Chapter 12 – Bridge and Structure Hydraulics

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Chapter 12 - Bridge and Structure Hydraulics

12.1 Introduction

12.1.1 Definition

Bridges are defined as:

- Structures that transport traffic over waterways or other obstructions
- Part of a stream crossing system that includes the approach roadway over the floodplain, relief openings, and the bridge structure
- Structures with a centerline span of 20 feet or more. However, structures designed hydraulically as bridges, as described above, are treated as bridges in this chapter, regardless of length

12.1.2 Analysis/Design

Proper hydraulic analysis and design is as vital as the structural design. Stream crossing systems should be designed for:

- Minimum cost subject to criteria
- Desired level of hydraulic performance up to an acceptable risk level
- Mitigation of impacts on stream environment
- Accomplishment of social, economic, and environmental goals
- Full compliance with the requirements of existing Federal Emergency Management Agency (FEMA) or other officially delineated or regulatory floodplains

12.2 Design Policy

12.2.1 FEMA Floodplain Compliance

- The final design selection should be in full compliance with the maximum water surface elevation allowed by FEMA
- The final design should not significantly alter the flow distribution in the floodplain
- Where design considerations permit, the "crest-vertical curve profile" should be considered as the preferred highway crossing profile when allowing for embankment overtopping at a lower discharge
- Degradation or aggradation of the river should be estimated and contraction and local scour determined and appropriate positioning of the foundation, below the total scour depth if practicable, should be included as part of the final design

12.3 Design Criteria

12.3.1 AASHTO General Criteria

Design criteria are the tangible means for placing accepted policies into action and become the basis for the selection of the final design configuration of the stream-crossing system. Criteria are subject to change when conditions so dictate as approved by the Department.

Following are certain American Association of State Highway Transportation Officials (AASHTO) general criteria adopted by the Department related to the hydraulic analyses for bridges as stated in their highway drainage guidelines:

- Backwater will not significantly increase flood damage to property upstream of the crossing
- Velocities through the structure(s) will not damage either the highway facility or increase damages to adjacent property
- Maintain the existing flow distribution to the extent practicable
- Pier spacing and orientation and abutment designed to minimize flow disruption and potential scour
- Foundation design and/or scour countermeasures to avoid failure by scour
- Minimal disruption of ecosystems and values unique to the floodplain and stream

12.3.2 Department Criteria

These criteria augment the general criteria. They provide specific, quantifiable values that relate to local site conditions. Evaluation of various alternatives according to these criteria can be accomplished by using water surface profile programs such as HEC-RAS, HEC-2, or WSPRO.

12.3.2.1 Travelway

Inundation of the travelway dictates the level of traffic services provided by the facility. The travelway overtopping flood level identifies the limit of serviceability. Desired minimum levels of protection from travelway inundation for functional classifications of roadways are presented in Chapter 6, Hydrology.

12.3.2.2 Risk Evaluation

The selection of hydraulic design criteria for determining the waterway opening, road grade, scour potential, riprap, and other features should consider the potential impacts to:

- Traffic
- Adjacent property
- Environment
- Infrastructure of the highway

The consideration of the potential impacts constitutes an assessment of risk for the specific site. The least total expected cost (LTEC) alternative should be developed in accordance with Federal Highway Administration (FHWA) HEC-17 only where a need for this type of analysis is indicated by the risk assessment. This analysis provides a comparison between other alternatives developed in response to considerations such as environmental, regulatory, and political.

12.3.2.3 Design Floods

Design floods for such things as the evaluation of backwater, clearance, and overtopping, unless available from FEMA or other appropriate sources, should be established predicated on local site conditions. They should reflect consideration of traffic service, environmental impact, property damage, hazard to human life and floodplain management criteria. Design floods for roadway inundation are specified in Chapter 6, Hydrology. It should be noted, in the case of bridged waterways, that the design flood is normally whichever of the customarily documented events (i.e. the 2, 5, 10, 25, 50, 100, & 500-yr. floods) that will pass under the bridge superstructure at its lowest elevation with at least one or more feet of freeboard, provided that level of protection is acceptable to the bridge designer.

12.3.2.4 Backwater/Increases Over Existing Conditions

Designers shall conform to FEMA regulations for sites covered by the National Flood Insurance Program (NFIP). It is the Department's policy not to allow any increase in the level of the 1 percent flood for delineated floodplains established under the NFIP and for the increase to not exceed one foot during the passage of the 1 percent flood for sites not covered by NFIP. Refer to section 12.6.1 for additional details.

12.3.2.5 Clearance

Where practical a minimum clearance of one foot should be provided between the design approach water surface elevation and the low chord of the bridge for the design flood. Where this is not practicable, the bridge designer should establish the clearance based on the desired level of protection.

12.3.2.6 Flow Distribution

The conveyance of the proposed stream crossing should be calculated to determine the flow distribution and to establish the location of bridge opening(s). The proposed facility should not cause any significant change in the existing flow distribution. Relief openings in the approach roadway embankment or other appropriate measures should be investigated if there is more than a 10 percent redistribution of flow.

12.3.2.7 Scour

Design for bridge foundation scour should consider the magnitude of the flood that generates the maximum scour depth. The design should use a geotechnical design practice factor of safety from 2 to 3. The resulting design should then be checked using a super-flood such as the 0.2 percent event and a geotechnical design practice safety factor of at least 1.0. A plot or sketch showing the scoured bed profile for both the design and super-flood events shall be prepared and included with documentation (LD-293) described in Section 12.6.5.2.

12.4 Design Concepts

12.4.1 Methodologies

A step-backwater computer model is usually employed to perform the hydraulic analysis in these situations due to the complexity of the hydraulic conditions and the risk involved. No single method is ideally suited for all situations. If a satisfactory computation cannot be achieved with a given method, an alternate method should be attempted. However, it has been found that, with careful attention to the setup requirements of each method, acceptable results can usually be achieved regardless of the step-backwater computer model being employed. Where the use of a one-dimensional step backwater computer model is indicated, the Department accepts any computer model currently approved by FEMA but prefers HEC-2 or HEC-RAS.

12.4.2 Bridge Scour or Aggradation

The Department employs the procedures and criteria presented in the FHWA's "Evaluating Scour at Bridges" (HEC-18) and "Stream Stability at Highway Structures" (HEC-20) to determine and counteract the impact of scour and long term aggradation/degradation on bridges. Both these publications can be accessed and/or downloaded from the publications section of the FHWA's Internet web site at http://www.fhwa.dot.gov/bridge/hydpub.htm .

12.4.3 Riprap

Riprap is not to be used for scour protection at piers for new bridges. Riprap may be used to protect exposed abutment slopes or as a scour countermeasure at existing bridge piers and abutments. Design guidelines for placement and sizing of riprap are presented in the FHWA's "Bridge Scour and Stream Instability Countermeasures" (HEC-23) publication. This publication can be accessed and/or downloaded from the publications section of the FHWA's Internet web site at http://www.fhwa.dot.gov/bridge/hydpub.htm.

12.5 Design Procedure

12.5.1 Hydraulic Performance of Bridges

The stream-crossing system is subject to either free-surface flow or pressure flow through one or more bridge openings with possible embankment overtopping. These hydraulic complexities should be analyzed using a one-dimensional step backwater computer model (i.e. all flow is assumed to be proceeding in a downstream direction). See Section 12.4.1 for the Department's recommendation for an appropriate "1-D" computer model. There will be situations where a two-dimensional model (i.e. flow can move in a lateral as well as downstream direction) such as FESWMS would be more appropriate. The use of a two-dimensional model in any given situation should be approved by the Department's Hydraulics Section.

It is impracticable to perform the hydraulic analysis for a bridge by manual calculations due to the interactive and complex nature of those computations. However, the procedures recommended by the FHWA are described in their publication *Hydraulics of Bridge Waterways* (Hydraulic Design Series No. 1).

12.6 Performance and Documentation of Riverine H&HA

12.6.1 Background

A detailed hydrologic and hydraulic analysis (H&HA) should be performed for all of the Department's new or replacement major drainage structures, bridged waterways and significant lateral encroachments (resulting from the placement of highway fill embankments within a floodplain). It is necessary to do this such that Department construction is in compliance with national (i.e. FHWA, FEMA, etc.), state Department of Conservation and Recreation (DCR, etc.), and municipal (locally delineated floodplains) rules and regulations. Detailed analysis, as used here, means that the hydraulic analysis shall be performed using an appropriate step-backwater computer model. See Section 12.4.1 for recommendations. In the case of Department construction in or in proximity to a FEMA floodplain, the same step-backwater computer model used to establish the FEMA floodplain should be employed to assess the impact of the Department's construction. A detailed H&HA should be performed for all bridged waterways regardless of whether or not it falls within a FEMA or other officially delineated floodplain. A detailed H&HA should also be performed for floodplain encroachments (brought about as a result of filling in conjunction with VDOT construction) when they fall within a FEMA or other officially delineated floodplain.

Major culvert installations that do not fall within a FEMA or other officially delineated floodplain may be analyzed using procedures such as presented in the FHWA's Hydraulic Design of Highway Culverts (HDS-5) publication. A "major culvert" in this sense would be defined as one conveying a stream for which the 100-year peak discharge is equal to or greater than 500 cubic feet per second (cfs).

Regardless of whether a bridged waterway, culvert, or encroachment is being evaluated, in situations where a FEMA or other officially delineated floodplain is being considered, no increase in the established natural 100-yr. flood level will be permitted either up or downstream. In situations where no FEMA or other officially delineated floodplain exits, it will be acceptable to increase the level of the 100-yr flood event not to exceed one foot up or downstream, provided such increase does not adversely impact adjacent properties, buildings, etc. If an increase in the 100-yr flood level will cause such adverse impact then no increase shall be permitted. The department’s State Hydraulics Engineer must approve exceptions to either of the above criteria.

12.6.2 Necessary Resources

The resources necessary to perform an H&HA usually include, but would not be limited to: topographic maps, aerial photographs, and sufficient roadway plans, profiles, and typical sections to cover the width of the floodplain in the vicinity of the crossing.

In the event a FEMA floodplain (or other officially delineated floodplain) is involved, it will be necessary to have any available flood profiles, maps, and hydraulic model data (i.e. HEC-2 input data, etc.). The Department will secure and provide any necessary hydraulic model data. In the event a bridged waterway is involved, it will be necessary to have a schematic bridge layout or proposed bridge plan, bridge situation survey, and bridge data sheet.

12.6.3 Hydrologic Analysis

If the site is not covered by a FEMA (or other officially delineated) floodplain, it will be necessary to determine a range of design peak discharges to use in the subsequent hydraulic analysis. This would typically be done, for ungaged sites, using empirical procedures such as the:

- "Franklin Snyder" method (See Chapter 6 – Hydrology)
- "Effects of Urban Development on Floods in Northern Virginia" (1968) by Daniel G. Anderson (USGS)
- "Methods for Estimating the Magnitude and Frequency of Peak Discharges of Rural, Unregulated Streams in Virginia" (1994) by James A. Bisese (USGS)
- Areal adjustments of design peak discharges from appropriate gaged sites

See Chapter 6, Hydrology, for detailed information on application and procedures of hydrologic methods.

For gaged sites (or streams having stream gages in the proximity of the site), a Log-Pearson Type III frequency distribution (prorated up or downstream as appropriate) would be the preferred method for determining peak discharges. Regional drainage area versus discharge curves would also be appropriate.

Methods employing total storm runoff (i.e. a hydrograph) consideration, such as the USACE HEC-1 or HEC-HMS or the NRCS' TR-20 and TR-55 models can be employed but shouldn't normally be necessary unless a hydrograph (as opposed to an instantaneous peak) is otherwise needed as in the case of an impoundment structure.

If the site is covered by a FEMA (or other officially delineated) floodplain, the peak discharges employed in making the official floodplain delineation shall be employed. Any exception to this policy must be approved by the VDOT Hydraulics Section.

In situations where the officially delineated study considers only the 10, 50, 100, and 500-year flood events, it will be acceptable to estimate the magnitude of intermediate frequency events such as the 2, 5, and 25-year flood events.

In all cases the 2, 5, 10, 25, 50, 100, and 500-year flood magnitudes will either be determined or obtained (from appropriate sources) and employed and documented in the subsequent hydraulic analysis. In addition, the Ordinary Highwater discharge, usually taken to be the drainage area (in square miles) times 1.1 in units of cubic feet per second, shall be determined and documented for the purposes of applying for certain environmental permits.

12.6.4 Hydraulic Analysis

The Department prefers a three-step procedure for performing the hydraulic analysis using an approved (or preferred) step-backwater computer model as described above.

12.6.4.1 Existing Conditions Model

If a FEMA (or other officially delineated) floodplain is involved, the first step will be to mathematically reproduce the hydraulic model using the same step-backwater computer model on which the original floodplain was predicated. This means if the original study was done using HEC-2, the VDOT study shall be done using HEC-2. If the step-backwater computer model used to perform the original hydraulic analysis is no longer available or is not readily available, one of the approved computer models may be employed as long as it is adjusted to match the official model as closely as practicable. Any exception to this criteria must be approved by the Department’s hydraulics section. The first hydraulic model will be referred to as the "EXISTING CONDITIONS model.

12.6.4.2 Adjusted Existing Conditions Model

The second step would be to add, delete, and/or adjust any natural ground cross sections to the locations necessary to subsequently model any proposed construction. It should be emphasized that any changes made in the "EXISTING CONDITIONS" model should be solely for the purpose of facilitating the modeling of proposed conditions. This should not be taken as the latitude to change the official model for such things as n-values, new cross section geometry, peak discharges, etc. This model then becomes the basis for measurement of any changes that would take place as a result of the proposed construction. This second hydraulic model will be referred to as the "ADJUSTED EXISTING CONDITIONS" model. For this model, it will also be necessary to determine the magnitude of the 2, 5, and 25-year flood events (by interpolation and/or extrapolation of the known magnitudes as necessary) and include them (in this and the "PROPOSED CONDITIONS" model).

12.6.4.3 Proposed Conditions Model

The third hydraulic model will include any and all proposed construction (superimposed on the "ADJUSTED EXISTING CONDITIONS" model) and will be referred to as the "PROPOSED CONDITIONS" model. If the "PROPOSED CONDITIONS" model shows any change (from the "ADJUSTED EXISTING CONDITIONS" model), be it in water surface elevations, velocity of flow, or flow distribution, the proposed construction is to be considered as unacceptable and must be adjusted until no changes occur. It should be noted here that VDOT's policy is to permit no increase in either the 100-year natural floodplain elevation or 100-year floodway elevation, despite FEMA's policy of allowing up to one-foot increase in the natural 100-year floodplain.

12.6.4.4 Procedure When No Existing Conditions Model is Available

In instances where there is no FEMA (or other officially delineated) floodplain involvement, and a bridged waterway is involved, it will be necessary to establish the existing hydraulic conditions through a process that will be referred to as calibration. This calibration is to be accomplished by attempting to reconcile the historical high water elevation secured by or during the survey. The procedure is:

Step 1: Set up a hydraulic model reflecting existing conditions using whatever topographic and terrain data (i.e. bridge situation survey) that's available. Run the model to determine what discharge is required to generate the recorded high water elevation.

Step 2: Using either stream gaging data or hydrographic analysis using actual rainfall data, determine the peak discharge that occurred on the date recorded for the historical high water elevation.

Step 3: If the two discharges match or are close, the existing conditions hydraulic model may be considered calibrated.

If the two discharges don't match (or aren't very close), it will be necessary to either revise the hydraulic model or the hydrologic calculations or both until the two discharges match or are very close. In doing this, extreme care must be taken not to go beyond the realm of reason with either the hydraulic or hydrologic computations. It is always possible that the recorded high water elevation (and/or date) may be in error. If a legitimate calibration can not be achieved, the documentation should fully describe the process leading to the unsuccessful attempt and an explanation offered as to the inability to achieve a calibration.

The hydraulic model used for "calibration" purposes may be either a separate model or may be a part of the "EXISTING CONDITIONS" model. The "PROPOSED CONDITIONS" model should reflect all proposed construction. In this situation, VDOT's policy is to permit up to but not exceeding a one-foot increase in elevation for the 100-year flood event provided the increase doesn't impact upstream development.

12.6.5 Documentation

12.6.5.1 Detailed Hydrologic & Hydraulic Analysis (H & HA) Outline – LD-293D

The first part consists of an outline in which every item shown is to be addressed in its entirety. The outline will be permanently filed as part of the computation assembly. A blank outline is included in Appendix 12B-2. In the case of a lateral floodplain encroachment due to a highway fill embankment, the outline may be adjusted to allow for the fact that a drainage structure is not involved. In such cases it should also be modified or supplemented as necessary to include a tabulation or spreadsheet showing existing and proposed water surface elevations at various locations along the highway embankment. A separate narrative and tabulation may be prepared in lieu of using the outlined in this case.

12.6.5.2 Multipart Letter - LD-293

The second part consists of multi-part letter, officially known as the LD-293 assembly, advising various disciplines within the Department of the results of the hydrologic and hydraulic analysis. Both the outline and the LD-293 assembly are available, upon request, as a series of blank document files in "MICROSOFT WORD" word processing formats. A copy of the entire LD-293 assembly is to be retained with the permanent computation file. Blank copies of the LD-293 assembly are included in Appendices 12B-3 through 12B-5. It should be noted that form LD-293 is only used in the case of a bridge waterway to report the results of the H&HA to the bridge designer. Form LD-293B may additionally be used for a major culvert installation to report pertinent hydraulic design information to the road designer, as a cover letter forwarding form LD-293C (hydraulic commentary necessary for environmental permit applications) to the appropriate District Environmental Manager, and as notification of anticipated hydraulic impacts to the Location & Design Public Involvement Section. A form letter is not available which addresses the hydraulic impacts associated with a lateral encroachment due to a highway fill embankment. However, any necessary changes, modifications, etc. affecting the roadway alignment and/or grade must be coordinated with the road designer.

12.6.5.3 System of Units

The LD-293 assembly will be prepared reflecting exclusively those units employed in the road plans and/or bridge plans whereas, the remainder of the documentation (the actual hydrologic and hydraulic analysis, the outline, etc.) will be left to the discretion of the engineer performing the work. The reason for this is that most of our resources (topographic sheets, mapping, gage records, computational procedures, etc.) are still predicated almost exclusively on English units. It will not be necessary to prepare an LD-293 assembly for H&HA's not associated with a bridged waterway.

12.6.5.4 Level of Precision for Documentation

The following are guidelines governing the level of accuracy to show in the LD-293 assembly:

1. Elevations, etc. obtained from the survey, whether in English or metric, are to be shown exactly as obtained.

2. Elevations, distances, etc. obtained from the plans, whether in English or metric, are to be shown exactly as obtained.

3. The magnitude of peak discharges should be shown to three significant digits in English or metric units. For example, show 12,687 cfs as 12,700 cfs. Show 359.3 cms as 359 cms.

4. Show velocities to the nearest 0.5 (half) fps or 0.1 (tenth) mps.

5. Show calculated water surface elevations to the nearest 0.5 (half) ft. or 0.1 (tenth) m.

6. Show changes in calculated water surface elevations to the nearest 0.5 (half) ft. or 0.1 (tenth) m.

7. Show watershed areas to the nearest sq. mi. or sq. km.

8. There will be occasions where it will be necessary to show a higher level of precision than 0.5 ft. or 0.1 m (e.g. FEMA or other officially delineated floodplains which are

typically shown to the nearest 0.1). If there is any question whatsoever, guidance should be sought from the VDOT Hydraulics Section.

12.6.6 H&HA Submission

When the H&HA has been performed by the consultant for the Department, the following items should be submitted to the Department:

1. The completed H&HA outline, as a document file on diskette and/or as a hard copy printout;

2. The completed LD-293 assembly, as a document file on diskette and as a hard copy printout (including a hardcopy printout or sketch of the anticipated final scoured bed profile for both the design and check flood events) in the event the H&HA was for a bridged waterway.

3. A diskette containing any and all copies of the hydraulic model data on which the H&HA was predicated (i.e. HEC-2, HEC-RAS, etc. data files).

4. Hard copy printouts of all hydraulic model data calculations (i.e. output); Hard copies of full output reports should be printed out for analyses using HEC-2. For HEC-RAS, printouts of Standard Tables 1 and 2 should be submitted. The consultant should contact the Department for guidance when using other models.

5. Copies of any supplemental calculations incidental to the H&HA;

6. Copies of any supplemental documentation not covered in either the H&HA outline or the LD-293 assembly. Any materials and/or resources that have been loaned out by the Department to assist in performing the H&HA such as FEMA studies, etc.

7. If the project crosses or otherwise impacts a FEMA regulatory floodplain or floodway, an excerpt from the FEMA Community Map Panel covering the site should be included.

This information is to be submitted to the VDOT contact person who has been designated as the coordinator for drainage design.

12.7 H&HA for Major Tidal Structures and Bridges

12.7.1 Background

A detailed hydrologic and hydraulic analysis (H&HA) should be performed for all of the Department's new or replacement major tidal drainage structures and/or bridged tidal waterways. It is necessary to do this in order that VDOT construction be in compliance with national (i.e., FHWA, FEMA, etc.), state (DCR, etc.), and municipal (locally delineated floodplains) rules and regulations.

Detailed analysis, as used here, means that for the analysis of bridge crossings of tidal waterways, a three-level analysis approach similar to the approach outlined in HEC-20 and HEC-18 will be employed to assess the impact of the Department's construction and to evaluate the potential for scour around bridge foundations in order to design new and replacement bridges to resist scour. The complexity of the hydraulic analysis increases if the tidal structure or bridge constrict the flow and affect the amplitude of the storm surge (storm tide) so that there is a large change in elevation between the ocean and the estuary or bay, thereby increasing the velocities in the constricted waterway opening.

12.7.2 Necessary Resources

The resources necessary to perform an H&HA of tidal crossings, as for riverine crossings, usually include, but would not be limited to: topographic maps, aerial photographs, maintenance records for the existing bridge, bridge data sheet, bridge situation survey, proposed bridge plans, and sufficient roadway plans, profiles, and typical sections to cover the width of the floodplain in the vicinity of the crossing.

Other resources necessary for tidal analysis are: velocity meter readings, cross section soundings, location of bars and shoals, magnitude and direction of littoral drift, presence of jetties, breakwater, or dredging of navigation channels, and historical tide records. Sources of data include NOAA National Ocean Service, USACE, FEMA, USGS, U.S. Coast Guard, local universities, oceanographic institutions and publications in local libraries. NOAA maintains tidal gage records, bathymetric charts, and other data on line at www.nos.noaa.gov. Also refer to Chapter 13, Shore Protection, for details on working with tidal datums.

12.7.3 Coastal Bridge and Culvert Design Techniques

The hydraulic design guidelines for coastal or tidally influenced waterway bridge openings lags behind similar designs on riverine systems. The complicated phenomenon is difficult to simulate for several reasons, but primarily because tidal simulations often require modeling dynamic (time-varying) conditions. Coastal waterways are subject to storm surges and astronomical tides which play an important role in hydraulic behavior. The collection of adequate data to represent the actual condition also adds to the complexity of the problem. Data such as flows and storm surge description may be difficult to estimate. For small bridges, complex modeling may not be cost effective since the cost of the study may exceed the cost of the bridge.

Presently there is no standard procedure for the design of tidally influenced waterways. In many cases, the bridge hydraulic opening is designed to extend across the normal open water section. This may be an appropriate design from an economic standpoint; since the total cost of a larger bridge may approximates the cost of a smaller bridge considering approach embankments and abutment protection measures. This design is also desirable from an environmental perspective since it results in minimal environmental impacts. In most designs, the extent of detail in the analysis must be commensurate with the project size or potential environmental impacts. However, analytical evaluation of the opening is often required and is necessary when a full crossing cannot be considered or when the existing exhibits hydraulic problems. The complexity of these analyses lends themselves to computer modeling.

Because of the lack of standard procedures for the design of costal waterways, research is being conducted on this matter. A FHWA pooled fund study coordinated by the South Carolina Department of Transportation has developed recommendations for modeling of tidally influenced bridges. In addition to design guidelines, technical research needs to be conducted to better understand the hydraulics in tidally influenced waterways.

Research is needed in the following areas: sediment transport and scour processes, coastal and tidal marsh ecosystems, environmental impacts and the development of comprehensive coastal hydraulics models.

The FHWA, in their publication Evaluating Scour at Bridges (HEC-18), presents procedures for performing hydraulic analysis of tidal waterways. HEC-18 procedures are recommended until better or more standardized methods are developed. The FHWA intends to publish HEC-25 from results of the tidal pooled fund research project, which will provide design guidance for tidal hydraulic modeling of bridges.

12.7.4 Computer Modeling

Existing models cover a wide range from simple analytical solutions to heavy computer intensive numerical models. Some models deal only with flows through inlets, while others describe general one-dimensional or two-dimensional flow in coastal areas. A higher level includes hurricane or other storm behavior and predicts the resulting storm surges.

One-dimensional steady state models are the most commonly used models because they demand less data and computer time than the more comprehensive models. Most analyses for tidal streams are conducted with steady state models where the tidal effects are not simulated. This may be an adequate approach if the crossing is located inland from the mouth where the tidal effects are insignificant. Computer modeling for steady state hydraulics is generally preformed with the Corps of Engineers HEC-RAS (or HEC-2) or the U. S. G. S. FHWA WSPRO (HY-7).

In the event that either tidal fluctuations or tidal storage are significant, simulation of the unsteady hydraulics is more appropriate. Unsteady flow computer models were evaluated under a FHWA pooled fund research project administered by the South Carolina Department of Transportation (SCDOT). The purpose of this study was to identify the most promising unsteady tidal hydraulic models for use in scour analyses. The study identified UNET, FESWMS-2D, and RMA-2V as being the most applicable for scour analysis. The research funded by the FHWA pooled fund project is being continued to enhance and adapt the selected models so that they are better suited to the assessment of scour at tidal bridges.

The pooled fund research project also resulted in guidance on the appropriate methodology to use based on the geomorphic characteristics of the tidal waterway. Where complicated hydraulics exists, for instance as in wide floodplains with interlaced channels or where flow is not generally in one direction, a one-dimensional model may not represent adequately the flow phenomena and a two-dimensional model is more appropriate. Two-dimensional models in common use to model tidal flow hydraulics are FESWMS-2DH and RMA-2V. FESWMS-2DH, a finite element model was prepared for the FHWA by David C. Froehlich and includes highway specific design functions such as pier scour, weirs, and culverts. RMA-2V, also a finite element model, was developed by the US Army Corps of Engineers. FESWMS-2DH and RMA-2V can also incorporate surface stress due to wind. These models require considerable time for model calibration. Thus, they do not lend themselves for analysis of smaller structure sites.

The US Army Corps of Engineers' UNET model is widely accepted in situations where the more complicated two-dimensional models are not warranted or for use in making preliminary evaluations. UNET is a one-dimensional, unsteady flow model. The Corps of Engineers has now modified HEC-RAS to incorporate dynamic routing features similar to UNET.

Alternatively, either a procedure by Neill for unconstricted waterways, or an orifice equation for constricted tidal inlets can be used to evaluate the hydraulic conditions at bridges influenced by tidal flows. The procedure developed by Neill can be used for tidal inlets that are unconstricted. This method, which assumes that the water surface in the tidal prism is level, and the basin has vertical sides, can be used for locations where the boundaries of the tidal prism can be well defined and where only small portions of the inundated overbank areas are heavily vegetated or consists of mud flats. The friction loss resulting from thick vegetation tends to attenuate tide levels thereby violating the assumption of a level tidal prism. The discharges and velocities may be over estimated using this procedure. In some more complex cases a simple tidal routing technique (TIDEROUT) or a simple UNET or other 1-dimensional model (HEC-RAS) can be substituted with a similar level of effort. UNET includes storage areas that are assumed to fill as level pools.

12.7.5 Hydrologic Analysis

The flow associated with a tidal bridge generally consists of a combination of riverine and tidal flows. VDOT's Tidal Bridge Scour Data & Worksheet (Appendix 12C-2) will be used to calculate both the tidal and riverine flow components for tidal crossings. This worksheet utilizes a "VDOT only" modification of Neil's method for calculating tidal flow and USGS Regression equations for riverine flow. The data required to complete this worksheet is generally available from field data and limited research. A discussion which addresses the information needed to complete the Tidal Bridge Scour Data & Worksheet follows.

12.7.5.1 Bridge Location

- Bridge Number, Route, County, Length and River Crossing can be obtained from bridge plans and inspection reports.

- Tidal Bridge Category:

    - Islands: Passages between islands or between an island and the mainland where a route to the open sea exists in both directions.
    - Semi-Enclosed Bays & Inlets : Inlets between the open sea and an enclosed lagoon or bay where most of the discharge results from tidal flows.
    - Estuaries : River estuaries where the discharge consists of river flow as well as tidal flow.

12.7.5.2 Channel Cross Section

Channel cross section data may be obtained from several sources such as VDOT Central or District offices, bridge plans and/or bridge inspection reports.

12.7.5.3 Drainage Area Characterists

Drainage area characteristics are required for estimating peak flood discharges using the USGS regression equations for Virginia. (See FHWA Tidal Pooled Fund Study "Tidal Hydraulic Modeling For Bridges" Section 3.4 for guidance in combining storm surge and upland runoff.) Note: copies of this publication are available on request from the department's Hydraulics Section (as a ".PDF" file) until such time as the FHWA releases their upcoming HEC-25 publication.

- Drainage area estimated from USGS topographic maps (1:24000), NOAA Navigation maps or similar topographic maps from other sources such as county topographic maps.

- Percentage of forested area, main channel slope, average basin elevation and main channel length can be estimated from USGS topographic maps, street maps or other types of topographic maps.

12.7.5.4 Storm Tides

- The surface area of the tidal basin is required for estimating tidal flows. From USGS topographic maps or NOAA navigation maps, the surface area of the tidal basin can be obtained by planimetering several different contour line levels, and then developing a graph of the surface area vs elevation. Since the maximum tidal flow normally occurs at midtide, the preferred method of analysis is to determine the surface area of the tidal basin at this elevation. The surface area of the tidal basin at the midtide elevation can be determined from the graph by interpolation.

- The 10, 50, 100 and 500-year storm tides can be obtained from the maps and figures of the coastal regions of Virginia located in the appendix. The maps and table of storm tide description have been compiled and developed from existing FEMA Flood Insurance Study reports, NOAA tidal records, US Army Corps' tidal analysis and Ho's Hurricane Tide Frequencies Along The Atlantic Coast.

- Tidal flow is the product of the surface area and the rate change of the tidal height and may be expressed by the following equation:

$$Q = 24312\, A_S \frac{H}{T}$$

where Q = Tidal flow, in cfs

A_S = Surface area of the tidal area upstream from the bridge at the midtide elevation, in sq. mi.

H = Tidal height, between low tide and high tide, in ft.

T = Period of the storm tide, in hours. (See Note 1)

Note 1: Obtain both H and T from the maps and table in Appendix 12C-3 and 12C-4.

12.7.5.5 Flow Velocity

The flow velocities should be calculated for the flow conditions that may result in higher velocities. These conditions include: (a) the peak riverine flow with a low downstream water level and (b) the combined tidal flow and the flood peak flow, with the water level at the midtide elevation.

There is an additional condition, (c), that needs to be investigated for tidal bridges located on estuaries some distance upstream from a bay or ocean. The flow depth at bridges in such cases is less likely to be controlled by the tidal elevation in the bay and more likely to be controlled by the channel slope, boundary roughness and channel geometry. Using the low sea level to calculate the flow velocity for such bridges may result in an unreasonably high velocity due to underestimation of the flow depth and cross-sectional area. Manning's equation should be used to estimate the flow velocity in such cases. Engineering judgment should be applied when estimating the flow conditions and appropriate flow depth to be used in calculating the velocity of flow.

Particular attention needs to be directed at determining the appropriate combination of riverine and tidal flows for use in estimating worst case scour conditions.

The flow velocities estimated by the above methods represent an approximate value for use in the screening process. A detailed H&H Study is required if a more accurate estimation of velocities is desired.

The analysis of the flow velocity in this Worksheet assumes steady flow even though tidal flow is an unsteady flow phenomenon. The resulting velocity will generally be slightly different from a velocity calculated on the basis of unsteady flow. Since the rate of the vertical motion of storm tides is on the order of only three to eight thousandths of a foot per second, the velocity estimates obtained from the method discussed above should be reasonable for locations in unconstricted bays and estuaries where velocities are on the order of 3 feet per second or less.

Maps and figures of the coastal regions of Virginia that describe the tidal storm surge periods and predicted water surface elevations for the 10, 50, 100 and 500-year storms are shown in Appendix 12C-3 and 12C-4

12.7.6 Hydraulic Analysis

VDOT's Tidal Bridge Scour Data and Worksheet (Appendix 12C-2) will be used during the Level 1 Analysis (see HEC-18) in order to estimate the maximum flow velocities through the tidal bridge during the passage of a storm tide. This estimate should be considered as a first approximation for use in judging whether the proposed tidal bridge requires a more detailed H&HA.

Normally, Neill's method of analysis should provide an acceptable degree of accuracy for tidal inlets and estuaries that are not significantly constricted and where flow velocities are 3 feet per second or less.

Where the waterway is constricted and estimated flow velocities exceed 3 feet per second, it may be appropriate to route the storm tide through the structure for purposes of obtaining a more accurate estimate of storm tide velocities. The TIDEROUT computer program is recommended for use when making calculations involving tide routing through a structure. TIDEROUT is a BASIC computer program developed by Mr. Raja Veeranachaneni, MD SHA. A copy of the TIDEROUT program is available on request from the department's Hydraulics Section. If the estimated flow velocity from the Tidal Worksheet is 7 feet per second or greater, routing of the storm tide through the structure should definitely be considered.

Where the simplified methods yield overly conservative results, the use of routing techniques or unsteady flow computer models (Level 2) will provide more realistic predictions of hydraulic properties and scour.

For certain types of open tidal waterway crossings, worst-case scour conditions may be caused by the action of the wind. In other cases, such as passages between islands or an island and the mainland, the worst-case condition may represent a combination of tidal flow and wind forces. These specialized cases require careful analysis and should be studied by engineers with a background in tidal hydraulics.

Electronic spreadsheets are available which assist in the generation of storm surge hydrographs for use in defining downstream boundary conditions during hydrodynamic modeling. These spreadsheets are available on request from the department's Hydraulics Section. Maps showing the locations of ADCIRC stations along the Virginia coast where storm surge hydrographs are available are included in Appendix C of the "Tidal Hydraulic Modeling for Bridges" publication. Also available are spreadsheets that assist in the computation of time dependent scour and wave heights for tidal sites.

The FHWA Tidal Pooled Fund Study's "Tidal Hydraulic Modeling for Bridges" publication presents guidance on the appropriate methodology to use based on the geomorphic characteristics of the tidal waterway. As noted above, this publication will be available on request from the department's Hydraulics Section until such time as the FHWA's HEC-25 publication becomes available.

VDOT prefers a three-step procedure for performing the hydraulic analysis as described in the instructions for an H&HA of a riverine site (see Section 12.6.4).

12.7.7 Documentation

The documentation for a tidal H&HA will be the same as required for a riverine H&HA (see Section 12.6.5).

12.7.8 H&HA Submission

When the H&HA has been performed for a tidal site by the consultant for the Department, the level of documentation to submit to the Department should be the same as required for a riverine site (see Section 12.6.6).

12.8 Riprap for Protection of Bridge Abutments and Piers

Riprap is frequently used for protection of the earthen fill slopes employed in spill-through abutments. In such situations, it serves the two-fold purpose of protecting the underlying shelf abutment and piers against runoff coming from the approach roadway and bridge superstructure as well as from scouring due to impinging flow resulting from floodwaters. Riprap can also be used around solid, gravity abutments to protect against scour. Riprap is considered an acceptable scour countermeasure for protection of bridge abutments. The use of riprap at bridge piers, on the other hand, is not acceptable for use in new construction and is considered only as a temporary countermeasure in the case of rehabilitation. The Department employs the riprap design procedures presented in the FHWA publication “Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance” (HEC-23). The Department has developed a computer program entitled “BRRIPRAP” which performs all necessary riprap design calculations in accordance with HEC-23. It is available upon request.

12.9 Removal of Existing Bridge and Approach Embankments

When an existing bridge is to be removed, the bid item for removal of the existing bridge will include the entire superstructure and all portions of the substructure, such as abutments, wing walls and piers, pilings and riprap or slope protection. No portion of the approach roadway embankment is to be included in this bid item.

The limits of the approach roadway embankment to be removed will be furnished to the road designer by the Hydraulics Section and shown on Form LD-293B (Appendix 12B-3). These limits are to be shown on the road grading plans along with the following note:

> "The existing approach roadway embankments will be removed between Station _______________ and Station ____________________ ______________________ and will be included in the quantity for regular excavation."

When a portion of existing approach embankments are removed for flood control, the remaining approach embankment surface should be graded on an approximate 0.5 percent slope toward the waterway in such a manner as not to impound any water on the surface after the flood waters have receded or after normal rainfall as shown in Figure 12-1.

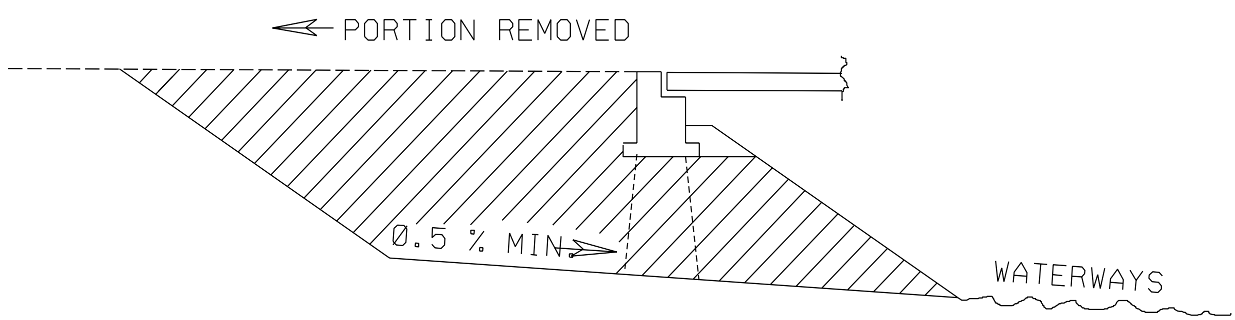

Figure 12-1. Removal of Approach Embankment

The determination of quantities for the removal of approach embankment should be set up on a cubic yard basis and included in the plan quantity for regular excavation. The limits for computing the quantity is a vertical plane through the joint between the approach pavement and the end of the bridge as shown in Figure 12-2.

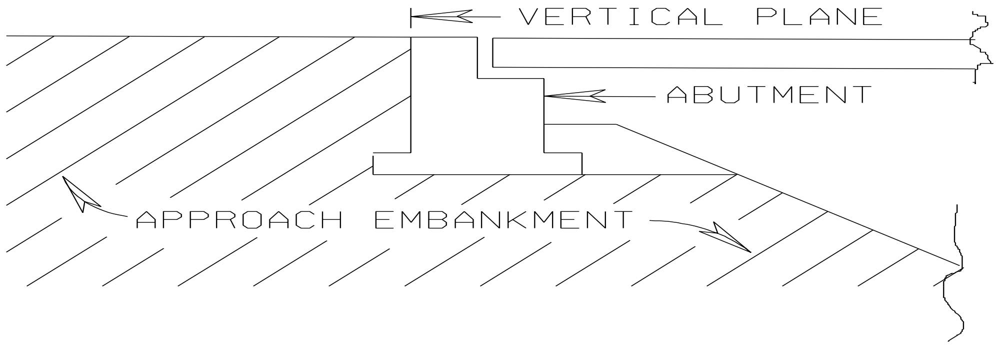

Figure 12-2. Quantifying Removal of Approach Embankment

The road designer will request such additional survey information as is necessary to delineate and estimate the quantities of the embankment to be removed.

The District Engineer must be afforded an opportunity to review and comment on the embankment removal proposal prior to completing the plans.

12.10 Temporary Construction Causeway Design

12.10.1 Background

The need to provide a construction access facility that will not have a significant impact on normal flow conditions has been identified by the Environmental Division.

12.10.2 Causeway Design

12.10.2.1 Design Objectives

- Provide a design that is reasonably convenient, economical, and logistically feasible for the contractor to build and remove.
- Provide a design that will not be subject to failure due to normal stream flow conditions. This should consider in-stream obstructions such as piers or islands that could direct high velocity jets at points along the causeway.
- Provide a design that will not cause a significant increase in the Ordinary High Water stage, will not significantly increase the velocity of flow through the causeway opening(s) for that flood, will not significantly alter flow distribution, and will not concentrate flow on the piers and foundations that would subject them to forces for which they were not designed. The causeway’s influence on flood flow elevations should be checked in the event that it does not wash out during a significant flood.

12.10.2.2 Plans

The temporary construction causeway should be designed as a rock prism. The design details and required notes should be shown on the typical section sheets (series 2 plan sheets) in the project plans or on a separate detail sheet for "Bridge Only" projects. A note, "Temporary Construction Causeway Required, See Sheet ______ of _____ for details" should be shown on the road plan sheet where the causeway appears. The design details and required notes for the "Temporary Construction Causeway" will be shown on the front sheet of Bridge plans for "Bridge Only" projects. A typical causeway design detail is shown in Figure 12-3.

The pay item(s) for causeways will be included with the road plans. For "Bridge Only" projects, the causeway pay item(s) will be included in the bridge plans.

The contractor should bid the rock causeway as shown on the plans. The contractor may elect to revise the design or substitute another design after being awarded the contract. If so, he should submit a revised design including necessary sketches and notes for review by the district construction, hydraulic and environmental personnel. The Department should obtain a revised environmental permit if necessary, for the contractor's revised design.

The material used in construction of the causeway should be Standard Class I Dry Riprap.

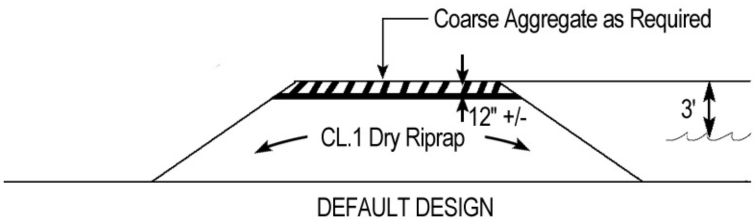

Figure 12-3. Temporary Construction Causeway Design

Show “Ordinary High Water” as the level that the top of the causeway is 3’ over.

12.10.2.3 General Notes

1. The basis of payment for the temporary causeway will be lump sum, which price should include all labor, equipment, materials and incidentals needed for construction, maintenance, removal and disposal of the causeway.

2. The Project Engineer may make minor adjustment in the location of the causeway provided that the adjustment does not change the design of the causeway.

12.10.3 Design Procedure

Step 1 *Set the alignment of the causeway to facilitate construction activity. Set the finished grade 3'± above the Ordinary High Water elevation. Set the side slope angle at the natural angle of repose (approx. 1½:1).*

Step 2: *Determine the required waterway opening(s) and the resulting hydraulic performance using appropriate hydraulic design techniques. It is recommended that pipes be used whose diameter (or rise as appropriate) is 2-feet less than the causeway is high. In other words, if the causeway is 6-feet high, then use 48-inch pipe(s).*

12.11 Daily Stream Flow Information

12.11.1 Background

In instances where a VDOT project crosses and/or is in the floodplain of a major waterway, it will be necessary to provide the contractor (or others as appropriate) with a means of determining which times of year would be most suitable for in-stream work (i.e. periods of normally extended low flows) as well as those times when larger or flood flows can be expected. When such information is available, the best source is usually stream gaging information from gage stations which provide daily flow data.

12.11.2 Development of a Composite Stream Flow Hydrograph

To provide the needed information, it will be necessary to plot approximately 10 consecutive Water Years of daily stream flow hydrographs, superimposed one upon the other, for a given stream gage. The department has developed computer software for this purpose. A "Water Year" starts October 1st of the previous year and goes through September 30th of the year under consideration. It is therefore desirable, when generating these plots, to have them start with October of the first Water Year under consideration and end in September of the last (usually 10th) Water Year. It is also desirable to use the most recent 10 consecutive years for which uninterrupted daily flow data is available for the stream gage being employed. Ideally, a stream gage would be used which is located relatively near (either up or downstream) of the project. It may not always be possible or feasible to utilize a stream gage located on the same stream and/or in very close proximity to the project. In such instances it will be acceptable to utilize a gage on another nearby stream, which in the judgment of the hydraulic engineer, can provide more appropriate stream flow information. The most important objective is to provide an indication of those times of year when sustained periods of low flow or high flows can be expected.

After selecting a stream gage, it is highly recommended that the gaging records be reviewed prior to utilizing the plotting software to insure that the gage is of the recording type (i.e. that daily stream flow records are available) and to determine the most recent 10 consecutive years for which uninterrupted data is available. The usual references for this information are the U.S. Geological Survey's annual publications entitled WATER RESOURCES DATA VIRGINIA, VOLUME 1, SURFACE-WATER-DISCHARGE AND SURFACE-WATER-QUALITY RECORDS (for each Water Year under consideration) and their Internet web site which is entitled "NWISWeb Data for Virginia", the "URL" for which is http://waterdata.usgs.gov\va\nwis\.

The software necessary to generate these plots – COMPOSITE HYDROGRAPH – is located on the Central Office Location & Design Division's 0501COLND file server. Access to the software will normally be granted to any VDOT personnel involved in drainage design and is an integral part of the department's "Hydraulic

Engr. Package" of software. Consultants needing these hydrographs must currently request them from the designated drainage design coordinator. Permission for access to the software must be requested of the Central Office Location & Design Division's AES Manager but shall not be granted without the approval of the State Hydraulic Engineer.

The software's database contains daily stream flow records for all recording stream gages in the state of Virginia. This data will, for gages currently in operation, be available up through the most recent Water Year for which data has been published. The software can, at the user's option, generate the hydrograph either as a ".BMP" file saved to disk or as a letter size hard-copy printout. The ".BMP" file should be made available to the Road Designer so he can import it into MicroStation and convert into a plan sheet for inclusion in the plan assembly. Probably the quickest and most convenient way to do this will be to attach the file to the cover e-memo used to transmit the usual "LD-293B" memorandum (in the case of a bridged waterway) to the Road Designer. If no bridged waterway is involved, as would be the case when the floodplain involvement is by virtue of a major culvert or roadway encroachment, the file should be generated and transmitted at the conclusion of the hydrologic & hydraulic analysis. An example daily stream flow composite hydrograph plot is included in the Appendix 12-E1, "Example Daily Stream Flow Information".

12.12 References

The Federal Highway Administration Hydraulic Engineering Circular No. 17, "The Design of Encroachments on Flood Plains Using Risk Analysis" – October 1980.

Survey Instructions Manual – Virginia Department of Highways & Transportation.

Hydraulics of Bridge Waterways, Federal Highway Administration – 1970.

HEC-2 Water Surface Profiles – U. S. Army Corps of Engineers.

Highways in the River Environment – Hydraulic and Environmental Design Considerations – Federal Highway Administration – 1975.