE (1995) reference presents guidelines for design of granular filter layers as a function of the armor stone size.

For toe scour protection, the USACE (1995) reference provides guidance on sizing stones and designing the apron width. Toe apron width will depend on both geotechnical and hydraulic factors. For a sheet-pile wall, you must protect the passive earth pressure zone. 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 percent of the depth at the structure. Compare this apron width to that required by geotechnical factors and adjust it appropriately. Regarding size 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 you don’t expect significant wave attack, include all options that are appropriate based on site conditions (e.g., fabric-formed concrete, standard rubble, cabled interlocking block, etc.; see Figure 5.5-13 through Figure 5.5-15). HEC 23 provides guidance for design of these protection systems, as follows:

- Design Guideline 8—Articulating Concrete Block Systems
- Design Guideline 9—Grout-Filled Mattresses (Fabric Formed Revetment Systems)
- Design Guideline 14 – Rock Riprap at Bridge Abutments

Figure 5.5-13: Example of Rubble Riprap Abutment Protection

Figure 5.5-14: Example of Articulating Concrete Block Abutment Protection

Figure 5.5-15: Example of Fabric Formed Revetment Abutment Protection

Document options shown to be appropriate for the site in the BHR. You may write a technical specification based on the use of the most ideal revetment material, with the option to substitute the other allowable materials at no additional expense to the Department. This recommendation would help to eliminate revetment CSIPs (Cost Savings Initiative Proposals) during construction. No matter what options are allowed, match the bedding (filter fabric and bedding stone) 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. Address the design of these structures 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. Design areas between the seawall and the abutment using the same procedures as spill-through abutments. These designs should ensure encapsulation of the fill behind the seawall (Figure 5.5-16) to prevent loss of fill and potential failure of the anchoring system (Figure 5.5-17).

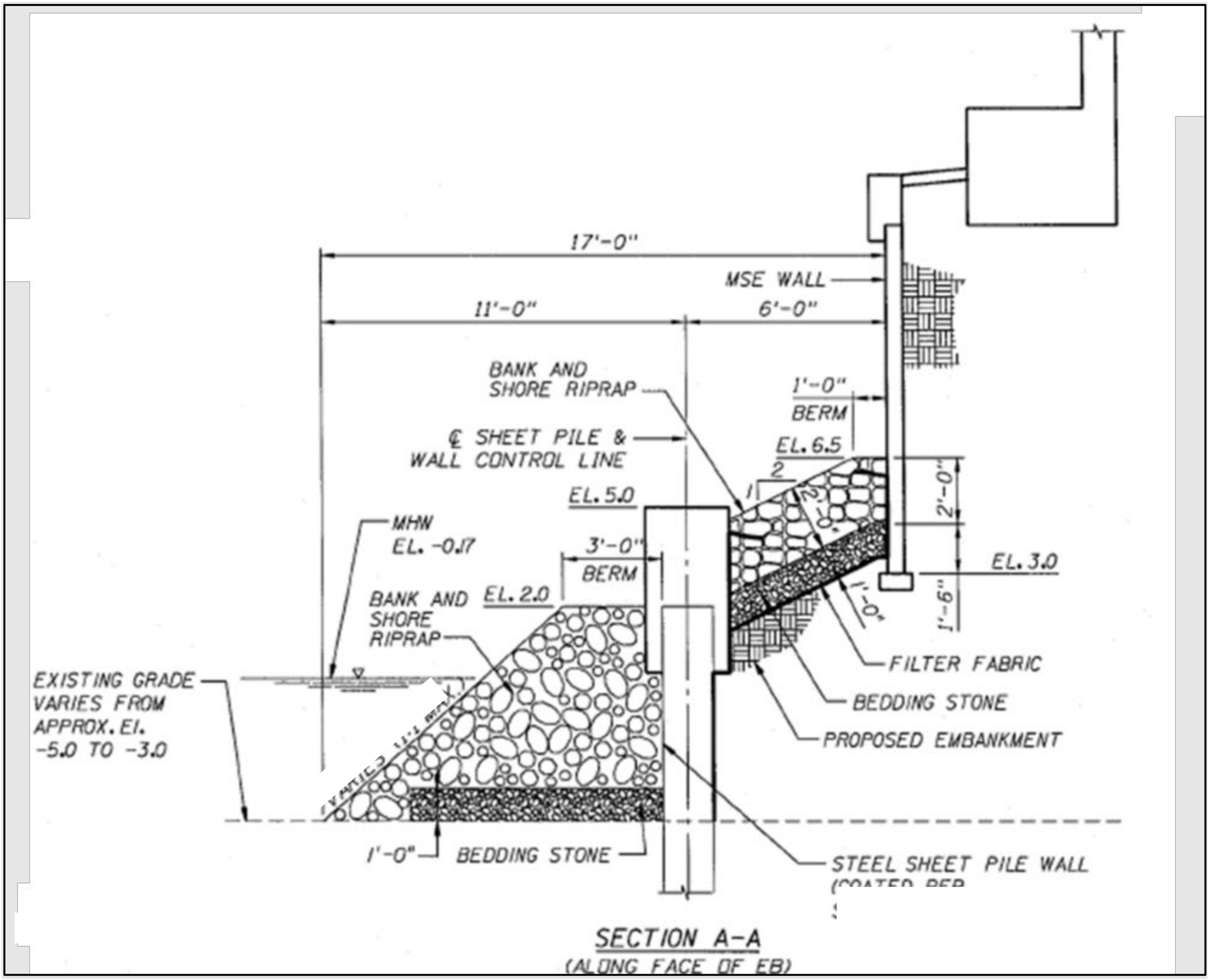


Figure 5.5-16: Example of Abutment Protection Design Including a Seawall

Figure 5.5-17: Seawall Failure Following Hurricane Frances (2004)

5.5.4.2 Scour Protection at Existing Piers

For bridges evaluated as scour critical and where monitoring is not an option, and the upstream design flow velocities do not exceed 7.2 fps for rectangular piles or bascule piers and 8.2 fps for round piling or drilled shafts, one of the countermeasures you should consider 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, fabric formed revetments, gabion/marine mattresses, 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 (Fabric Formed Revetment Systems)
- 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 safety factors
- Methodologies for sizing the material
- Recommendations for designing coverage extents, filter requirements, and installation guidelines

You will see several similarities between the procedures. All guidelines recommend ensuring that the top of the protection remain level with the bed of the approach. Suggestions for achieving this include placing 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 use of 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 articulating concrete block and gabion protection systems. You should review disadvantages and advantages of each system, including construction feasibility and cost.

5.6 DECK DRAINAGE

To drain the deck of a bridge, there are three options, in order of preference:

1. Rely on the longitudinal grade of the bridge to convey the deck runoff to the end of the bridge.
2. Use freely discharging scuppers or inlets to drain the deck runoff to the area directly below the bridge. These sometimes are 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 sometimes are 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. An evaluation of the bridge calculated spread during Maintenance of Traffic phases could affect the selection of options and must be included in the analysis for selection of deck drainage schemes.

5.6.1 Bridge End Drainage

If the profile grade of the roadway is sloping off of the bridge, roadway inlets collect runoff from the bridge, often immediately beyond the bridge approach slab. Inlets typically are 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 that you place an inlet 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 (See Standard Plans, Index 425-040, Gutter Inlet Type S). This inlet, including its 5-foot-long gutter transition, is usually located about 35 feet from the end of the approach slab to provide space for the guardrail's Approach Transition Connection to Rigid Barrier, including its curb transition to shoulder gutter (See Standard Plans, Index 536-001, Guardrail). Additionally, check the spread at the shoulder gutter inlet for the 10-year flow to ensure that runoff does not overtop the shoulder, causing erosion of the embankment (refer to Chapter 6 and Appendix H 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 5.6-1). For standard cross slopes of 0.02 ft/ft for bridge shoulders and 0.06 ft/ft for roadway shoulders, with a 10-foot wide shoulder, the longitudinal slope of the gutter due to the transition is 2.1 percent. For this situation, the roadway grade would need to be greater than 2.1 percent for roadway runoff to flow onto the bridge. Appendix H shows how this slope was determined, and the same method can be used to calculate the slope for other situations.

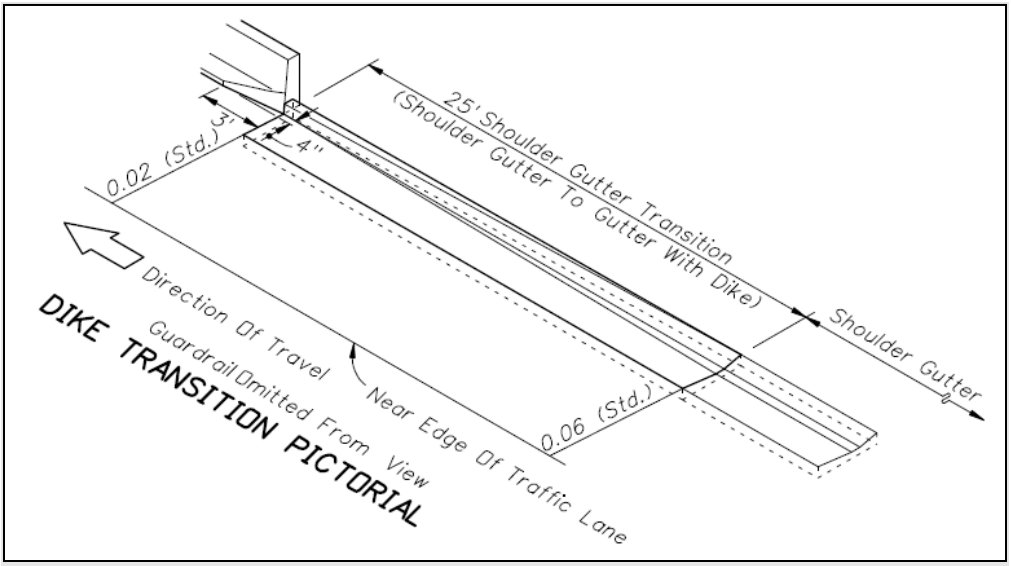


Figure 5.6-1: Shoulder Gutter Transition at Bridge End

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 toward the roadway immediately downstream of the bridge. The flow then can be picked up in the first curb inlet or barrier wall inlet off of the bridge.

5.6.2 No Scuppers or Inlets (Option 1)

If possible, allow stormwater to flow to the end of the bridge and collect in the roadway drainage system. To determine if this option is feasible, check the spread:

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

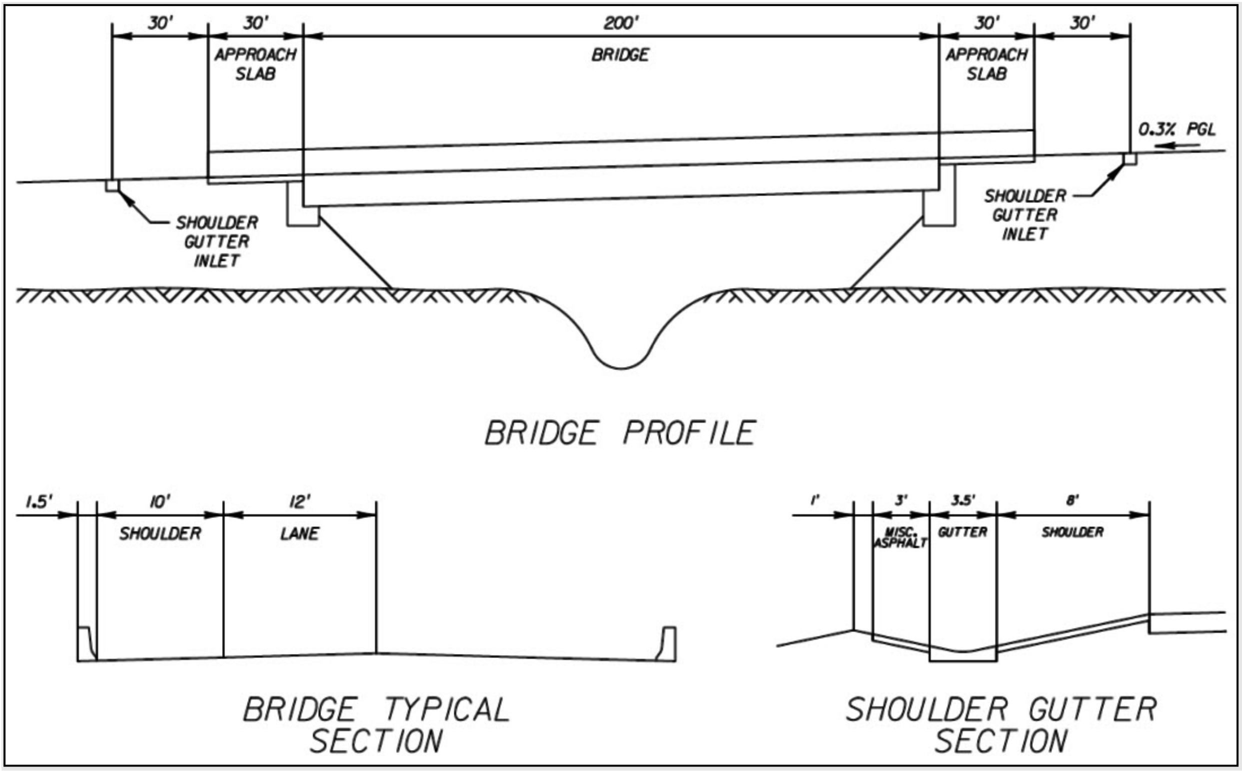
Calculate spread based on the Gutter Flow Equation in Section 6.3.2 of this document. Spread criteria are given in Chapter 3.9 of the *Drainage Manual.* If the spread exceeds the allowable spread criteria, then use scuppers or inlets on the bridge to reduce the spread. If the spread exceeds the criteria, consider adjusting the profile grade to reduce the spread before adding scuppers or inlets on the bridge. Reduce spread 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 5.6-1
A bridge for a two-lane rural roadway has the following characteristics:

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



Solution:

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 Chapter 6 for explanation)
A = Drainage area, acres

Solving the gutter flow equation for spread:

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

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

Also check the spread at the shoulder gutter inlet on the downhill side of the bridge. 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

Assume that the location of the bridge and the NOAA Atlas 14 Point Precipitation Frequency Estimates webpage are used to provide the 10-year, 10 minute rainfall intensity of 7.0 inches per hour. The flow to the inlet is:

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

Note that this value is slightly conservative. The one-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 Section 6.3.2.3 of this document 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 acceptable.

5.6.3 Scuppers (Option 2)

Scuppers typically are formed by tying PVC pipe into place prior to pouring the concrete for the bridge deck (Figure 5.6-2). The deck runoff will flow into the scuppers, through the deck, and then freefall to the ground or water surface below the bridge.

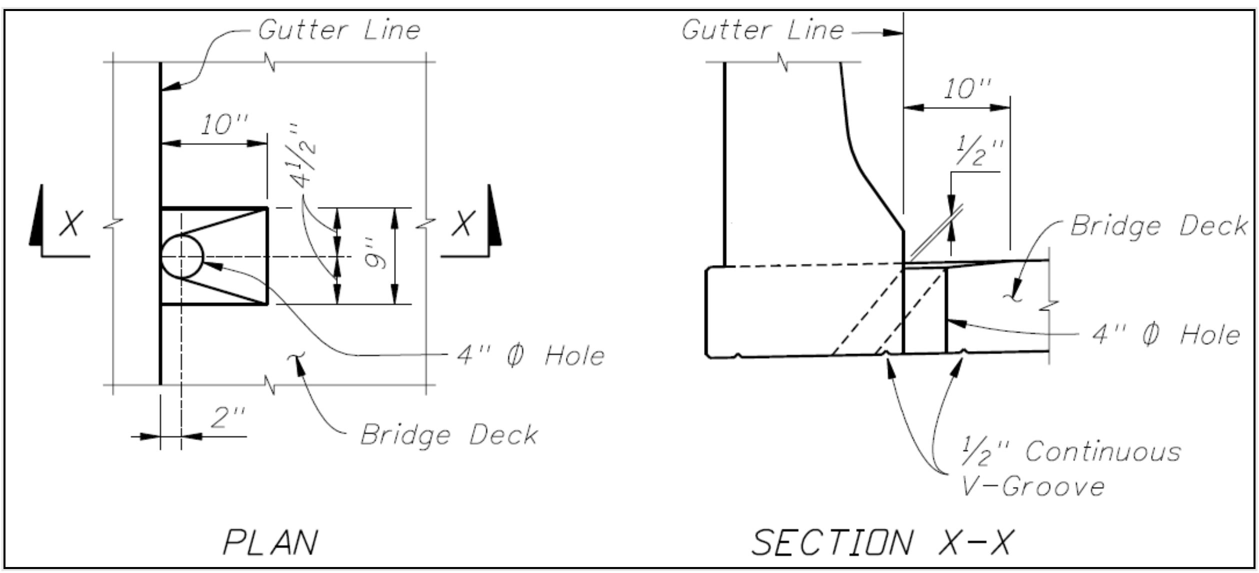


Figure 5.6-2: Standard FDOT Scupper Detail

Avoid placing scuppers over certain areas due to the direct discharge. These areas include:

- Driving lanes, railroad tracks, and sidewalks
- Major navigation channels
- Bridge bents
- Erodible soil, unless the free discharge is at least 25 feet above the soil
- Environmentally sensitive water bodies as negotiated with permitting agencies
- 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 four inches in diameter and spaced on 10-foot centers, unless spread calculations indicate closer spacing is required. Typically, the 10-foot spacing will provide adequate drainage for most bridges. You can evaluate the intercepted flow for four-inch bridge scuppers on a grade using the capacity curves in Figure 5.6-3 and Figure 5.6-4. The curves were derived from laboratory studies performed at the University of South Florida (Anderson, 1973).

Grated scuppers or inlets, as shown in Figure 5.6-5, are more uncommon, especially as free-draining scuppers. Although grated inlets can be used with open systems, they are normally used with closed systems. You might use this type of grated scupper, or perhaps one with a smaller grate, to drain a bridge sidewalk or if you expect significant bicycle or pedestrian traffic on the shoulder. The four-inch ungrated scuppers will not meet ADA requirements.

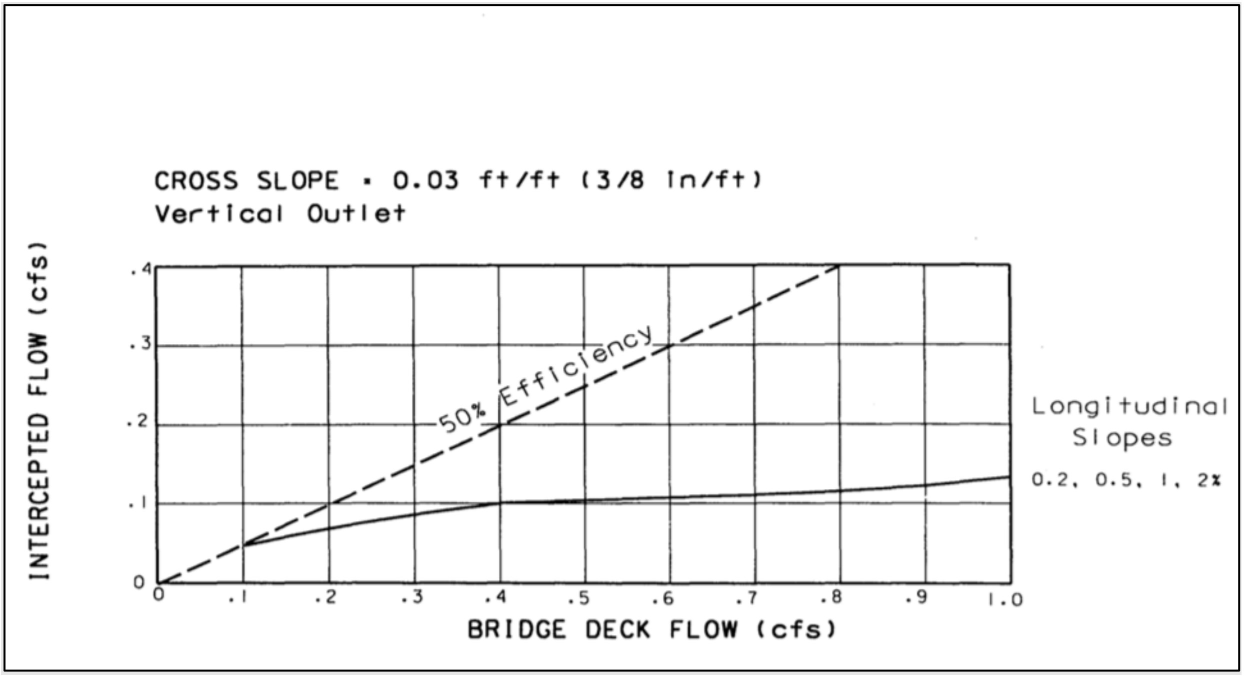


Figure 5.6-3: Intercepted Flow for 4-inch Bridge Scuppers
Cross Slope = 0.03 ft/ft

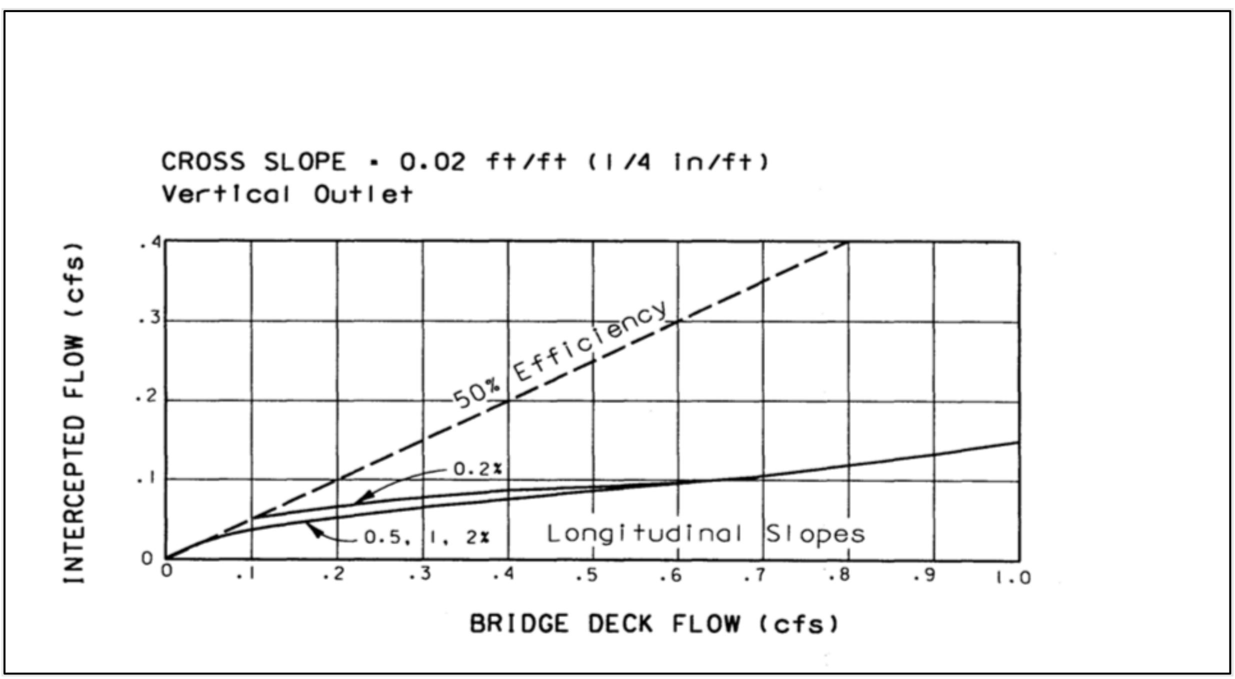


Figure 5.6-4 Intercepted Flow for 4-inch Bridge Scuppers
Cross Slope = 0.02 ft/ft

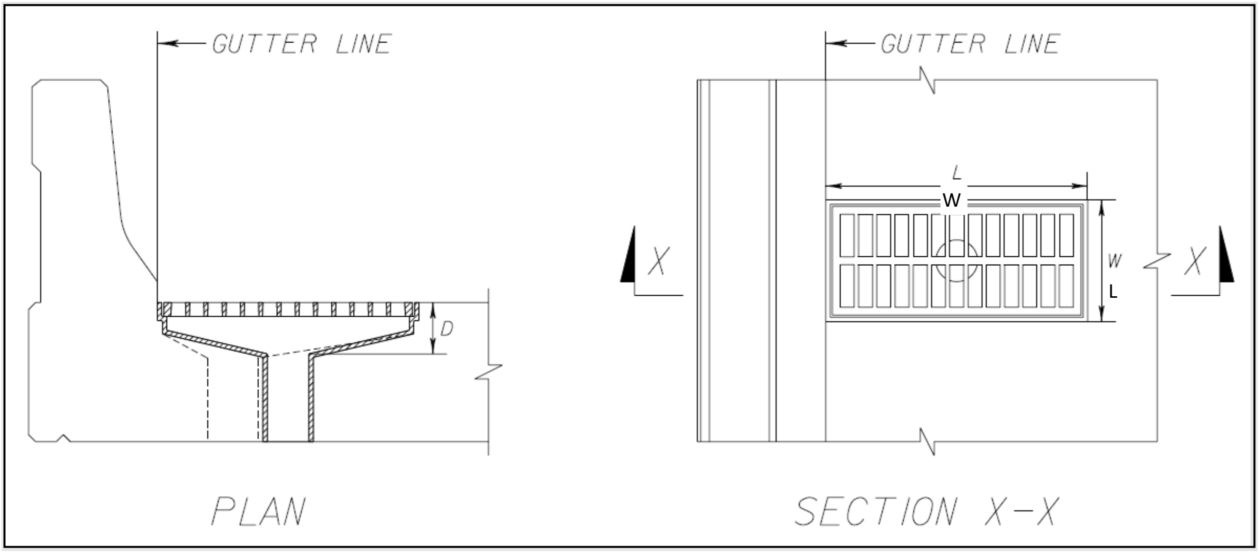


Figure 5.6-5: Grated Free-Draining Scupper

The Department does not have standard grated scuppers or inlets; therefore, it does not have capacity charts as with other standard Department inlets. Section 6.3.1.5 provides references to documents that you can use to derive inlet capacities. Manufacturers may publish capacity charts for their inlets. Keep in mind that 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. Coordinate the dimensions and locations of the inlets with the structural engineer. Use standard prefabricated inlets whenever possible. Refer to Section 7.4 for more information on grated scuppers.

Example 5.6-2
A bridge deck grated scupper is located where the shoulder width is 10 feet and the cross slope is 0.02 ft/ft. The longitudinal grade of the bridge is 1.5%. The dimensions of the grated scupper as defined in Figure 5.6-5 are:

W = 5 feet
L = 1 foot
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.

<u>Solution:</u>

The spread in the gutter prior to the inlet is:

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

Calculate the intercepted flow 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, $Q_W = E_0Q = 0.924\ (1.65) = 1.52$ 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 is called the splash-over velocity. Use Chart 7 of HEC 12 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 usually will 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 estimated conservatively 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:

$$R_F * Q_W = 0.988(1.52) = 1.50 \text{ cfs}$$

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

$$Q_S = Q - Q_W = 1.65 - 1.52 = 0.13 \text{ cfs}$$

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:

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

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.

Also check the capacity of the outlet pipe in the bottom of the scupper inlet 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.

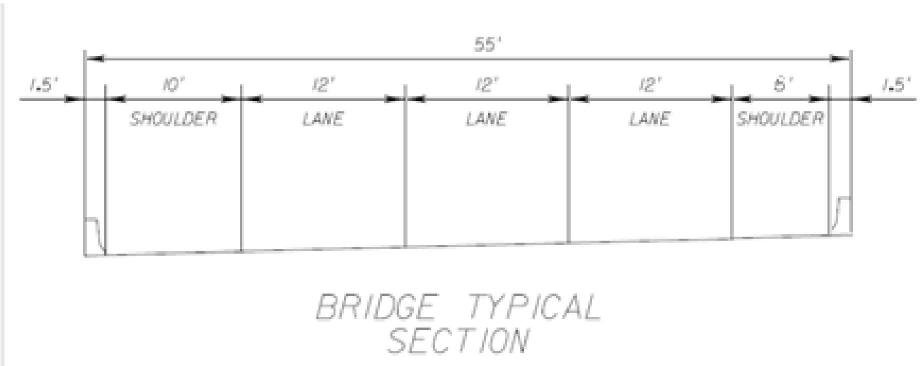
Example 5.6-3
Constant Grade
Scupper flow on bridges with a constant grade will reach an equilibrium state if the bridge is long enough. The equilibrium state occurs when the runoff from the area between scuppers is equal to the flow intercepted by the scuppers.

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 six-lane divided roadway
- The deck has a constant 0.02 ft/ft cross slope
- The typical section has three 12-foot travel lanes, a 10-foot outside shoulder, and a 6-foot inside shoulder. The barrier walls on each side are 1.5 feet wide. The total deck width is 55 feet.

- The longitudinal grade is a constant 0.2 percent. (Normally, the minimum gutter grade of 0.3 percent also should 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.)



<u>Solution:</u>
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 also will be equal to this flow rate. Using 0.096 cfs as the intercepted flow, you can use Figure 5.6-4 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:

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

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 are omitted near the end of a bridge, if not using bridge piping, due to potential soil erosion near the abutments. Add the runoff from this area and the approach slab to the bypass at the last scupper and the combined Q used to check the spread at the end of the approach slab.

Example 5.6-4
For this example, use the information for the bridge in Example 5.6-3, with the following substitutions:

- Omit scuppers in the last 50 feet of the bridge.
- Use a 30-foot approach slab for the bridge.

Determine the spread at the end of the approach slab.

<u>Solution:</u>
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 5.6-3, 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 5.6-3

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

$$Q_{Total} = 0.51 + 0.38 = 0.89\ cfs$$

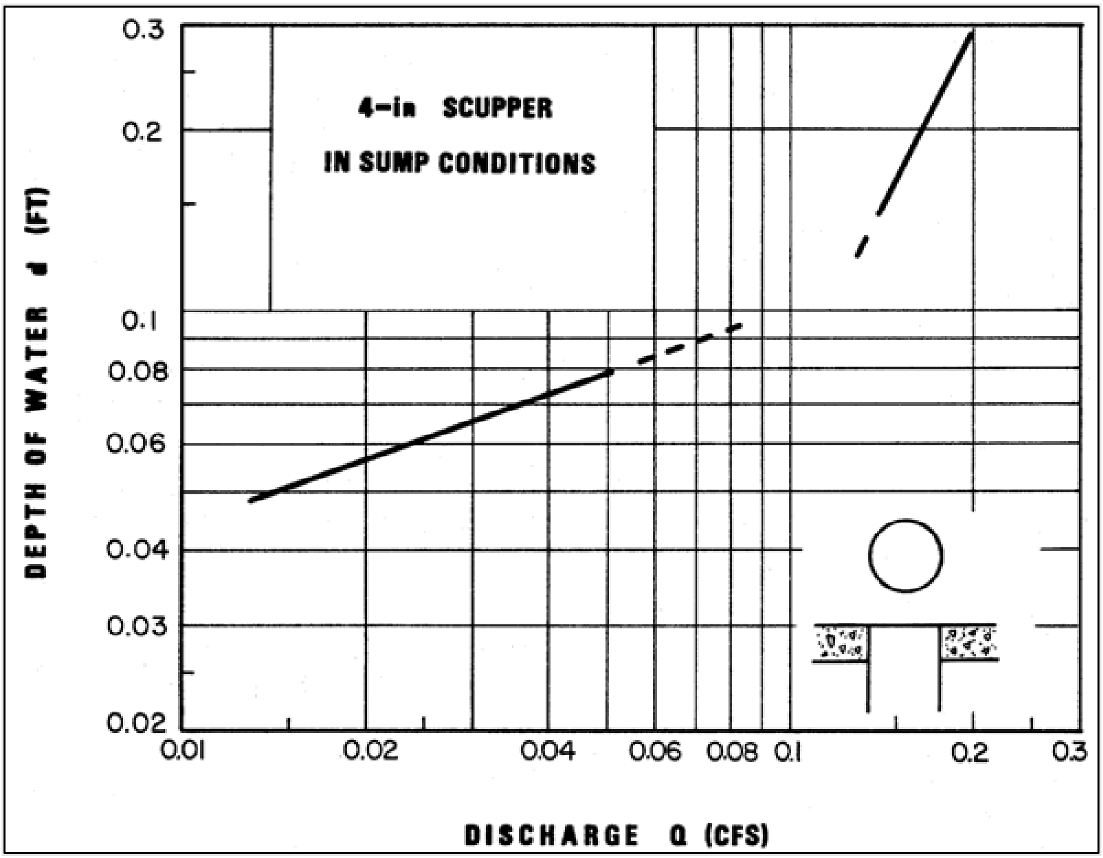
The spread can be conservatively estimated as:

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

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

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 5.6.4.

Example 5.6-5
Flat Grade
You can determine the capacity of a scupper on a bridge with 0-percent longitudinal grade from the figure shown below:



Scupper Capacity in Sump Conditions

Using the bridge from Example 5.6-3, except with a 0-percent grade, determine if standard scuppers are adequate.

Solution:
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 in Example 5.6-3 is 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 than the width of the shoulder, which is 10 feet, standard scuppers meet the criteria.

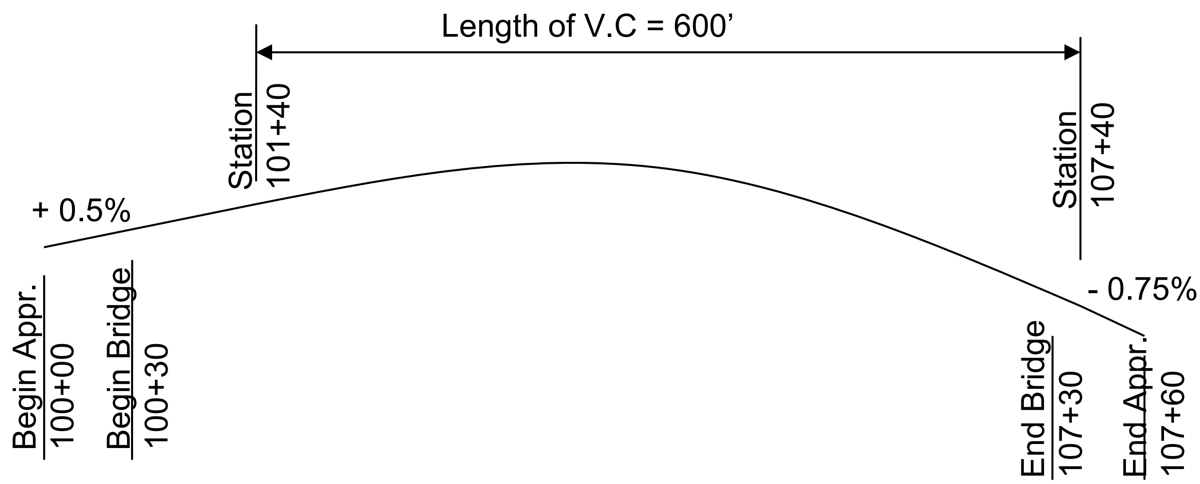
Vertical Curves

Vertical curves complicate the analysis of scupper interception and spacing. However, you can check scuppers on crest curves at various locations by assuming the grade at that location is a constant grade. This will be conservative for crest vertical curves, but also can be overly conservative. Consider using a more detailed analysis procedure, as described in Section 5.6.4, before using scupper spacing that 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 ft/ft, check the spread 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 5.6.4.

Avoid sag vertical curves. If this is not possible, then use the more detailed analysis procedure described in Section 5.6.4.

Example 5.6-6
Use the bridge from Example 5.6-3, except with the following roadway profile information:



The ground beneath 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:

$$X_{\text{HIGH POINT}} = (g1 \times L) / (g2 - g1)$$
$$= (0.005 \times 600) / (0.0075 - 0.005)$$
$$= 240 \text{ feet}$$

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 Chapter 6 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, which is less than 10 feet. However, after discussions with the roadway and the bridge engineers, if you cannot adjust the roadway grade, then consider using standard scuppers. 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, place standard scuppers 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 5.6-5 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 ft/ft will meet the spread criteria.
- Example 5.6-3 shows that standard scuppers on this bridge will meet the spread criteria for grades equal to or greater than 0.002 ft/ft.
- Example 5.6-4 shows that the spread at the end of the approach slab also will meet the spread criteria.

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

The evaluation above uses simplified, but conservative, assumptions of equilibrium flow. If the design failed to meet criteria under the conservative assumptions, then you can perform a more-detailed analysis to evaluate the design. The following will illustrate the detailed analysis procedure and explain how a spreadsheet can be used to automate the analysis.

Enter the values of the cells in Row 1 through Row 8 of the spreadsheet 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 feet, the spacing was entered as 20 feet to conservatively assume that every other scupper was clogged.

The vertical curve data are 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 the 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 four 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 on Figure 5.6-4. 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 ft/ft and 0.005 ft/ft, a value is interpolated between the two curves. Values for these two curves are determined in Column J and Column K.

Column G determines the scupper bypass flow.

Column J and Column K read the flows for the two curves of Figure 5.6-4. In the formulation of this spreadsheet, the curves are represented on another sheet named “Chart.” The values for the 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 changed to the constant of 0.005 ft/ft, 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 feet, the design meets the spread criteria.

As noted above, a separate sheet named “Chart” is included to represent the two curves in Figure 5.6-4. 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	

5.6.4 Closed Collection Systems (Option 3)

The third option is a closed system. You will need to use a closed system 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
- The roadway profile or shoulder width cannot be adjusted

Use grated inlets in closed systems to minimize debris in the piping system. Refer back to Section 5.6.3 for guidance on determining the interception capacity of grated inlets. Coordinate the dimensions and locations of the inlets with the structural designer. Analyze the above-deck design (i.e., size and location of the grated inlets) using a more detailed procedure rather than the equilibrium assumptions from the previous sections. Table 5.6-1 illustrates a typical procedure.

Table 5.6-1: 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 the 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 found at overpasses. Typically, you will locate the inlets near the pier, so there are few horizontal segments of pipe and flow is not combined from multiple inlets. Therefore, the controlling point hydraulically typically will 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 that will carry the combined flow of multiple inlets. Design the below-deck piping system using a procedure similar to the procedure in Chapter 6 of this document. The procedure may be modified to use the driver visibility-limiting rainfall intensity of four inches per hour.

Beside the hydraulic capacity of the piping system, the layout of the system also should 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.

- Underdeck closed drainage system—Design the system to minimize sharp bends, corner joints, junctions, etc. These features occasionally reduce the hydraulic capacity of the system but, more importantly, they provide opportunities for debris to snag and collect. Use Y-connections and bends for collector pipes and downspouts to help prevent clogging in mid-system.
- UV resistance—Pipes should be UV resistant. If they are not, then locate pipes to prevent UV exposure. Tucking the pipe system behind the bridge beams will prevent UV exposure.

Optional material for bridge collection pipes is located in Chapter 22 of the *Structures Detailing Manual.* No matter what type of pipe is used, give attention to the design of a hanger system, which the bridge design engineer should design, or design in coordination with the bridge design engineer. If the collection system is connected to a roadway structure, specify a resilient connector in the plans. For proper design, it is critical that you coordinate with the structures engineer.

5.7 BRIDGE HYDRAULICS REPORT FORMAT AND DOCUMENTATION

Section 4.11.2 of the *Drainage Manual* lists the minimum information that you must include 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. Reviewing this brief paragraph before compiling the documentation can help focus the BHR. Additional general guidance to follow while preparing the BHR is:

- Present the BHR in clear and concise language, without redundant information or unsubstantiated comments.
- Make sure graphics address the technical aspects of the project with the public's point of view in mind.
- Use a consistent report format, as well as consistent units with alternative units presented where appropriate.

5.7.1 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

The required documentation can be organized into this suggested outline.

5.7.1.1 Executive Summary

The Executive Summary should be a concise statement of findings. Describe the existing and proposed bridges. Include a summary of all design recommendations for the proposed bridge crossing (Items 1-10 for Category 1 and 2 bridges from Section 4.11.2.4 of the *Drainage Manual*).

The objective of the Executive Summary is to provide the findings in an opening statement so that when the reviewer assesses the report in the future, the reviewer would immediately understand the reasons for choosing the particular bridge. Include a brief conclusion recounting why you selected the proposed bridge length. The discussion should include other bridge considerations that were pertinent or had an important influence on this project. (For bridge widening, this discussion is not necessary.) 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

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

5.7.1.2 Introduction

The introduction should describe the location of the bridge briefly, 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. Include a figure showing a location map.

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 include only details that affect the hydraulics:

- Bridge length and width
- Span lengths
- Foundation type and sizes
- Low member elevations
- Deck and beam heights
- Bridge Skew
- Abutment type
- Condition of existing abutment and/or pier protection, if any
- Other details that affect the hydraulics such as piles not in line with the flow.

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 potentially will 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 latest Bridge Inspection Report (BIR) and include a copy of the report in an appendix. Discuss any information obtained from contact with Department Maintenance.

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

5.7.1.3 Floodplain Requirements

Discuss requirements of FEMA and other regulatory agencies (Section 5.1.2) 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.

5.7.1.4 Hydrology

Discuss the methods used to determine and check the flow rates applied 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.

5.7.1.5 Hydraulics

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 5.7-1(1) and 5.7-2 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

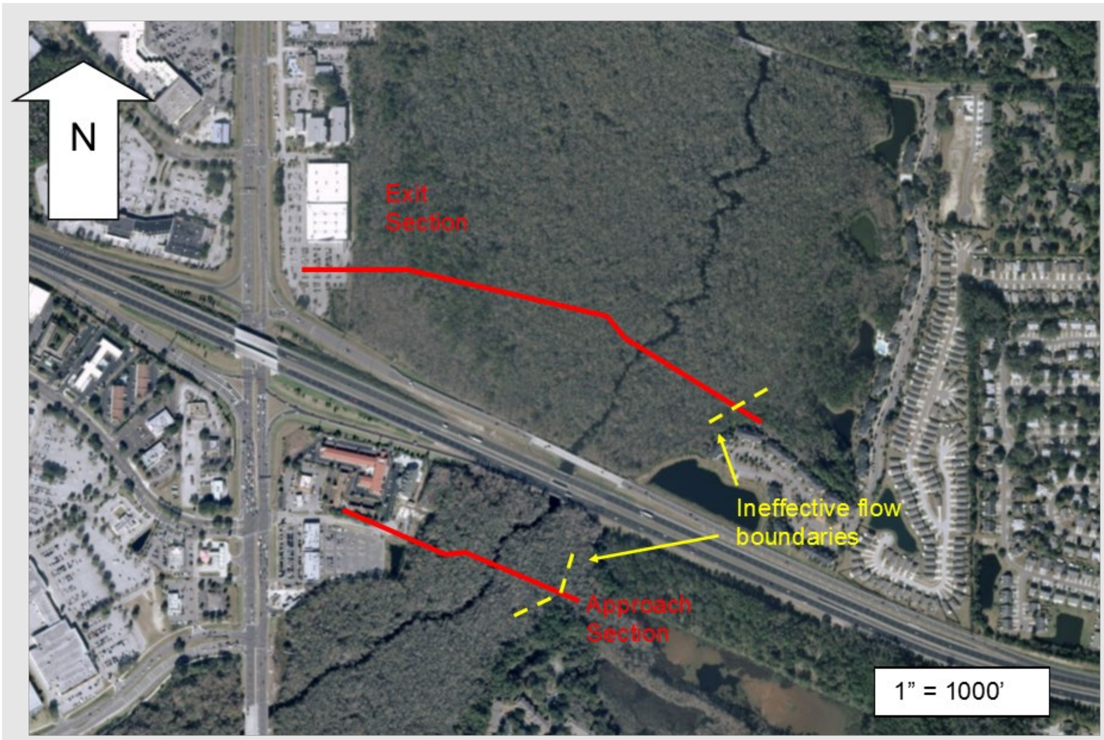


Figure 5.7-1: Example Cross Section Location Figure on an Aerial

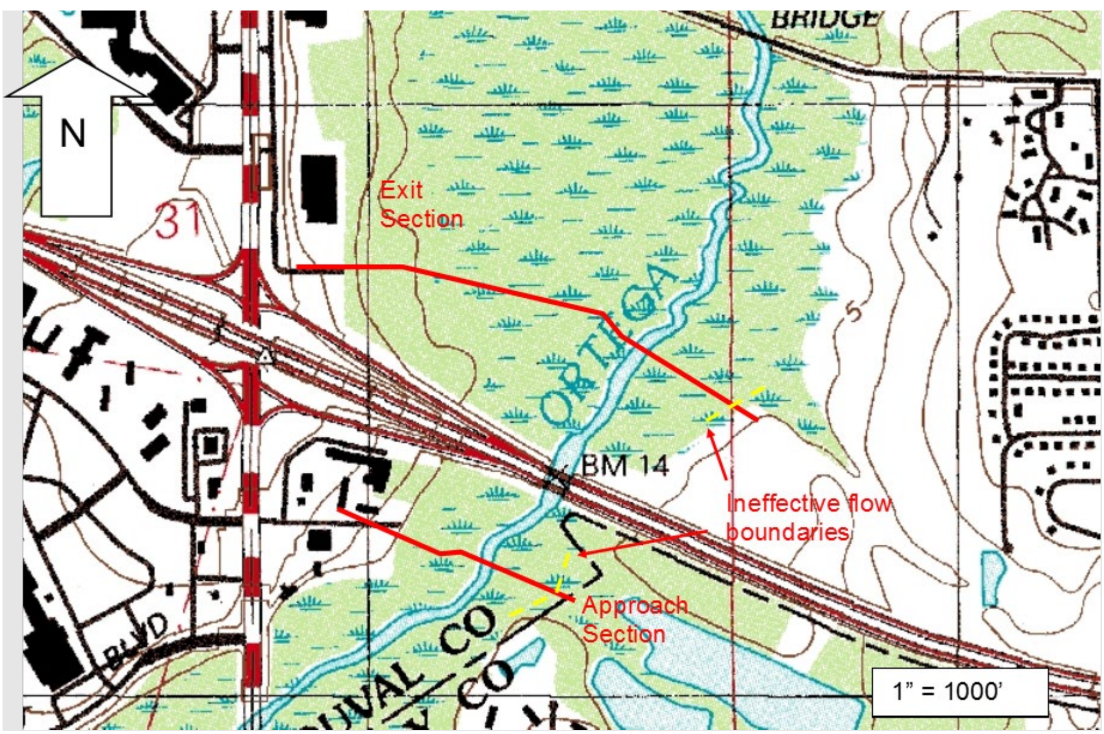


Figure 5.7-2: Example Cross Section Location Figure on a Quadrangle Map

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 appendixes. Electronic copies of the input files also will 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, you must still include the flood data at the site in the plans, per FHWA requirements. If you do not calculate the flood data, then you must obtain them 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 bridges. For the comparison to be valid, perform the water surface elevation comparisons at a section that is at a common location in each model. As illustrated in Figure 5.7-3, make the comparison at the location of the approach section that is farthest upstream.

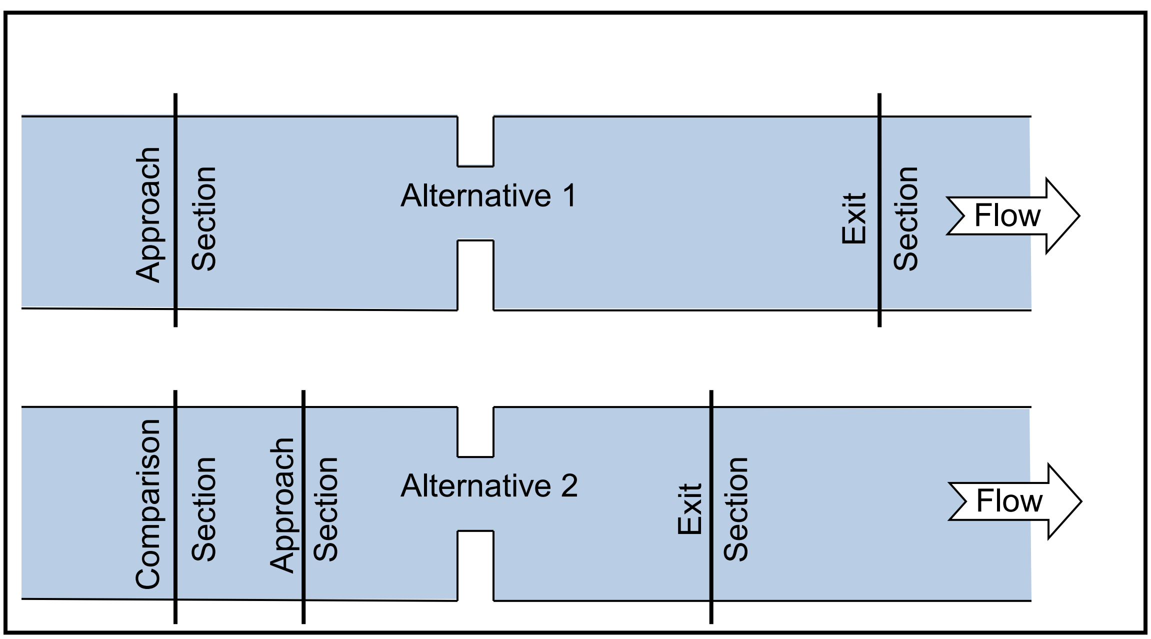


Figure 5.7-3: Water Surface Elevation Comparisons

Include a table that summarizes the water surface elevations for the existing and alternative bridges. Table 5.7-1 is an example of a table comparing water surface elevations.

Table 5.7-1: 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 feet.

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 5.7-4 through Figure 5.7-6). 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.

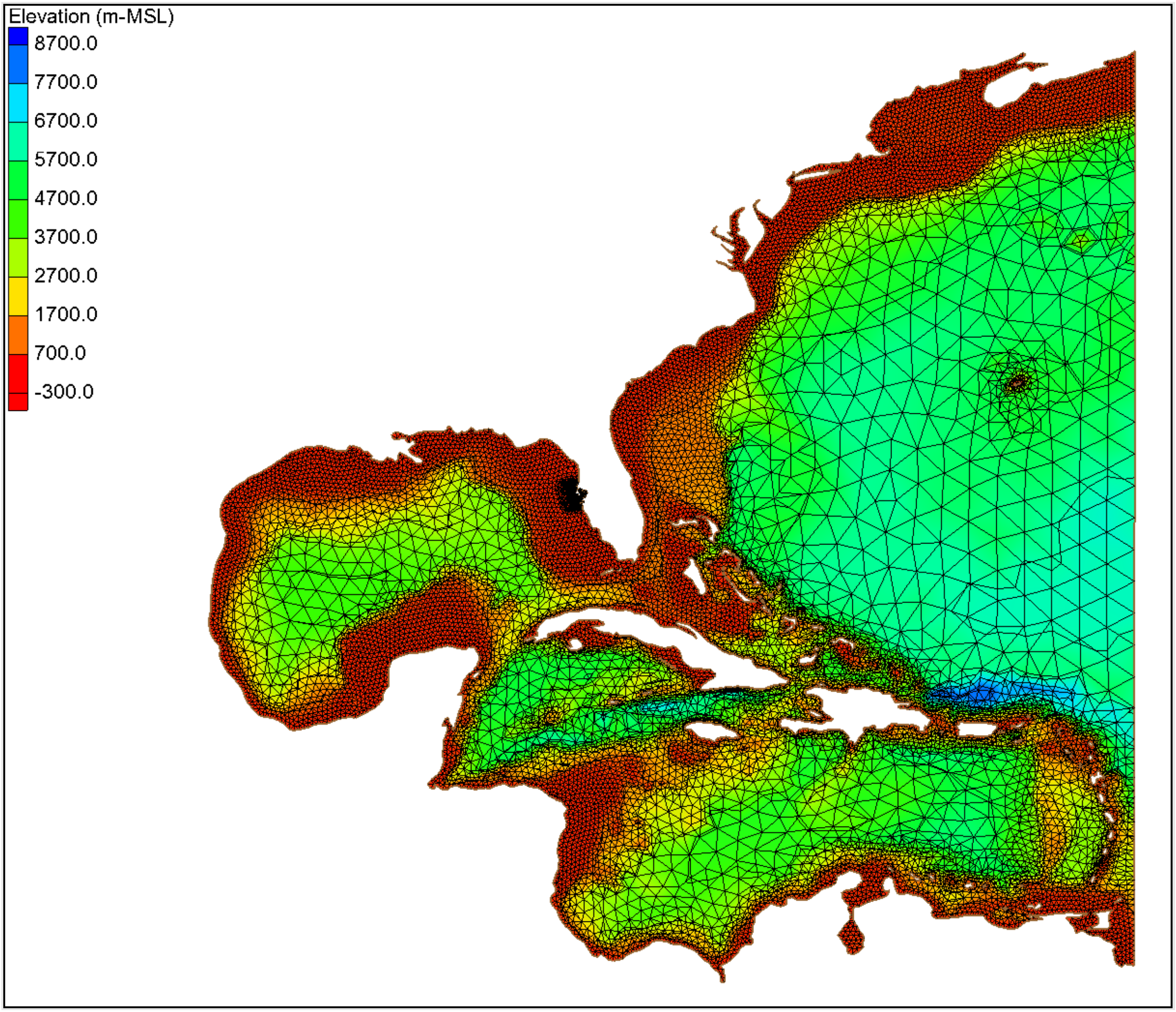


Figure 5.7-4: Tampa Bay Model Mesh Domain

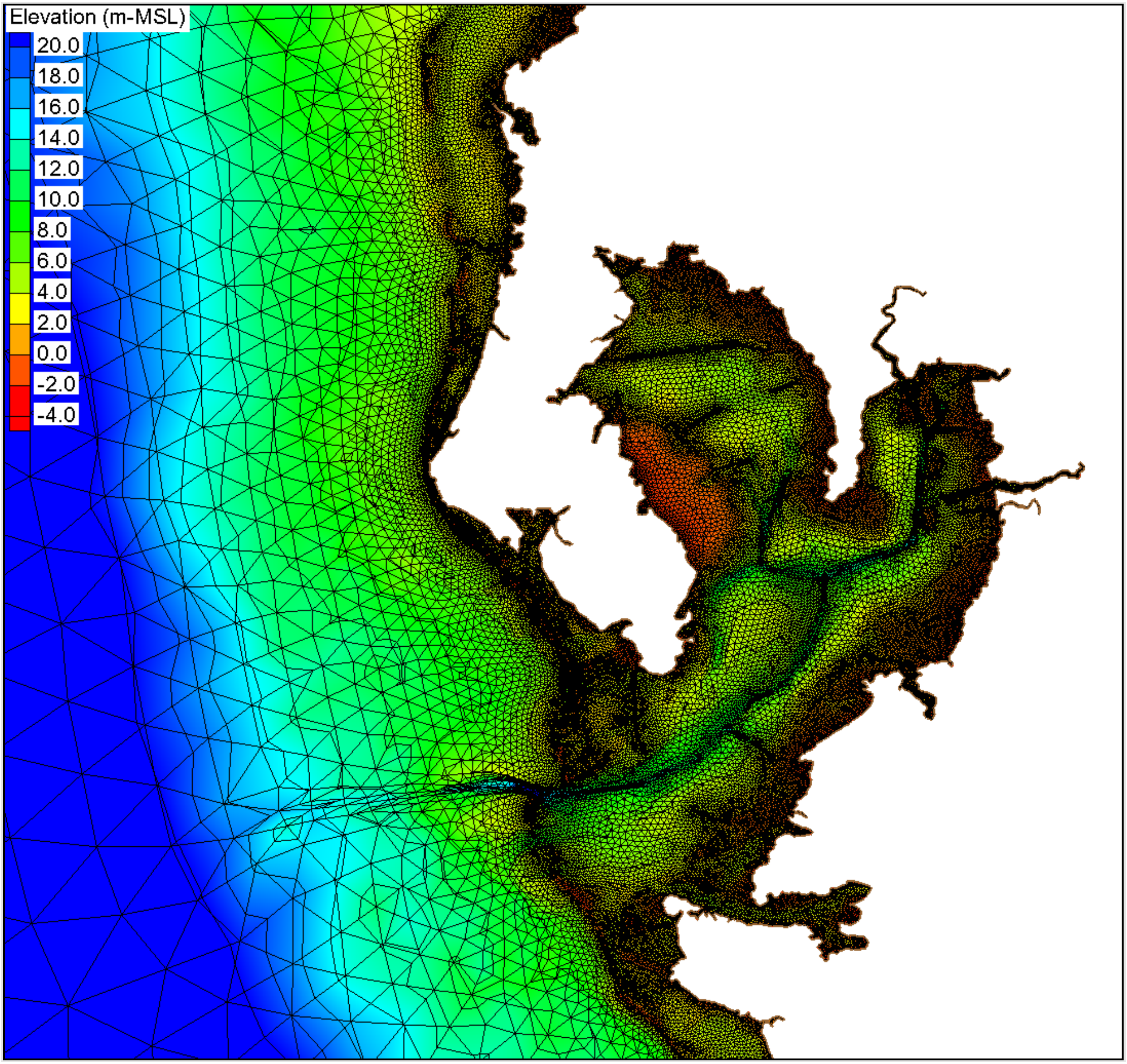


Figure 5.7-5: Model Mesh at Tampa Bay

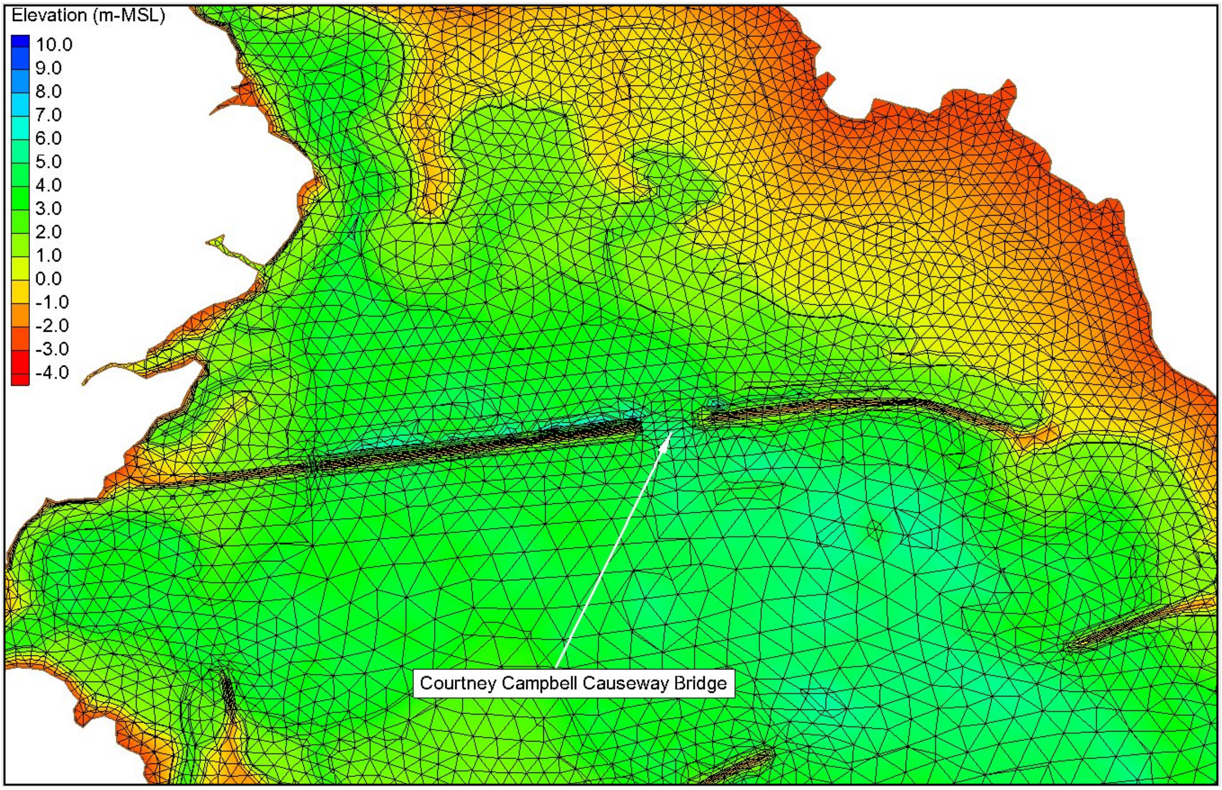


Figure 5.7-6: Model Mesh at the Courtney Campbell Causeway Bridge

Documentation of the two-dimensional model should include:

- A complete description of the calibration process
    - Calibration data
    - The simulations
    - Parameters changed to achieve calibration
    - Parameters of the model
- Both a qualitative and quantitative description of the model's capability to predict measured data
    - Calculation of mean error
    - Standard deviation
    - 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 5.7-7 and Figure 5.7-8, which show comparisons between measured and modeled water surface elevations and flow rates.

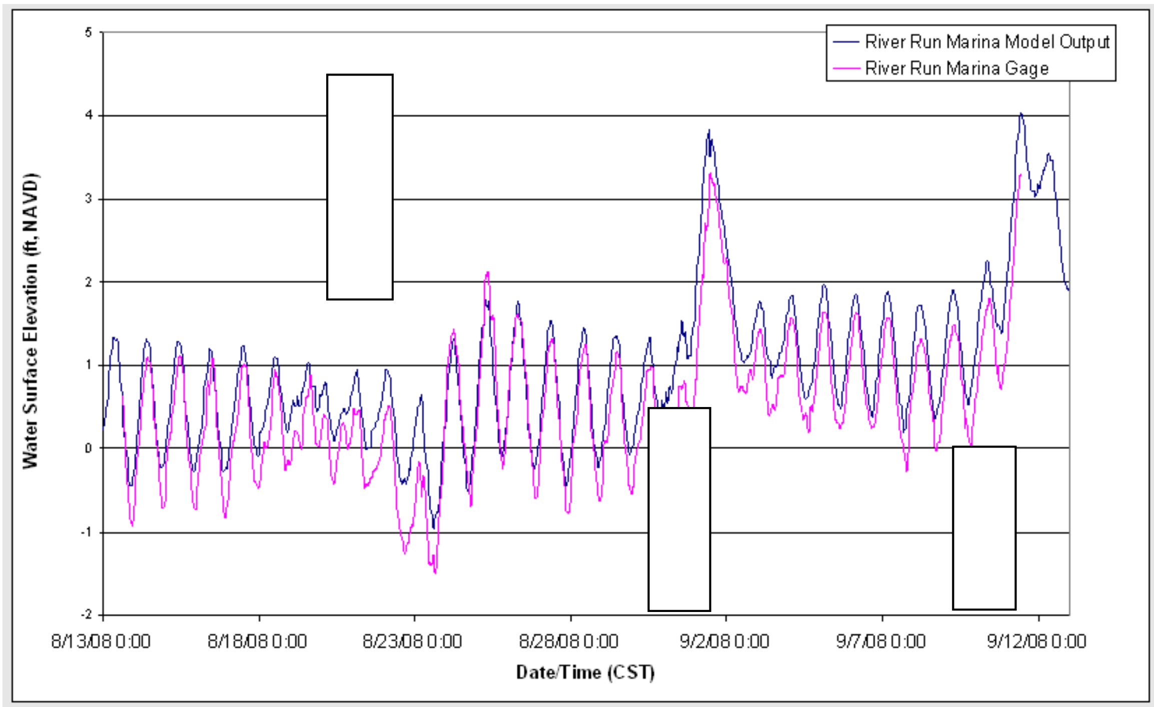


Figure 5.7-7: Model Calibration Plot for the US 90 Bridge over Macavis Bayou Replacement Project at the River Run Marina

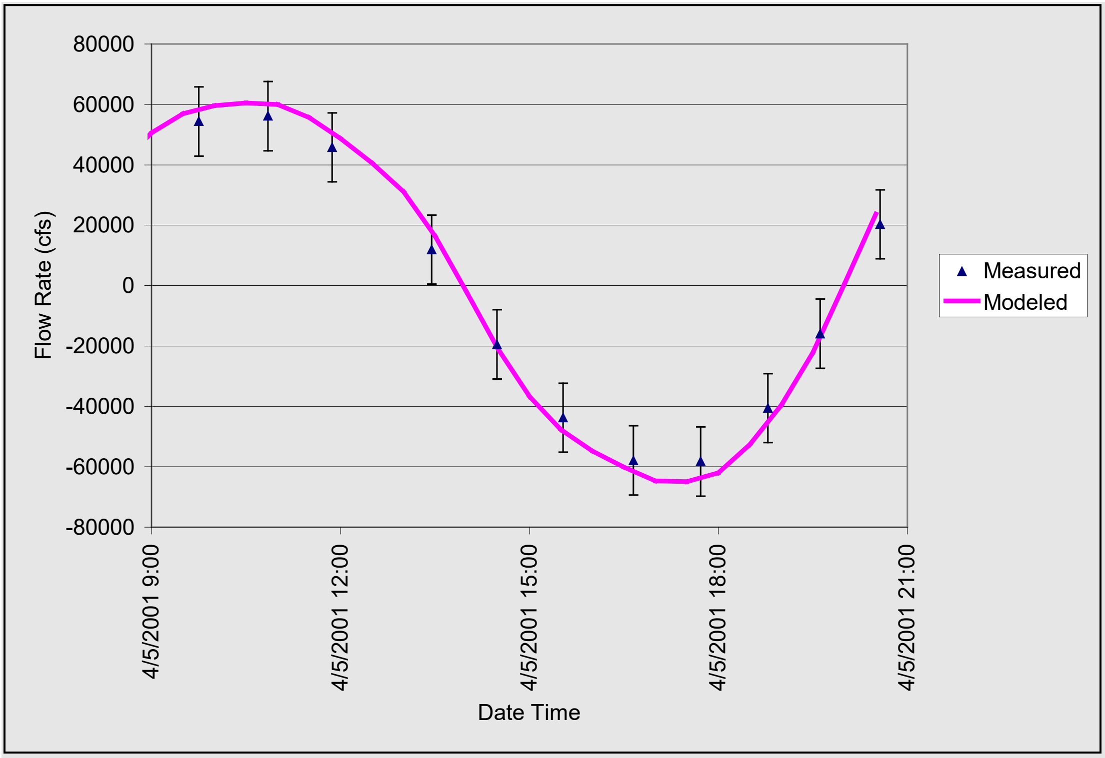


Figure 5.7-8: Flow Rate Calibration at Lake Worth Inlet (Error Bars Indicate 10% Error)

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 5.7-9):
    - Display contours of velocity magnitude
    - Velocity vectors displaying the direction of the flow across bridge
- For long bridges, hydraulic parameters at each pier or groups of piers should list:
    - Max stage
    - Max flow rate
    - Max velocity
    - Angle of attack
- Tidal analysis (time-dependent simulation)
    - Time series plot of design values for stage, velocity, and flow rate (Figure 5.7-10 through Figure 5.7-12)

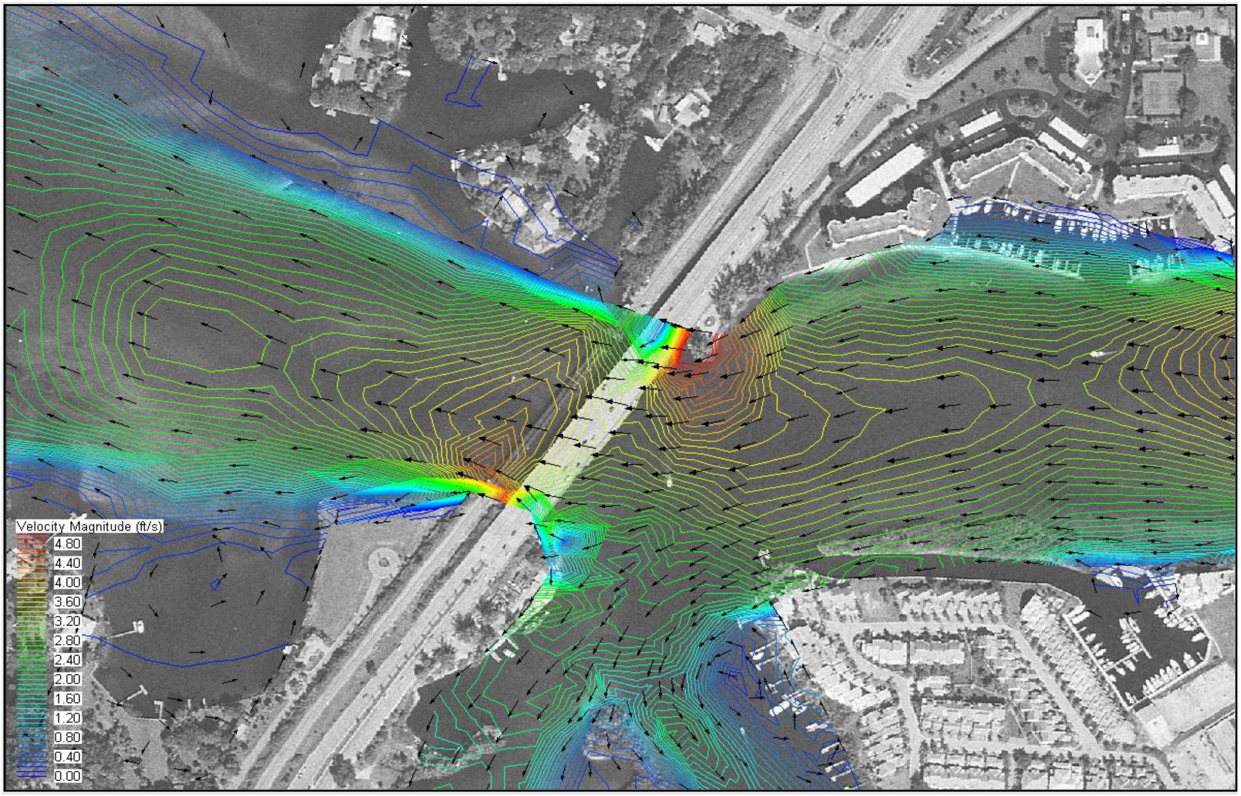


Figure 5.7-9: Velocity Magnitude Contours and Velocity Vectors at the Time of Maximum Velocity during the 100-Year Storm Surge Event at the SR-A1A Bridge over the Loxahatchee River

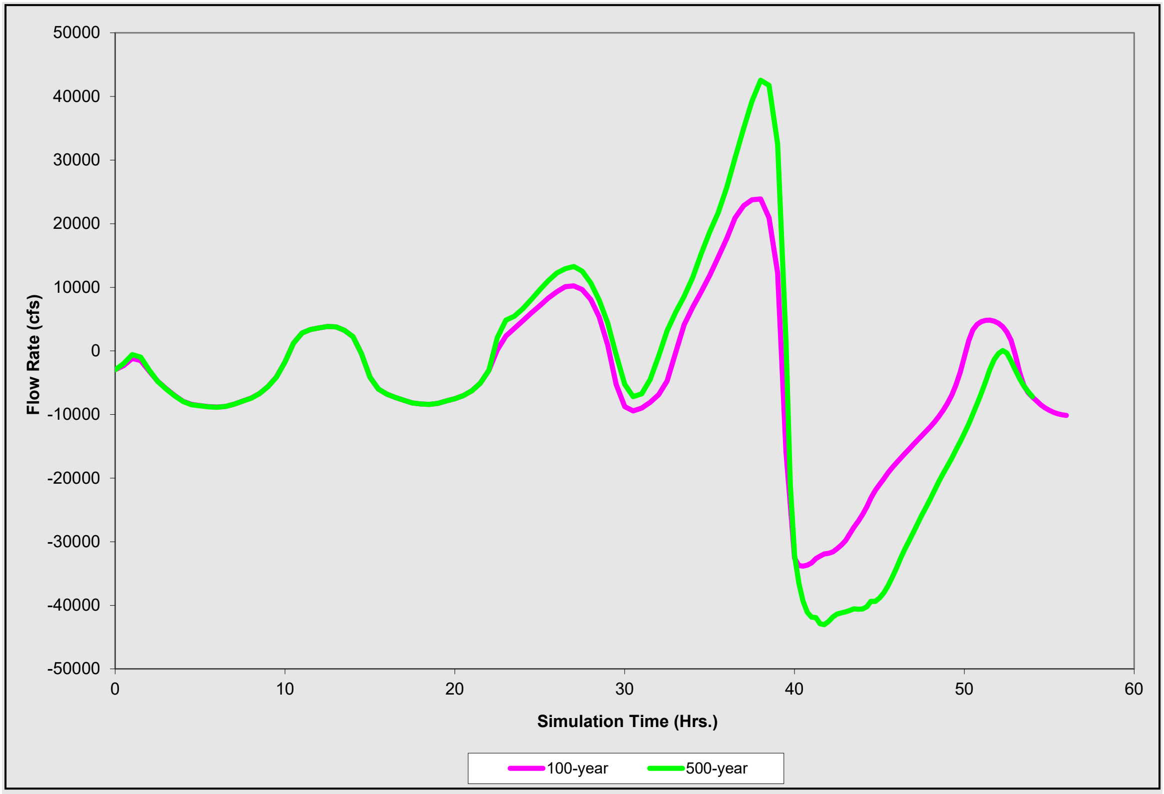


Figure 5.7-10: Flow Rate Time Series during the Design and Check Event at the SR-A1A Bridge over the Loxahatchee River

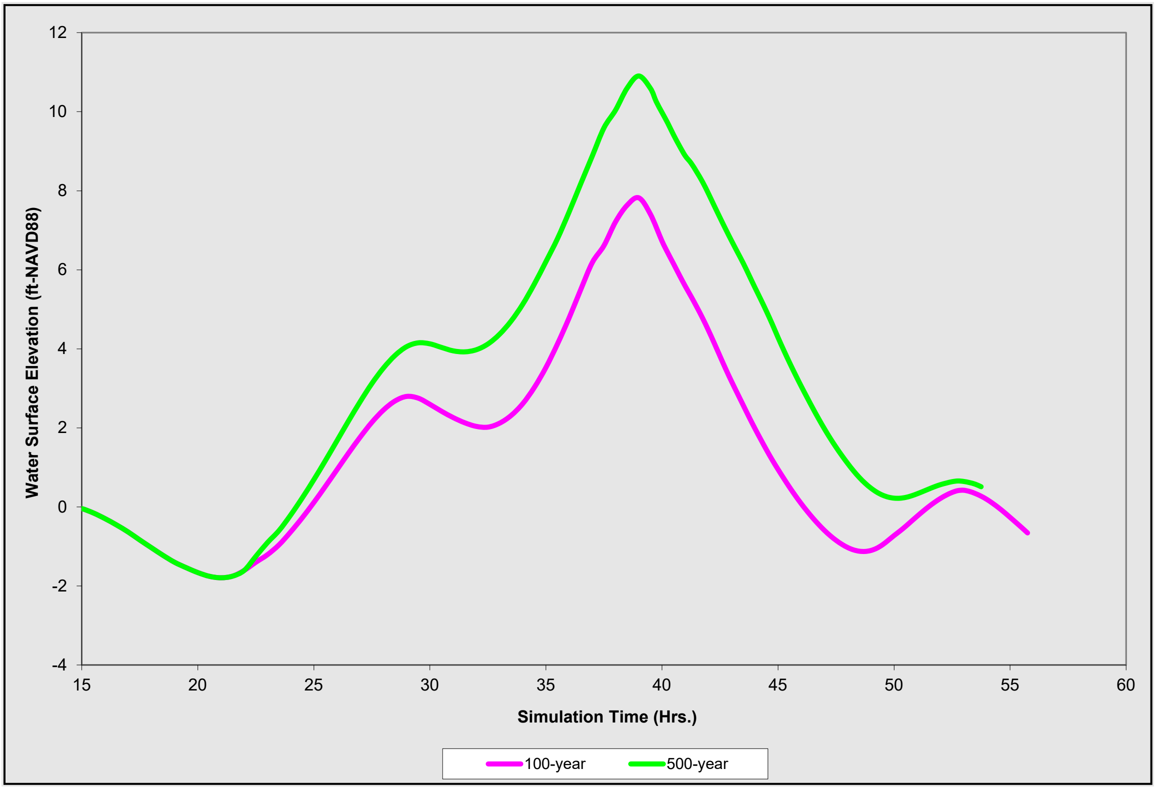


Figure 5.7-11: Water Surface Elevation Time Series during the Design and Check Event at the SR-A1A Bridge over the Loxahatchee River

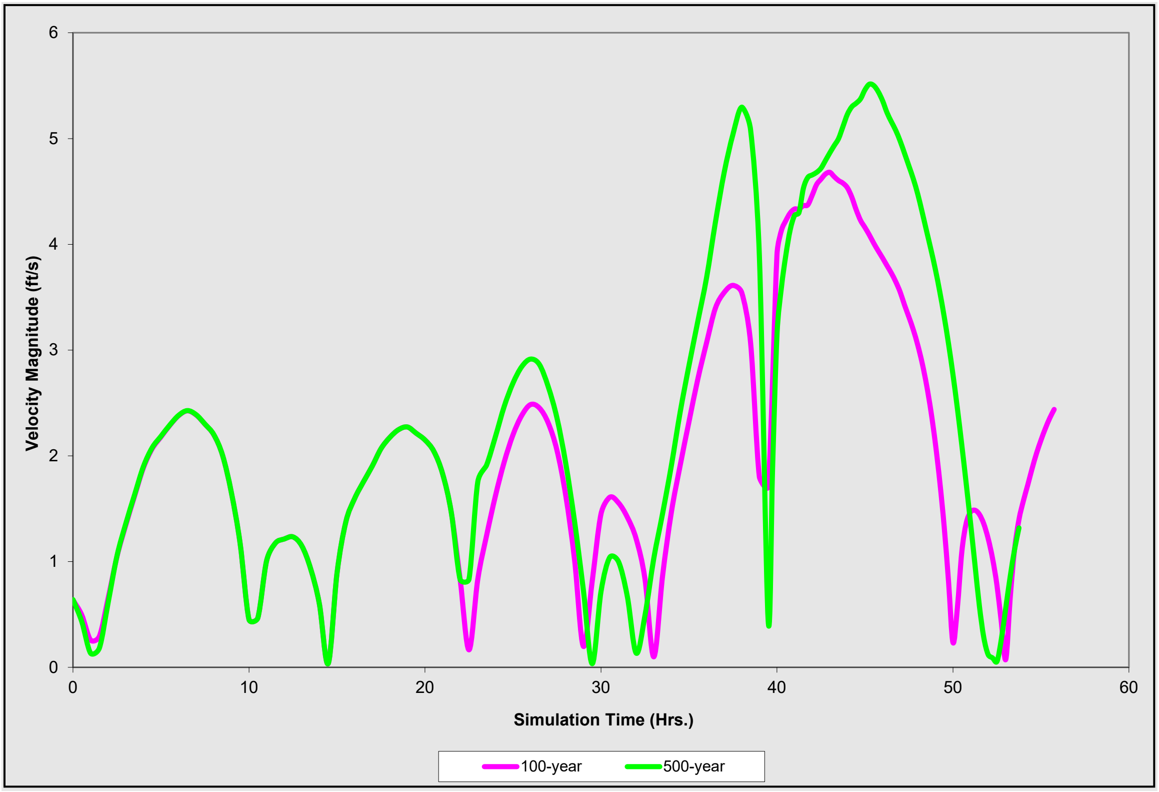


Figure 5.7-12: Velocity Magnitude Time Series during the Design and Check Event at the SR-A1A Bridge over the Loxahatchee River

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.

Alternatives Analysis

You will not need this section 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 still should 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.

5.7.1.6 Scour

You should plan to include a discussion of the stream geomorphology, the scour history, the long-term aggradation or degradation, and the scour values, including information on the methods used to determine each of these items. Plot scour depths in a figure.

Discuss the proposed abutment protection. If using one of the standard abutment protection designs given in Section 4.9.3.2 of the *Drainage Manual*, abutment scour need not be calculated and plotted. You may use other abutment protection designs in certain circumstances, but not without prior approval from the District Drainage Office.

5.7.1.7 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 collection systems.

5.7.1.8 Appendices

Include calculations and other backup documentation as appendixes to the BHR to avoid disrupting the flow of the main body of the report. Items to consider including in the appendixes are:

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

5.7.2 Bridge Hydraulics Report Process

FDM 250 specifies the multidisciplinary approach to follow for scour consideration, along with submittal requirements. Prepare the BHR in conjunction with the Bridge

Development Report and preliminary Structures Plans. Figure 250.2.1 of the *FDM* outlines a flow chart for the Structural Plans Development Process.

The process flow chart in Figure 5.7-13 shows the general sequence of events necessary to prepare a Bridge Hydraulics Report. You also may need to perform additional coordination, especially for projects involving floodways or for other complex elements.

After you have a relatively good idea of the approximate structure length and location, you should conduct a field review. Then, submit the preliminary structure length and location, along with preliminary scour depths and low member elevations to the Structures Design Office for their preliminary evaluation. After you have developed the proposed bridge configuration and foundation type and submitted them back for review, perform the final hydraulic and scour analyses and submit them to the Structures and Geotechnical Departments.

Have the BHR and BHRS reviewed internally (or by an outside consultant, if necessary). After you have addressed all comments, **approve** the BHR and BHRS and submit them to the Department for **concurrence**. After the BHR and BHRS receive concurrence from the Department, the final BHR and BHRS should be submitted to the structural and geotechnical engineers so that they can complete the BDR and geotechnical reports.

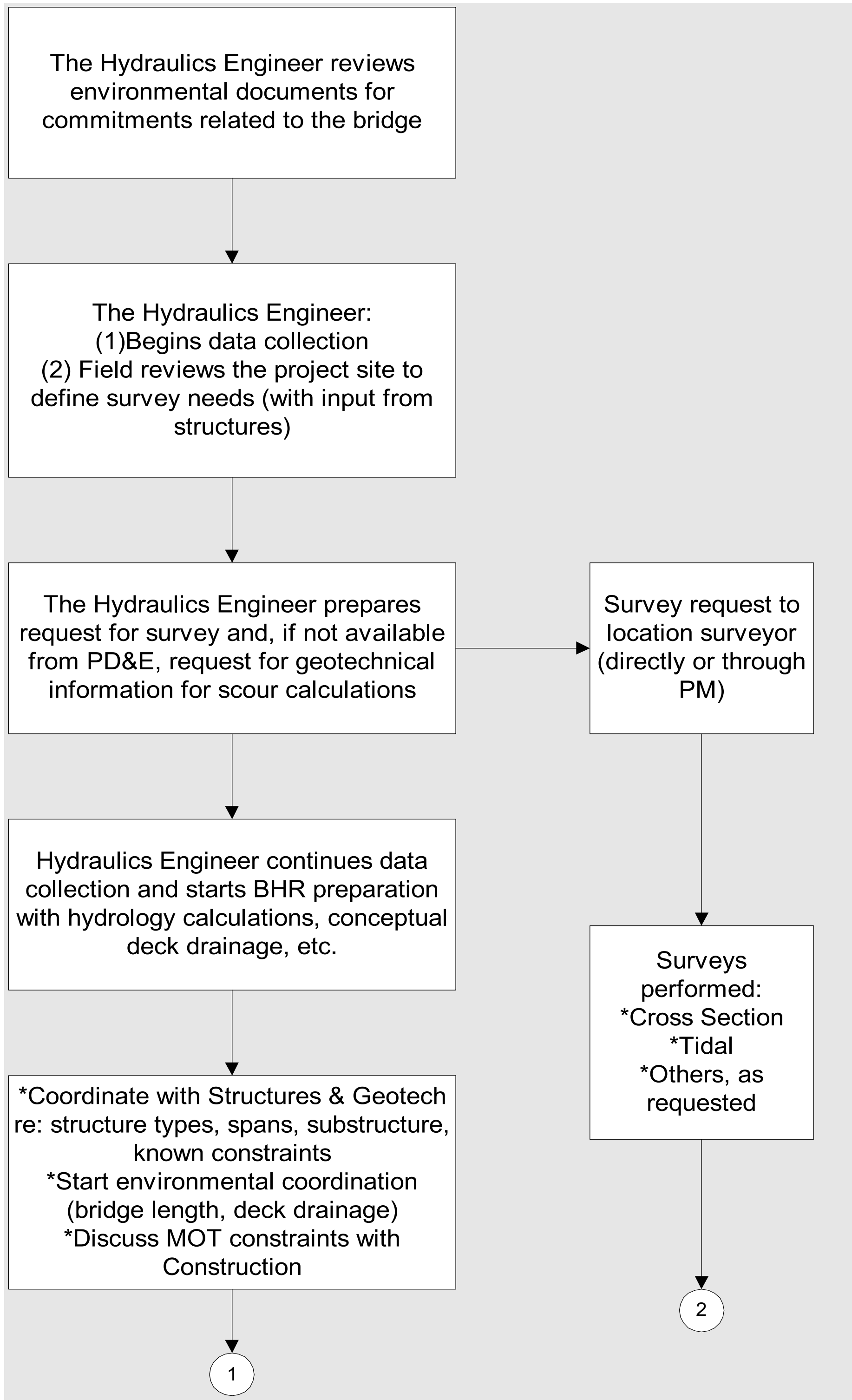


Figure 5.7-13: Bridge Hydraulics Report Process

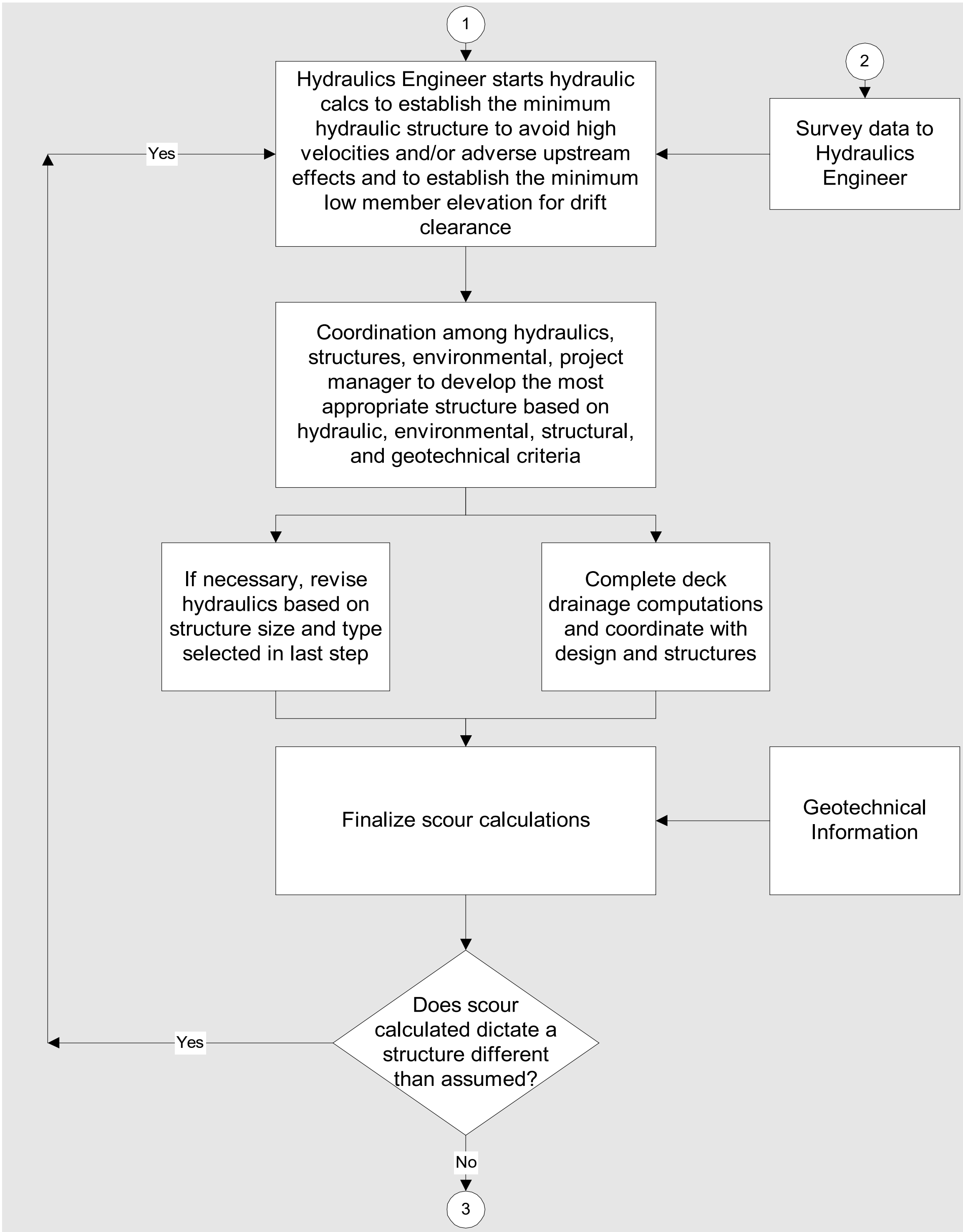


Figure 5.7-13: Bridge Hydraulics Report Process (continued)

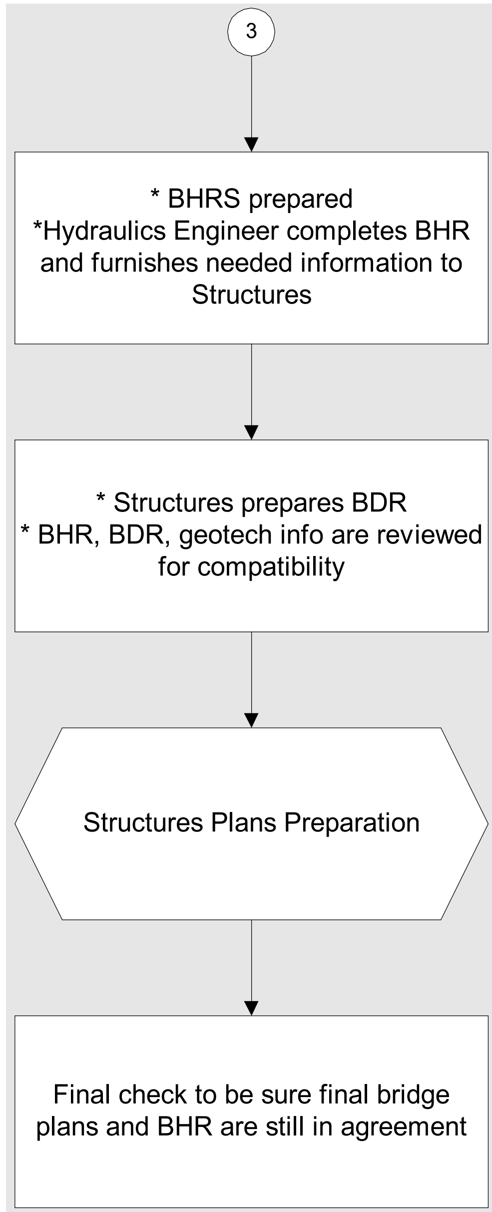


Figure 5.7-13: Bridge Hydraulics Report Process (continued)

5.7.3 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 Department:

- 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 these 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
        - Report format is consistent
        - Units are consistent, with alternative units presented where appropriate
        - Cross referencing of figures, tables, section numbers within the document have been double-checked
- 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)
            - 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)
  - Maximum 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 Department 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 Department review process

5.7.4 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 those given in the BHR and computer output.

The BHRS is divided into four sections:

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

FDM 305 gives the minimum requirements of the first three sections. In addition, consider the following items:

- In the Plan View, the *FDM* 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, you should show 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). Display local scour holes 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, you do not need to show percent of grade. Plot the PC, PI, and PT of vertical curves using their respective standard symbols; however, there is no need to note data (station, elevation, length of curve). Flag begin and end bridge stations.

Figure 5.7-14 shows a larger view of the section 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 are 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 5.7-14: BHRS Required Data

(1) Existing Structures: Structure 1 refers to the structure being replaced or modified. Structures 2, 3, and 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 of 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 that 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 that 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 the 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 Standard Plans, Index 400-289, Sheet 1, 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. See the *FDM* 260 for the minimum requirements. Other agencies may have minimum clearance requirements.

(40) Navigation Clearance (Vert.) (ft): The minimum vertical navigation clearance in feet required. The Department minimum clearances are given in the *FDM 260.* Other agencies may have minimum clearance requirements.

(41) Drift Clearance (Horiz.) (ft): The minimum horizontal debris drift clearance in feet required. The Department minimum clearances are given in the *FDM 260*.

(42) Drift Clearance (Vert.) (ft): The minimum vertical debris drift clearance in feet required above the design flood. The Department minimum clearances are given in *FDM 260*.

(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 5.5.4 of this document for details.

(46) Toe Horizontal Distance (ft): Horizontal extent in feet of the rubble protection measured from the toe of the abutment. See Section 5.5.4 of this document 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 must be a minimum of 1 foot above the wave crest elevation. The Department minimum clearances are given in *FDM 260*.