PHASE III

for the

POOLED FUND STUDY

DEVELOPMENT OF
HYDRAULIC COMPUTER MODELS
TO ANALYZE TIDAL AND COASTAL STREAM
HYDRAULIC CONDITIONS AT
HIGHWAY STRUCTURES

Ayres Associates, Inc.
P.O. Box 270460
Fort Collins, Colorado 80527
(970) 223-5556, FAX (970) 223-5578

Edge & Associates, Inc.
4911 Bay Oaks Court
College Station, Texas 77845

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Submitted to

South Carolina Department of Transportation
P.O. Box 191
Columbia, South Carolina 29202

AYRES ASSOCIATES

P.O. Box 270460
Fort Collins, Colorado 80527
(970) 223-5556, FAX (970) 223-5578

and

Edge & Associates, Inc.

4911 Bay Oaks Court
College Station, Texas 77845

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Technical Advisory Panel: Charles Smoak and Wayne Corley, South Carolina DOT; David Henderson and Max Price, North Carolina DOT; Sam Teal, Georgia DOT; Jack Harrell, Virginia DOT; Michael Hogan, Connecticut DOT; Michael Wight, Connecticut DOT; and Larry Arneson, FHWA.

16. Abstract

Highway structures in tidal waterways are subjected to foundation scour and stream instability resulting from dynamic flow conditions caused by tides, currents, storm surges and upland runoff. The phases of this study have focused on (1) evaluating computer models to determine which models were well suited for simulating unsteady tidal flow conditions at bridges (2) developing recommendations on selecting boundary conditions for computer simulations, (3) developing manuals for tidal bridge hydraulics and (4) providing training and technical support to Pooled Fund member states. In the current phase of this study additional research has been conducted, the manuals have been updated and additional training has been conducted. The research included (1) relating hurricane category to frequency of occurrence along the east coast, (2) developing a method of predicting the rate of contraction scour at a bridge, (3) developing recommendations on including upland runoff in hurricane storm surge analyses, (4) developing an alternative storm surge stage hydrograph (5) providing guidance on computing wave heights at bridge openings and (6) providing guidance on incorporating wind into tidal simulations.

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1. INTRODUCTION

The objective of the Pooled Fund Study is to advance the field of hydraulic and scour analyses of bridges over tidal waterways. This report presents the progress of Phase III of the Pooled Fund Study. Research and training have been the primary focus in this and the previous two phases of the project. The research has addressed (1) methods and computer models that are best suited for determining hydraulic conditions for unsteady tide and hurricane surge conditions, (2) model boundary conditions including tides, storm surges, upland runoff and wind stresses, (3) hurricane properties, (4) scour conditions for tidal bridges and (5) wave height prediction at bridges. The training has included (1) manual development, (2) short course presentation, and (3) technical support.

1.1 Summary of Previous Phases

1.1.1 Phase I Activities and Results

Phase I focused on three tasks: (1) compile a database of literature on tidal processes and computer models, (2) evaluate which computer models are well suited for tidal hydrodynamic investigations, and (3) evaluate sources and methodologies for determining ocean tide and storm surge characteristics. The results were included in the Phase I Final Report (Richardson et al. 1994).

For Task 1, the Research Team compiled 622 citations into a database on tidal processes and analytical procedures. All the citations were assigned keywords and many included abstracts.

For Task 2, 21 models were reviewed to determine their applicability to tidal hydraulic and scour studies. One-, two-, and three-dimensional steady and dynamic models were included. All the models included hydraulic computations. Some models included sediment and/or contaminant transport directly or could be linked with transport models. Necessary and desirable model characteristics were developed by the Research Team and the Technical Advisory Panel to identify which models were well suited for hydraulic analysis of tidally affected highway encroachments. Based on the necessary and desirable characteristics, four models were recommended to the Pooled Fund States. These were the DYNLET, UNET, RMA2V, and FESWMS-2DH models.

For Task 3, the Research Team reviewed available data on surges along the Atlantic and gulf coasts. FEMA and NOAA publish peak storm surge elevations based on frequency of occurrence or hurricane severity. FEMA and NOAA publications do not include the hurricane stage hydrographs so alternative methods were reviewed to develop stage hydrographs based on surge peak elevation and other readily available hurricane characteristics. A very useful methodology for computing storm surge hydrographs has been developed by the Corps of Engineers (USACE). This methodology uses peak storm stage (from FEMA, NOAA, and other sources) and storm properties (storm forward speed and radius of maximum winds) to develop storm surge hydrographs. Appropriate storm properties are contained in a USACE report. Another source of storm surge data is a USACE storm surge atlas. This atlas contains storm surge hydrographs from historic hurricanes. This resource is not as easily used as the USACE methodology, but can be used if a storm surge contained in the atlas is similar to a desired storm surge or if a simulation of an actual event is desired.

1.1.2 Phase II Activities and Results

Phase II included (1) enhancing the selected computer models, (2) testing and developing case studies for the selected models, (3) developing methods for computing storm surge hydrographs, (4) writing a Users Manual on tidal hydraulic modeling for bridge applications to supplement the existing model users manuals and (5) providing training and technical support to the Pooled Fund States. The results were included in the Phase II Final Report (Zevenbergen et al. 1997a).

The primary enhancement to UNET was the inclusion of metric computation capabilities. The work was performed by the model developer as a subcontract to this project. The work was performed on U.S. Army Corps of Engineers Version 3.0 of UNET. This version is available from the Research Team for metric simulations.

FESWMS enhancements were performed as part of the ongoing software development supported by FHWA. The primary advance in the use of FESWMS has been the FHWA supported development of the graphical user interface called Surface Water Modeling System (SMS, BYU 1998). This user interface is used for model network development, run control, variable assignment and output analysis.

The Research Team developed utility programs for scour calculations using the output from UNET and FESWMS. The Research Team has also developed an interim procedure to analyze submerged deck bridge hydraulics (pressure flow) because of difficulties with the FESWMS pressure flow computational routine.

Another contribution of this study is the development of methods for predicting storm surge hydrographs. A synthetic hydrograph can be computed from the peak storm surge elevation and the hurricane characteristics of radius of maximum winds and forward speed. Guidance is provided on selecting the appropriate values of the hurricane characteristics. Peak storm surge elevations for 50-, 100-, and 500-year storm surges were also developed as part of this study for numerous locations along the east and gulf coasts and within Chesapeake Bay.

The primary product of Phase II was a Users Manual for Tidal Hydraulic Modeling for Bridges (Zevenbergen et al. 1997b). The Users Manual is intended to supplement the UNET and FESWMS users manuals. The Users Manual contains recommendations on model selection including when the simplified methods are applicable. The storm surge hydrology methods and procedures are also contained in the Users Manual along with chapters on the use of UNET and FESWMS for tidal applications. The Users Manual contains several appendices including charts of hurricane properties, predictions of storm surge elevations, maps of the locations of the storm surge predictions, UNET and FESWMS case studies, and the interim methodology for bridge pressure flow computations with FESWMS.

1.2 Phase III Objectives

The objectives of Phase III fall into three major activities. These are (1) continued research, (2) testing updated models, (3) revisions to the training and manuals.

The additional research topics included: (1) Methods were recommended for estimating wave heights at bridges. (2) Previous guidance on timing the storm surge with mid-rising tide was reviewed. (3) An alternative surge hydrograph was developed that better represents the actual hydrograph falling limb. (4) Hurricane peak stage frequency was compared with hurricane category along the coastlines of the Pooled Fund States. (5) Guidance was developed on when to incorporate upland runoff into storm surge simulations. (6) A methodology was developed for computing the rate of contraction scour during the short duration storm surge. (7) Guidance was developed on incorporating wind into tidal simulations.

Continued development of the UNET and FESWMS models has required additional model testing. UNET Version 3.2 became available at the end of Phase II. FESWMS Version 2, which became available during Phase II, could not be used in tidal simulations that required element wetting and drying. FESWMS Version 3 became available during Phase III. It incorporates the correction for the wetting and drying problem and several other features have been added. These models were tested to assure their continued suitability for tidal hydraulics.

Late in Phase III, UNET Version 4.0 were released. The primary update to UNET is incorporation of SI computations into the version distributed and supported by U.S. Army Corps of Engineers. Of greater significance for the Pooled Fund Project is the release of HEC-RAS 3.0. This release includes UNET unsteady flow computations, which combined with the existing HEC-RAS graphical user interface makes this program an excellent program for 1-dimensional tidal modeling.

2. PHASE III RESEARCH TOPICS

2.1 Alternative Storm Surge Hydrograph

In Phase II, a theoretical storm surge hydrograph shape was recommended as the ocean boundary condition for computer simulations. The equation (reported in Cialone et al. 1993) is based on the atmospheric pressure distribution, hurricane forward speed, hurricane radius of maximum wind, and maximum surge level. Since the pressure is assumed to vary radially from the hurricane eye, the resulting surge hydrograph is symmetrical before and after the time of the peak surge elevation. The hydrograph is for the ocean boundary and is shown in Figure 2.1. The hydrograph is for the surge only and is superimposed on the daily tide for application in a computer model. The equation is:

$$S_t(t) = S_P\left(1 - e^{-\left|\frac{D}{(t-t_0)}\right|}\right) \tag{2.1}$$

where:

S_t	=	storm surge elevation at time t, ft (m)
S_p	=	peak storm surge elevation at landfall time (t_0), ft (m)
D	=	R/f = half the storm duration, hr
R	=	radius of maximum wind, nautical miles (km)
F	=	storm forward speed, knots (km/hr)
t	=	time (hr)
t_0	=	time of landfall and peak surge (hr)

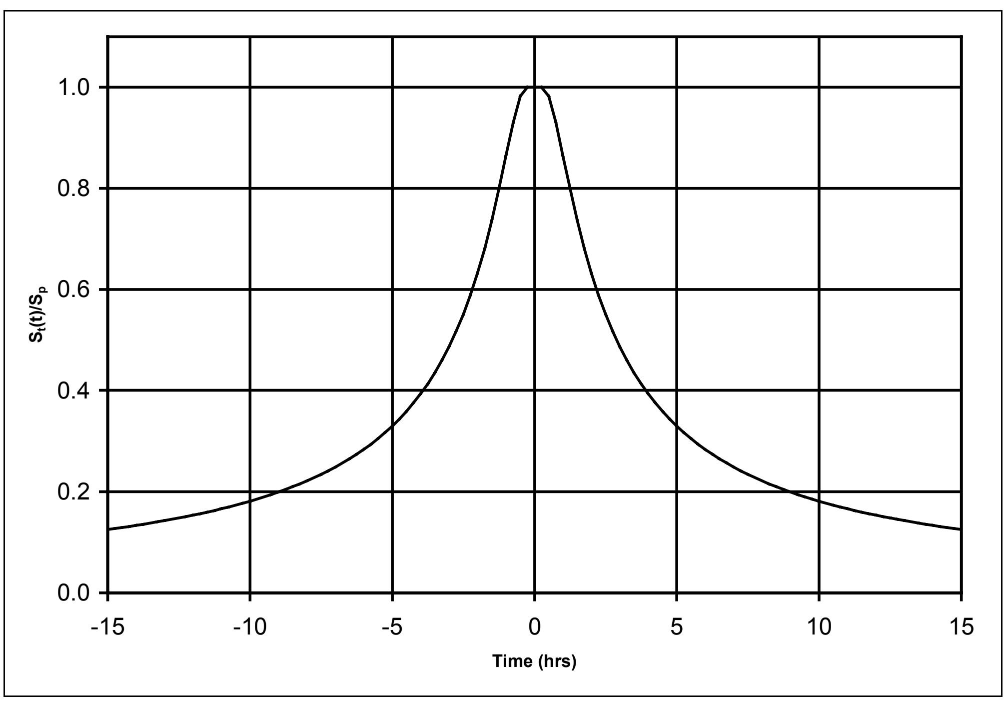

Figure 2.1. Synthetic surge hydrograph.

In Phase II, the recommended value of the half-duration (R/f) was the ratio of mean radius of maximum wind to mean forward speed. This value is typically around two hours for most hurricanes. The values of R and F were selected because, for a given surge elevation, longer or shorter durations are possible, but are less likely. Therefore, flatter (D > 2 hrs) or steeper (D < 2 hrs) surge elevation hydrographs are possible, but are also less likely. For the hydraulics at a bridge, depending on the type of tidal waterway and the location of the bridge from the coastline, more severe hydraulic conditions could result from either steeper or flatter surge hydrographs. Therefore, as a standard approach for any tidal bridge, it was recommended that the surge hydrograph be developed using the average forward speed and average radius of maximum wind for the location of interest.

To determine whether the synthetic storm surge hydrograph should be modified, surge hydrographs were reviewed from ACDIRC stations along the Atlantic coast to see if there was any consistency in the hydrograph shape. The two largest events were selected at nine ADCIRC stations equally spaced along the coast from Miami, Florida to Machias, Maine. If a particular hurricane would have been included more than twice in the overall data set (by occurring at more than two stations), then the third largest event was selected in its place. Based on this criteria, use of the third largest event was required at only two of the nine stations. **Figure 2.2** shows all the surge hydrographs normalized by dividing the surge hydrograph by the peak surge for that event and location. This figure shows that surge hydrographs have a tendency for steep falling limbs and frequently drop below the mean sea level reference datum. Also shown in Figure 2.1 is the mean and +/- one standard deviation of these hydrographs. The mean and standard deviation was computed at 15-minute time intervals for storm events. The mean hydrograph illustrates that the falling limb is, on average, twice as steep as the rising limb and falls slightly negative.

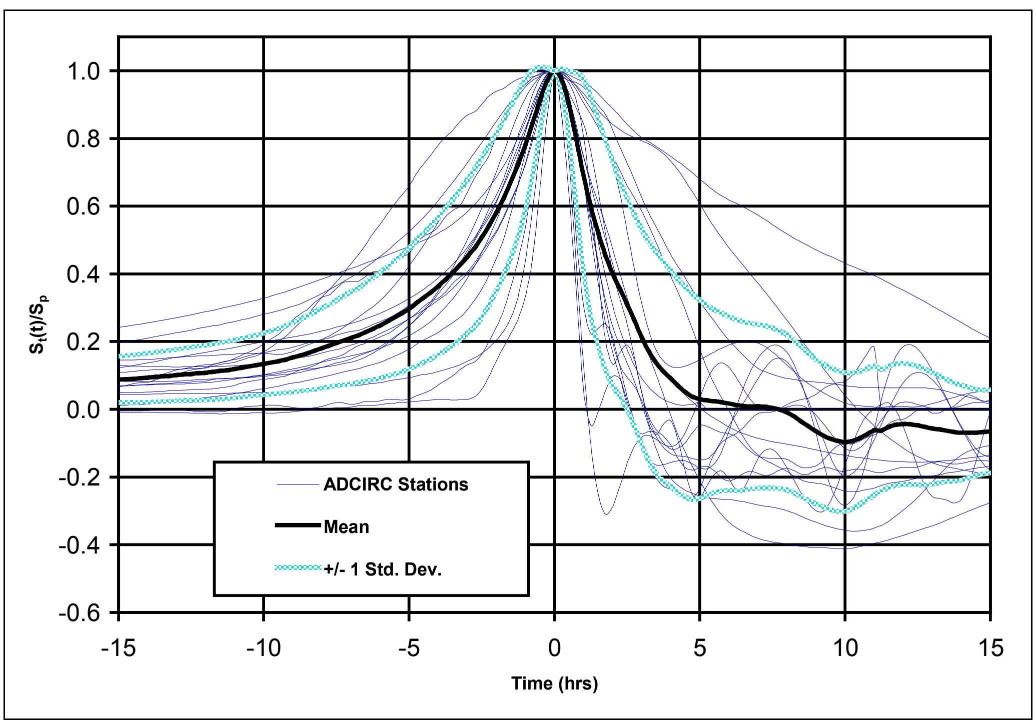

Figure 2.2. Normalized ADCIRC surge hydrographs.

Figure 2.3 shows the mean and +/- one standard deviation of the hurricane surges from Figure 2.1. Also shown are the synthetic hydrograph (Equation 2.1) and an alternative synthetic hydrograph developed to follow the general trend of the mean ADCIRC hydrographs. It should be noted that the synthetic hydrograph matches the mean ADCIRC hydrograph very well along the rising limb, but is much flatter than the mean ADCIRC hydrograph on the falling limb. On the falling limb, the synthetic hydrograph follows the plus one standard deviation indicating that only around 16 percent of the ADCIRC results were at this level or flatter. The shape of the synthetic surge hydrograph is based on the theoretical atmospheric pressure distribution, storm radius of maximum wind and forward speed (Cailone et al. 1993). If pressure alone were the cause of storm surge, then this shape would be expected for both the rising and falling limbs. Depending on the location along the coastline and the storm track, wind can reinforce or counteract the pressure surge. Although wind effects play a significant role in causing the surge, it appears that wind has little effect on the shape of the rising limb but does have a significant effect on the shape of the falling limb. The equation for the alternative synthetic surge hydrograph was developed by adding a term to Equation 2.1. The term was developed to match the mean falling limb well, but has no theoretical basis. The alternative surge hydrograph equation is

$$S_t(t) = S_P\left(1 - e^{-\left|\frac{D}{(t-t_0)}\right|} - 0.14(t-t_0)e^{-0.18(t-t_o)}\right) \qquad \text{for } t > t_0 \qquad (2.2)$$

where all the terms are as defined for Equation 2.1. This equation applies only to the falling limb ($t > t_0$) and equation 2.1 applies to the rising limb ($t < t_0$). At the peak ($t = t_0$) both equations are not defined but at this point $S_t = S_P$. A practical way of performing the computation in a spreadsheet is to add a very small number to the time variable to avoid division by zero.

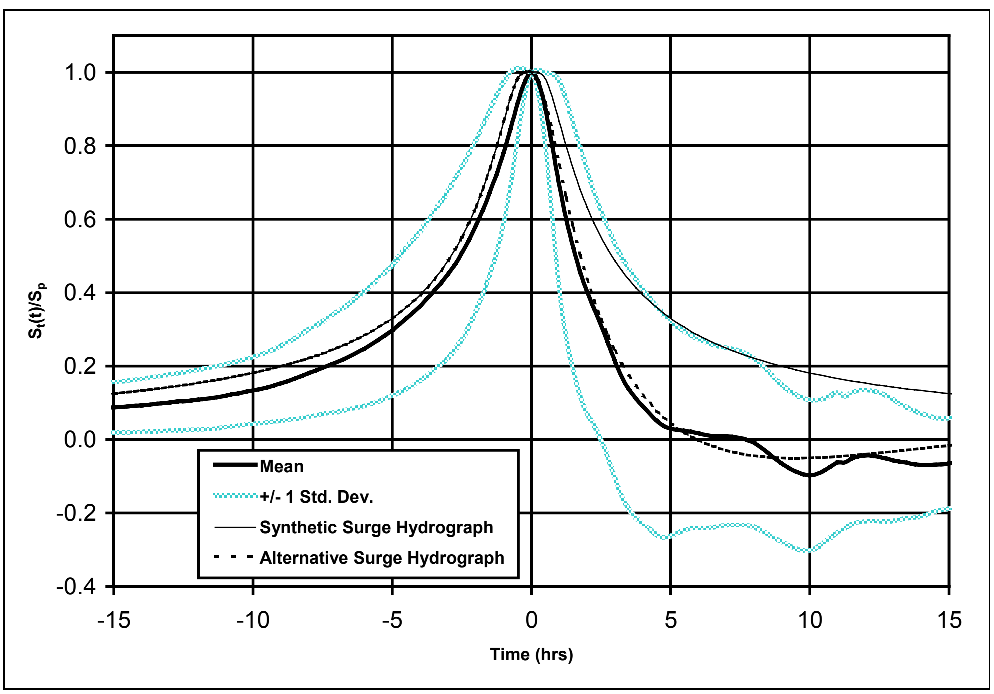

Figure 2.3. Synthetic and alternative surge hydrographs.

Equation 2.2 should be used in place of Equation 2.1 because it more realistically represents the falling limb of the surge hydrograph. Peak flow conditions, however, typically occur during the rising limb of the surge (when Equation 2.1 applies). This is the case because the surge wave is usually translating upstream during much of the falling limb. Equation 2.2 is recommended because its use should better identify the locations where the most extreme hydraulic conditions occur during the ebb storm tide.

Figure 2.3 also supports the use of a synthetic hydrograph shape for storm surge analysis. While steeper or flatter hydrographs definitely occur, on average the use of a mean duration (D = R/F) of approximately 2 hours appears reasonable. For a particular analysis, one could select a steeper or flatter hydrograph shape. If the +1 standard deviation hydrograph shape were selected and combined with a 100-year surge elevation, then there would be an approximate one-sixth chance of this event or steeper, and a five-sixth chance of a flatter hydrograph. The mean hydrograph is selected because it is the most likely to occur.

2.2 Combining Daily Tides and Storm Surges

Storm surges can occur at any time during the tide cycle. Therefore, one approach for determining scour at a bridge is to perform numerous simulations and perform a statistical analysis of the resulting velocities. This is the EST method described in the Phase II documentation (Zevenbergen et al. 1997a, 1997b). To completely implement the EST approach, one would not only vary the timing of the surge with respect to the tide, but also vary the surge height, storm forward speed and radius of maximum winds with statistically and physically meaningful variability. Enough simulations would have to be performed to develop a reasonable statistical distribution of velocity. If a site had eight historical hurricanes that had occurred near the site, it would require 32 simulations to develop the appropriate response statistics. Recognizing the significant effort that is often required to perform a single hurricane surge simulation, especially for two-dimensional modeling, the single hydrograph method was recommended for most practical applications.

The single hydrograph method requires the engineer to use either Equation 2.1 or 2.2 and select a time of landfall during the tide cycle. **Figure 2.4** shows four representative conditions of the peak surge coinciding with low tide, mid-rising tide, high tide and mid-falling tide. The condition shown is a 10-foot total surge plus tide, or storm tide, resulting from a 4-foot daily tide combined with varying levels of storm surge using Equation 2.2 for the storm surge shape.

Figures 2.5 and 2.6 show discharge and velocity hydrographs for the four conditions at a bridge crossing an inlet. For this case, the most extreme discharge occurs on the flood tide for the mid-rising tide condition. The maximum velocity, however, occurs on the ebb tide for the mid-falling tide condition. From this simulation, the high tide condition produces values close to the peak for both the flood and ebb tides. Another consideration in selecting the tide and surge combination is wind effects. If the same simulations are performed with a Category 3 hurricane (125 mph maximum winds) tracking through the inlet and bay at the worst possible track, flood tide discharges and velocities increased by approximately 17 percent and ebb tide discharges and velocities increased by approximately 12 percent.

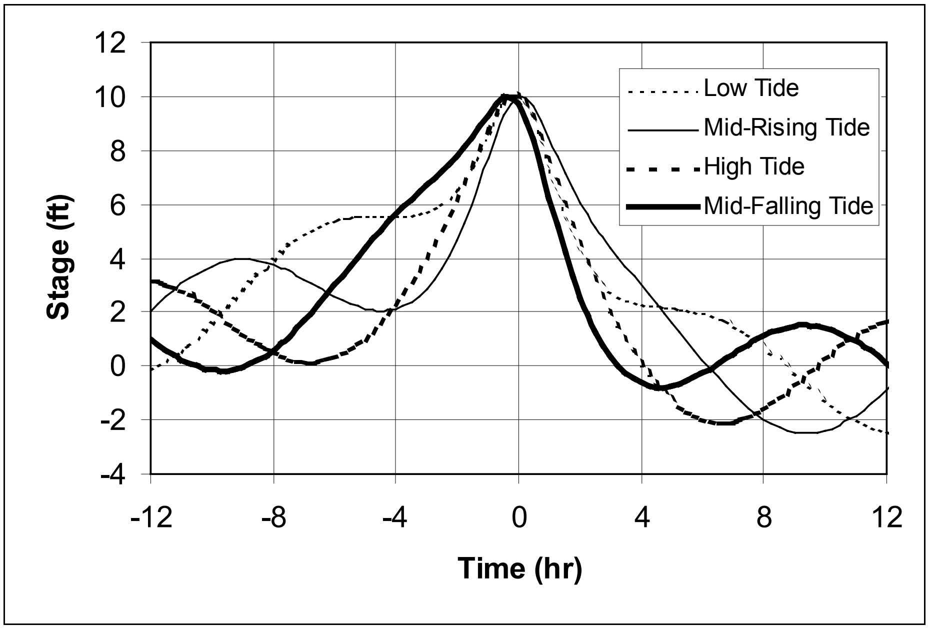

Figure 2.4. Combined surge and tide hydrographs.

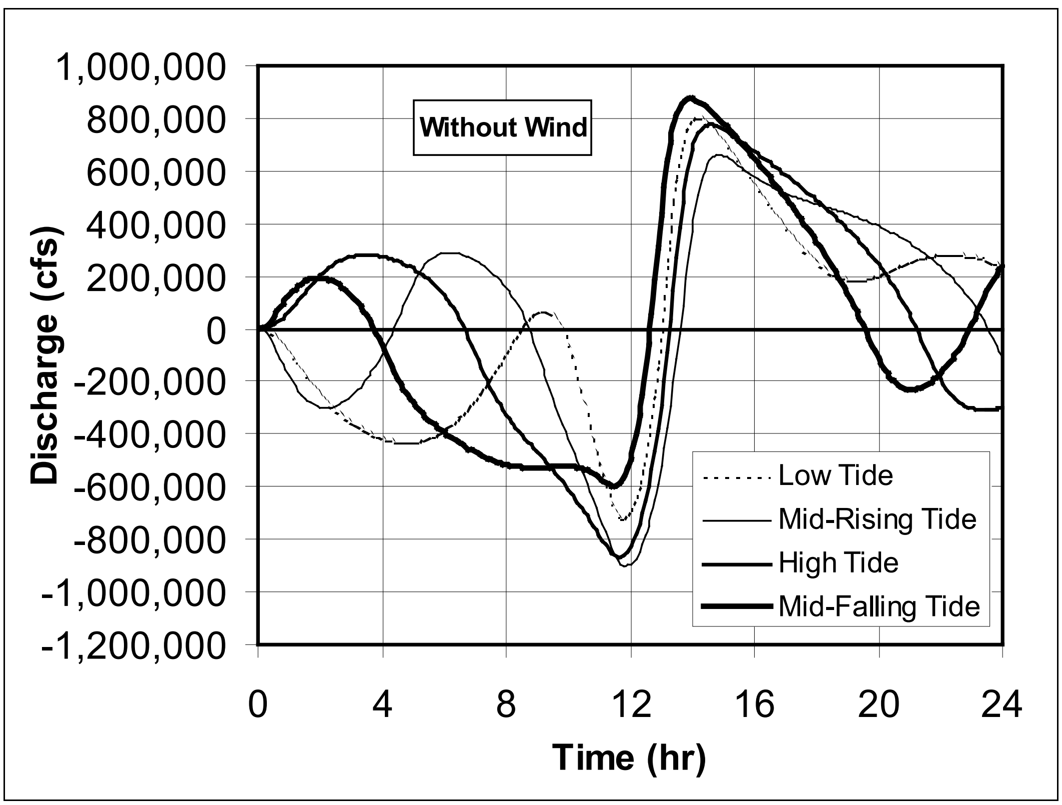

Figure 2.5. Discharge hydrographs without wind effects.

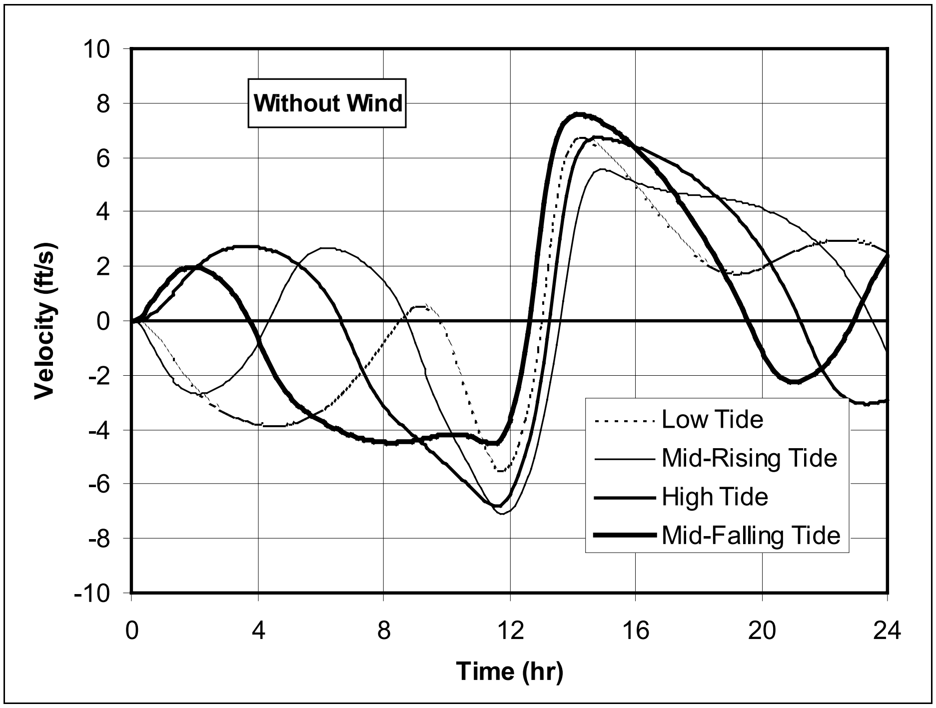

Figure 2.6. Velocity hydrographs without wind effects.

The situation used for illustration was an inlet to a bay. This is the situation where flood and ebb hydraulic conditions for a surge are most similar. For estuaries, especially long estuaries or estuaries with large amounts of floodplain inundation, the flood tide is expected to be significantly worse than the ebb tide. Therefore it is recommended that mid-rising tide be used if the single hydrograph method is required. If analysis time permits, it is useful to test the sensitivity of the results to storm surge timing and the engineer should select the tide and surge combination that produces severe conditions at the bridge.

2.3 Correlating Hurricane Category and Return Frequency

The migration of people to coastal areas and the resulting increased development makes hurricane awareness and preparedness critical. The public and government officials rely on hurricane categories for evacuation planning and storm response purposes. Hurricane categories are not typically used for design purposes but can be correlated to frequency indexed storm surges more commonly desired for engineering analysis and design.

2.3.1 Hurricane Categories

Hurricanes are rated on the Saffir-Simpson Hurricane Scale. The scale is a one through five rating (called categories) based on the hurricane's intensity. It is an indicator of the property damage and coastal flooding that results from a hurricane landfall. Since storm surge values are highly dependent on the continental shelf slope in the landfall region, wind speed is the determining factor in the scale. Winds are evaluated using the maximum one-minute surface (10 meters above water or ground) wind speeds. The wind speed, storm surge, and potential damage for all five hurricane categories are summarized below.

- **Category One Hurricane**: Winds are 74-95 mph (64-82 kt or 119-153 kph). Storm surge is generally 4-5 ft above normal. Damage is primarily to unanchored mobile homes, signs, shrubbery, and trees. No real damage is expected to building structures. Flooding is limited to coastal roads.
- **Category Two Hurricane**: Winds are 96-110 mph (83-95 kt or 154-177 kph). Storm surge is generally 6-8 ft above normal. Building roofing materials, doors and windows will incur some damage. Considerable damage is expected to mobile homes, signs, shrubbery, and trees. Coastal and low lying escape routes will flood two to four hours before the arrival of the hurricane center.
- **Category Three Hurricane**: Winds are 111-130 mph (96-113 kt or 178-209 kph). Storm surge is generally 9-12 ft above normal. Damage will occur to small residences and utility buildings. Mobile homes and signs are destroyed. Flooding near the coast will destroy smaller structures and the battering of floating debris damages larger structures. Terrain that is continuously lower than 5 ft above mean sea level may be flooded inland for 8 miles (13 km) or more.
- **Category Four Hurricane**: Winds are 131-155 mph (114-135 kt or 210-249 kph). Storm surge is generally 13-18 ft above normal. Damage to structures is more extensive including some complete roof structure failures on small residences. Major damage occurs to lower floors of structures near the shore. Terrain lower than 10 ft above sea level may be flooded.
- **Category Five Hurricane**: Winds are greater than 155 mph (135 kt or 249 kph). The storm surge is generally greater than 18 ft above normal. Damage includes complete roof failure on many residences and industrial buildings with some complete building failures. Major damage to lower floors of structures along the shoreline. Flooding may occur 5-10 miles (8-16 km) inland.

2.3.2 The HURDAT Database

Due to the wide range of damage associated with the various hurricane categories, it is useful to relate the return period with each hurricane category for any given location along the coast. The National Hurricane Center maintains a database (HURDAT) that contains the 6-hourly center locations (latitude and longitude in tenths of degrees) and intensities (maximum 1-minute surface wind speeds in knots and minimum central pressures in millibars) for all tropical storms and hurricanes from 1886 through the present. Using the HURDAT database, it is possible to correlate hurricane category and return frequency along the entire United States coastline. HURDAT can currently be accessed at http://www.nhc.noaa.gov/pastall.html.

2.3.3 Return Period Maps

The data obtained from the National Hurricane Center were input into an ArcView Geographic Information System (GIS). A custom script was written to enable the hurricane tracks to be input using the 6-hour latitude and longitude of the storm's center and create a graphical representation of the hurricane track. The hurricane tracks were color coded by category and integrated with a map of the United States Atlantic Coast. The location of the hurricane eye for each landfall is determined by the intersection of the hurricane track with the shoreline. The coastline was divided into 50 nautical mile segments beginning at Key West, Florida and extending north to Maine. The number of occurrences for each coastline segment was obtained graphically using the GIS map.

The maps seen in **Figures 2.7 to 2.9** show return periods along the East Coast for category 1 or greater, category 2 or greater, and category 3 or greater hurricanes, respectively. The number in parentheses is the number of landfall occurrences of hurricane eye for each 50 nautical mile segment during the 113-year period from 1886 to 1999. An approximate return period was calculated by taking the total number of occurrences and dividing it by the 113 years of record. The maps only indicate the return frequency of the storm eye for each window. The maps do not take into account other factors such as the size of the storm, storm asymmetry, land falling or land exiting. Along-shore storms are not included.

The maps show that the majority of hurricanes make landfall along the Florida and North Carolina coasts. The strongest hurricanes (category 3 or greater) are most frequent in south Florida and the Outer Banks of North Carolina. Category 3 storms are rare north of Virginia and the eye of a storm greater than category 3 has never made landfall north of Wilmington, North Carolina.

The return periods reflect the frequency with which the eye of a storm crosses a particular coastline segment. The storm eye is easy to track but is only one aspect of a storm that must be considered when determining its impact. The storm (and storm surge) may extend 50 or more miles from the center. The most intense winds of a hurricane are on the right side of the storm. Therefore, a storm that is heading west making landfall near Miami may have a significant impact on the coast 50 to 100 miles to the north. For a storm that is heading north and making landfall at the Outer Banks of North Carolina, the most intense winds would be east of the eye and over open water, reducing the impact on coastal areas. Factors such as these should be considered when using the maps.

The correlation of occurrence and hurricane category is not a design tool but should be used as an indicator of the frequency that one may expect. The charts can also be used to select wind speeds for wave heights calculations and for input for 2-dimensional models. For example, along the south part of the South Carolina coast, a 15-year wind speed would be associated with a Category 1 storm, a 25-year wind speed would correlate to a Category 2 storm, and a 50- to 100-year wind speed would correlate to a Category 3 storm.

2.4 Combining Storm Surge and Upland Runoff

As discussed in the Tidal Users Manual (Zevenbergen et al. 1997b), as the drainage basin size upstream of the bridge increases there is less chance that upland runoff can significantly affect flow at the bridge during a storm surge. This is true for either the flood or ebb surge condition. This section addresses the conditions when significant upland runoff would result from a hurricane and provides recommendations for combining upland runoff with a storm surge. The information contained in this section draws primarily from the Corps of Engineers Storm Surge Analysis Manual (USACE 1986).

The Corps' manual includes data on hurricane induced rainfall along a the gulf coast from Apalachicola, Florida to Brownsville, Texas. The rainfall is distributed relative to the storm track for a zone that extends from the coastline to 25 miles inland. **Figure 2.10** is an example distribution of rainfall for the six hours preceding landfall. This figure indicates that significant rainfall does occur as the result of hurricane conditions and that the maximum rainfall occurs to the right of the storm and in the vicinity of the maximum winds.

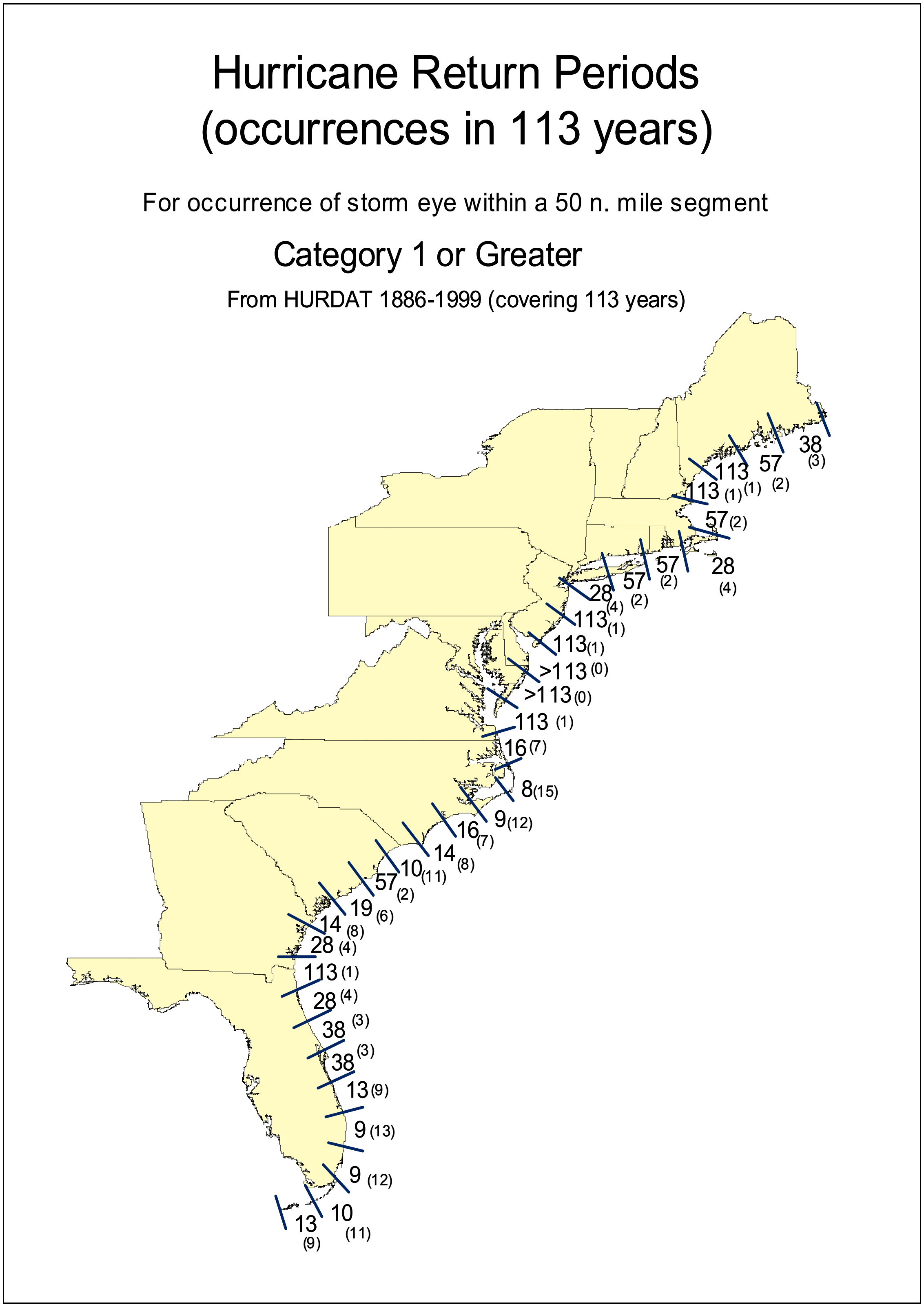

Figure 2.7. Return period of Category 1 or greater hurricanes along the Atlantic coast.

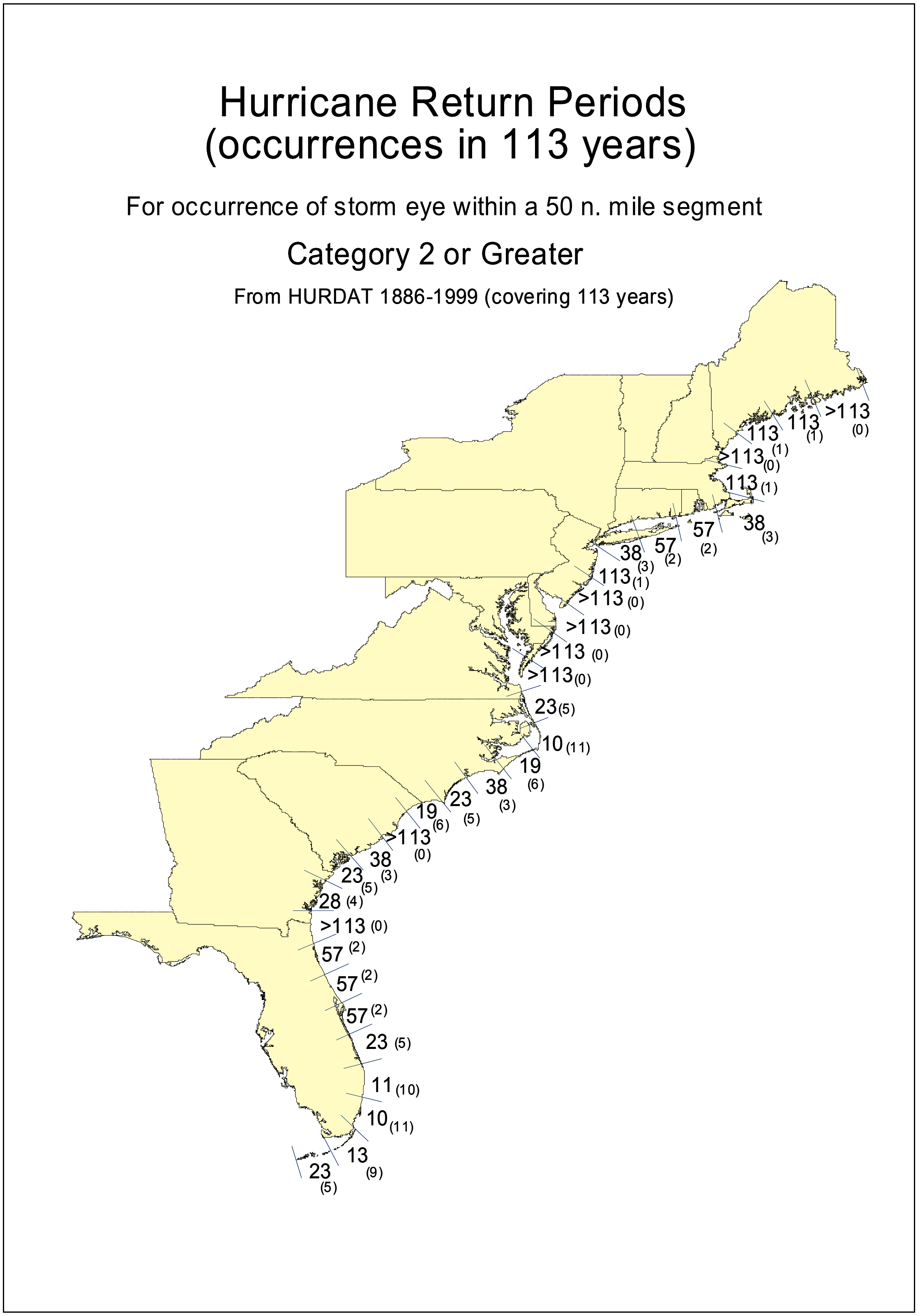

Figure 2.8. Return period of Category 2 or greater hurricanes along the Atlantic coast.

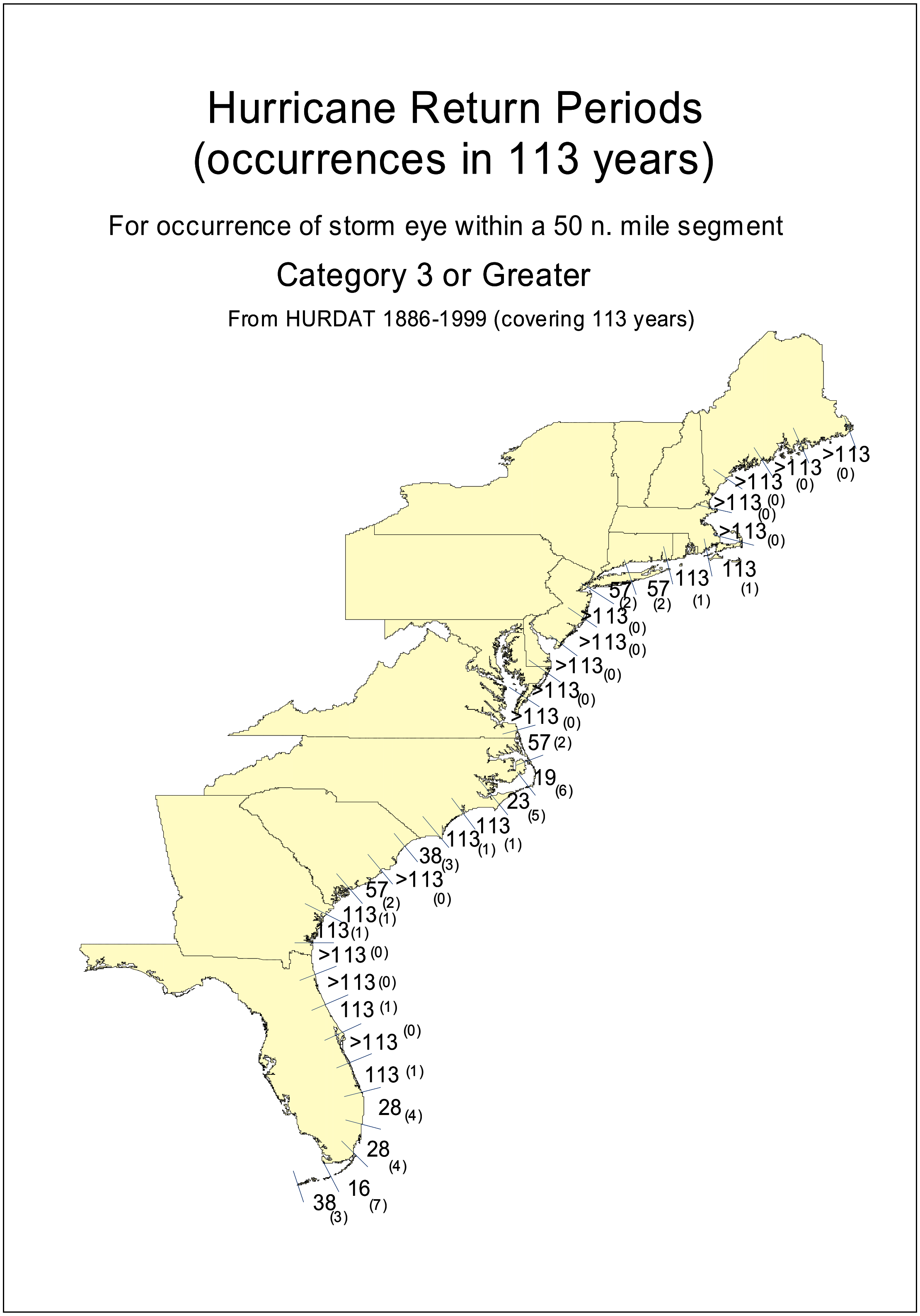

Figure 2.9. Return period of Category 3 or greater hurricanes along the Atlantic coast.

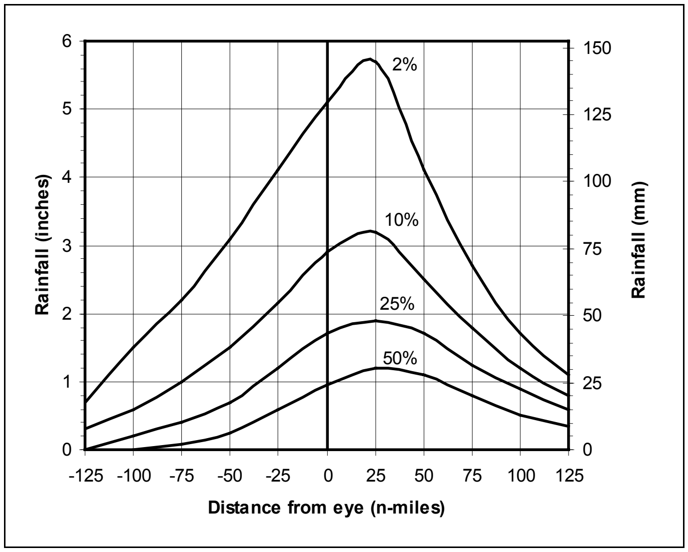

Figure 2.10. Rainfall for six hours preceding landfall (after USACE 1986).

For comparison, the 6-hour rainfall amounts from NOAA (Hershfield 1961) for this part of the coastline (from Brownsville, Texas to Apalachicola, Florida) are approximately 8, 7, and 6 inches for 50-, 25-, and 10-year recurrence intervals compared with the 5.7 inches for the two percent (50-year recurrence) probability shown in Figure 2.10. The frequency distributions for NOAA rainfall amounts are not related to any specific type of weather condition; it includes thunderstorms, fronts, and tropical storms. Therefore, if upland runoff is included in a storm surge simulation, some lower frequency rainfall (and runoff) than what would be expected from use of the NOAA atlas should be used than the surge frequency.

Figure 2.11 shows the two percent probability (50-year recurrence) temporal rainfall distribution near the coast and 25 nautical miles to the right of a storm. The rainfall amounts have been converted to average intensities. For example, the average intensity for the six hours preceding landfall is 5.7/6 = 0.95 in/hr for a 50-year frequency event. The six hours preceding landfall are the most intense. Since the peak ebb flow generally occurs two to three hours after landfall (Figure 2.6) the amount of the basin that could contribute to the ebb flow is limited by the short lag time. Lag time is defined as the time from the centroid of excess rainfall to centroid of runoff. Excess rainfall would probably occur within the six hour period preceding landfall for the peak upland runoff to occur during the ebb tide. Therefore, depending on the bridge location and the forward speed of the storm, the lag time should be in the range of five to ten hours. The amount of drainage area contributing to flow during the peak ebb surge would be limited by the lag time. Portions of the basin outside the contributing area are too far removed to have an effect during the ebb flow caused by a surge.

Given a target lag time, it is possible to determine the basin area contributing to flow. Using South Carolina as an example, regression equations are available to predict peak flow as a function of drainage area (Guimaraes and Bohman 1988). The drainage area used to predict flow is the lag time limited drainage area. Unit hydrographs and lag times are available for South Carolina and are also a function of drainage area (Bohman 1989). Therefore, since for South Carolina the lag time is approximately equal to the time to peak, the drainage area and complete upland flow hydrograph can be computed, as illustrated in the following example.

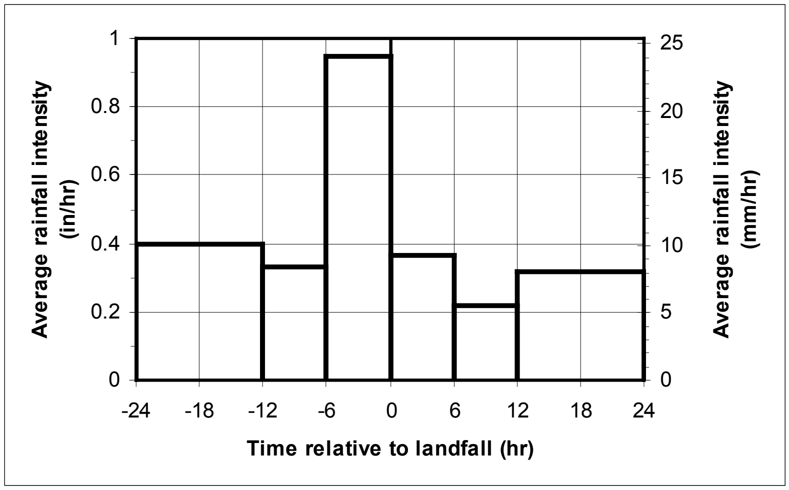

Figure 2.11. Temporal rainfall distribution 25 n-miles right of the eye.

A 100-year storm surge analysis is being performed for a basin with a total drainage area of 50 square miles. From dynamic storm surge modeling, the peak ebb surge of 5,000 cfs occurs three hours after the landfall. Assuming that the centroid of excess rainfall is three hours before landfall, then the desired time to peak is six hours and this value is also the approximate lag time. Assume that a 50-year peak runoff could result during the 100-year surge. The 50-year Q_p (in cfs) for the South Carolina Lower Coastal Plain is $Q_{p50} = 275A^{0.58}$ and the lag time in hours is $LT = 6.95A^{0.348}Q_p^{-0.022}$ for the Lower Coastal Plain Region 1. Manipulating the two equations results in a contributing area of 0.94 square miles and a 50-year upland peak flow of 266 cfs for the 6-hour lag time. The flow would reach approximately this value three hours after landfall and can be added to the peak ebb surge or the hydrograph could be included in the numerical model as an inflow at the bridge site. After the peak ebb surge, the ebb flows would decrease and the upland runoff would increase due to greater contributing areas. The peak 50-year flood flow would be 2,660 cfs occurring 20 hours after landfall at a time when daily tides would again predominate. If the bridge were located in an urban area, the South Carolina urban hydrograph procedure could be used (Bohman 1992). The urban hydrograph procedure would be more difficult to apply because lag time is a function of channel length and slope rather than area, so an iterative process would be required.

This example illustrates that for large tidal waterways, upland runoff could only have an impact on a storm surge if a totally unrelated upland flood occurred at the same time. The combined probability of the two events is so low that it does not need to be analyzed. For most moderately sized tidal waterways, it is also safe to assume that upland runoff will not have a significant affect on the results because the magnitude of upland runoff occurring during the ebb surge should be relatively small. For these situations, a mean daily flow or base flow should be included. The Corps surge manual recommends increasing the mean daily flow by 20 percent to conservatively account for rainfall and runoff in the lower basin.

The only situation that would be expected to have significant upland flow contributions is a small tidal waterway in an urban area. For these areas the tidal hydraulics should be performed, lag times and contributing areas estimated for upland flows with recurrence intervals approximately half the surge frequency, and the two flows combined. Lag times should be from approximately three hours before landfall to the time of the peak ebb flow (approximately three hours after landfall) resulting in a lag time of between five and ten hours. The lag time should not exceed the total basin lag time for small drainage basins.

2.5 Time Dependent Contraction Scour

Hurricane storm surges often produce extreme hydraulic conditions for time periods of only a few hours. Computing ultimate contraction scour amounts for these conditions may not be reasonable. Ultimate contraction scour is reached when the sediment supply from upstream is matched by the sediment transport capacity in the scoured bridge opening. Equating sediment transport capacity to upstream supply results in the HEC-18 contraction scour equation, which uses a simplification of the Laursen sediment transport equation. Sediment transport relationships could be used directly to compute ultimate contraction scour although the considerably greater effort (needed to apply sediment transport equations directly) would yield results very similar to the HEC-18 equation. Applying sediment transport formulas to contraction scour is recommended in HEC-18 for more complex situations. Specifically, HEC-18 states:

> Both the live-bed and clear-water contraction scour equations are the best that are available and should be regarded as a first level of analysis. If more detailed analysis is warranted, a sediment transport model like BRI-STARS (Molinas 1990) should be used.

A sediment transport model, such as BRI-STARS, could be used to compute ultimate contraction scour conditions for a constant flow rate as long as a sufficient simulation duration was used. It could also be used for unsteady conditions and/or for shorter durations. Similarly, sediment transport relationships could be used directly (outside a sediment transport model) to make predictions of ultimate scour and, since the equation produces a rate of sediment transport, the rate of contraction scour.

Generally, sediment transport modeling is beyond the scope of most scour studies. Therefore, it is useful to apply sediment transport techniques, but in a simplified approach. The method selected was to use the same data required by the standard HEC-18 contraction scour equations within a spreadsheet sediment transport application. The required data are channel width, discharge and average flow depth within the bridge opening and at an approach section, median bed material size, fall velocity and an estimate of Manning n. One other parameter that must be estimated for this procedure is the scour hole slope. Hoffmans and Verheij (1997) suggest upstream scour hole slopes of 1V:2H for clear-water and 1V:4H for live-bed and downstream slopes ranging from 1V:8H to 1V:40H. For this application, 1V:1H upstream and downstream slopes have been assumed for conservative results. The steeper the upstream and downstream scour hole slopes, the faster that scour will occur because a smaller volume of material is eroded. The spreadsheet also allows the user to adjust for the void space in the bed material. The sediment transport equation predicts a particular rate of transport, but this rate includes no void space. Therefore, the rate of erosion is greater than the transport rate by a factor or $1/(1-\eta)$ where η is the void ratio. The default for η is 0.4 (40 percent void space) which is reasonable for sand.

The spreadsheet application requires the peak hydraulic conditions at the bridge and approach. These hydraulic conditions should be obtained from 1- or 2-dimensional dynamic storm surge modeling for peak flow conditions. The spreadsheet application computes the rate of sediment deficiency at the bridge opening (transport capacity in the bridge opening minus sediment supply from upstream) for a time interval, computes the volume of sediment scoured during that time interval, updates the hydraulics based on the amount of scour and repeats this process until the ultimate scour is reached. As with the derivation of the HEC-18 contraction scour equations, hydraulic depth is used rather than hydraulic radius, but the sediment transport function is used directly without further simplification.

2.5.1 Sediment Transport Formulas

Several sediment transport formulas were used to test the sensitivity of the rate of scour. These included the Laursen, Engelund-Hansen, and Ackers-White functions as presented in Sediment Engineering (ASCE 1975), and Yang (1996). These equations were selected based primarily on their ease in application. **Figure 2.12** shows the first 100 hours of applying these formulas to a significant contraction scour situation. Also shown on Figure 2.12 is the ultimate scour predicted by the HEC-18 equation. Within the first 45 hours, the Ackers-White and Engelund-Hansen formulas have exceeded the ultimate scour predicted by the HEC-18 equation. Logarithmic scaling is required to plot the complete scour development (**Figure 2.13**). Surprisingly, the four sediment transport equations predict ultimate scour within 0.5 m (1.6 ft) of the HEC-18 equation which, in this case, is approximately 4 m (13 ft). This is within 13 percent of the HEC-18 estimate. Both the Laursen and Yang formulas predict ultimate scour within 0.1 m (0.3 ft) of the HEC-18 equation.

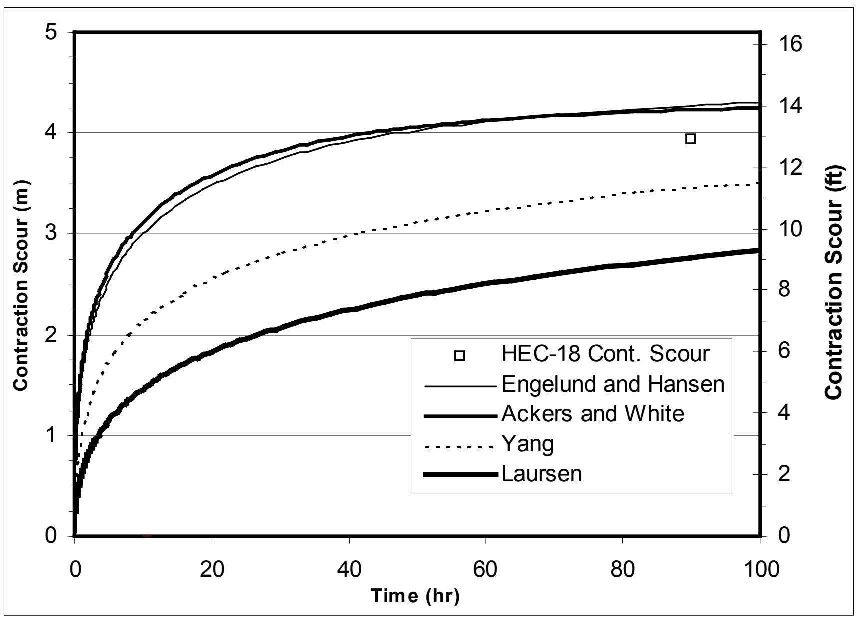

Figure 2.12. Contraction scour for several sediment transport formulas (first 100 hours).

The reason that there is not exact agreement between the Laursen and HEC-18 results is the simplification related to excess shear stress in the derivation of the HEC-18 equation. The Yang equation generally predicts slightly greater scour than the standard contraction scour equation. The consistency of the ultimate scour predictions (even though four different sediment transport functions are used) is based on the fact that the same transport function is applied to both the approach and bridge sections. The Yang equation was selected for application because (1) it is simple to use, (2), it predicts ultimate scour amounts similar to the HEC-18 equation, (3) it is more conservative (with regard to rate of scour) than the Laursen equation and (4) it has wide acceptance. The subsequent analyses presented in this section are based on the Yang equation.

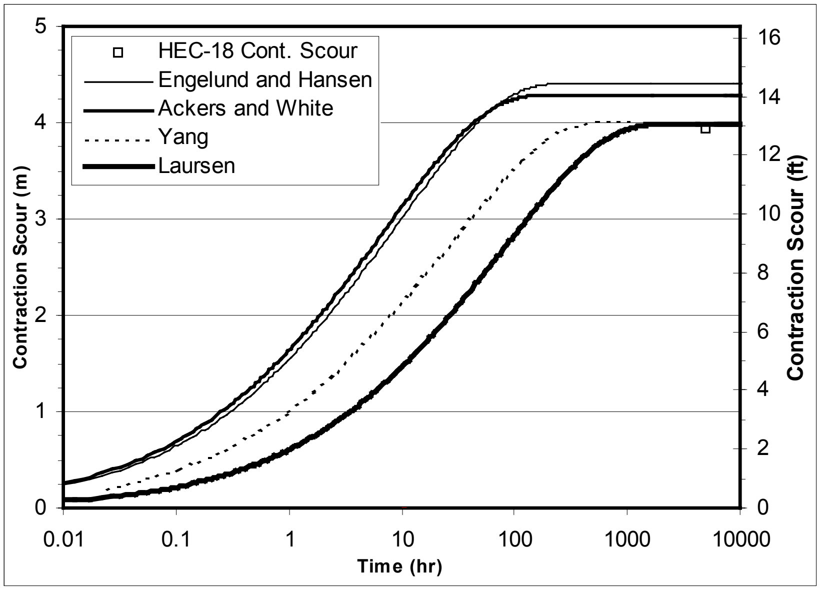

Figure 2.13. Complete development of contraction scour for several sediment transport formulas.

2.5.2 Verification Using BRI-STARS

The spreadsheet application was compared to a BRI-STARS (Molinas 1990) model using the Yang equation and similar input and assumptions. This comparison was performed to ensure that the sediment transport equations were properly applied over the range of hydraulic conditions and that the computations converting sediment transport rates to volumes of scoured material were applied correctly. **Figure 2.14** shows a comparison of the spreadsheet results and the BRI-STARS results. Agreement is excellent.

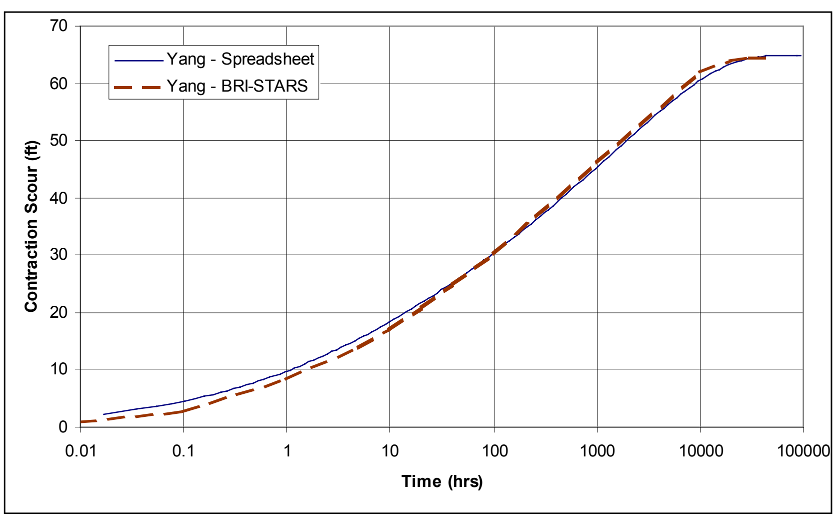

Figure 2.14. Comparison of spreadsheet contraction scour to BRI-STARS.

2.5.3 Sediment Supply

For a particular hydraulic condition at a bridge, the time it takes to reach ultimate scour depends on the amount of sediment supplied from upstream. **Figure 2.15** shows the time to reach ultimate scour for varying upstream sediment supply rates. The hydraulic condition in the constricted bridge opening was held constant and the discharge in the approach section channel was adjusted to provide varying rates of sediment supply. As shown in **Figure 2.16**, the primary cause of the faster development of the live-bed scour hole is due to significantly lower levels of ultimate scour. Although the scour at ten hours ranges from 20 percent to 100 percent of the ultimate scour, the magnitude for ten hours ranges from 0.12 m (0.4 ft) to 2.3 m (7.5 ft) and the range in ultimate scour is from 0.12 m (0.4 ft) to 11 m (36 ft). However, to reach a particular amount of scour, say 2 m (6.6 ft), Figure 2.16 shows that increasing sediment supply delays the scour development. This is because the scour rate is the difference between the transport capacity and sediment supply and, therefore, a scour hole of a particular depth should take longer to develop as sediment supply increases. Figure 2.16 also illustrates that the rate of scour is not significantly affected by upstream supply until the scour hole has reached approximately half the ultimate depth.

2.5.4 Application

This methodology is a simple spreadsheet alternative to sediment transport modeling but can result in a prediction of considerably less scour in some situations. By using the peak hydraulic conditions and steep upstream and downstream scour hole slopes, the method should be conservative. This level of conservatism is warranted due to the rapidly varied flow in a bridge constriction. Based on the surge hydrograph or flashy upland stream flow hydrograph, the engineer can select a reasonable time interval for the scour prediction. For hurricane storm surges, the time interval would typically range from two to five hours. It is recommended that the discharge and velocity hydrographs be reviewed to establish a reasonable time interval to apply the peak discharge. Alternatively, the method could be adapted to use time varying hydraulics or a sediment transport model could be used.

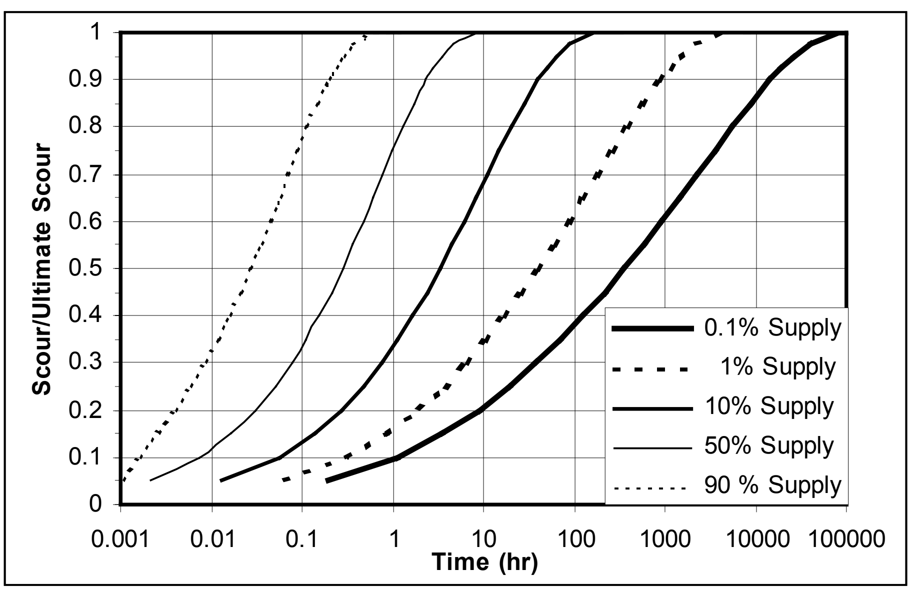

Figure 2.15. Effect of sediment supply on relative contraction scour rate.

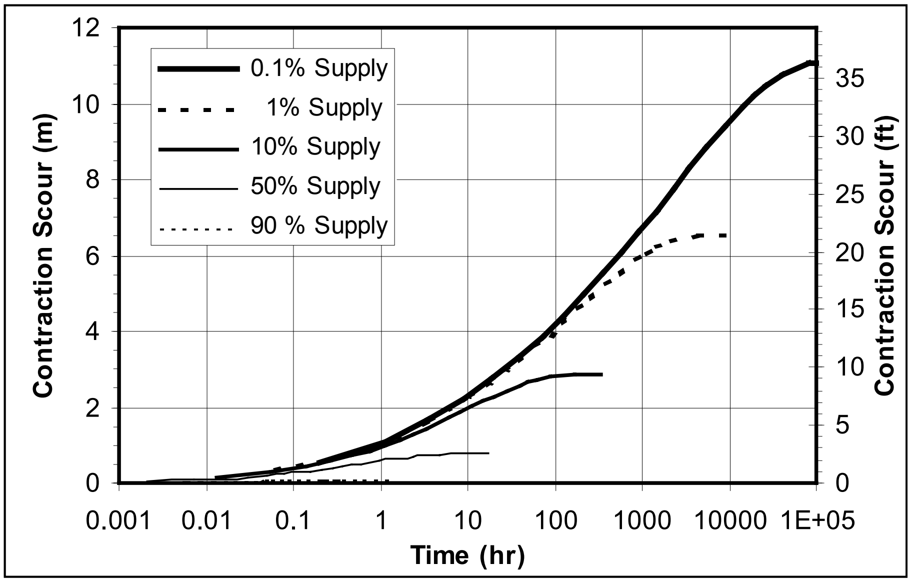

Figure 2.16. Effect of sediment supply on contraction scour rate.

2.5.5 Time Dependent Local Scour

Several methods exist for predicting rates of pier and abutment scour. Gosselin and Sheppard (1998) concluded that more research is needed before meaningful relationships can be developed for time dependant local sour. This is because most of the research has been conducted on clear-water conditions (approach velocity less than the critical velocity for sediment transport) and at small scales. It is generally accepted that local scour in live bed conditions occurs much more rapidly than for clear water conditions. As this area of research evolves there may be benefits to computing time dependant local scour amounts. One additional complication is that these amounts would have to be added to ultimate local scour amounts produced by daily tides.

2.6 Computing Surge Plus Wave Height At A Bridge

It is useful to determine both the surge and wave height when establishing the low chord of tidal bridge decks. South Carolina, for example, uses the 10-year surge plus wave height plus two feet freeboard as the minimum elevation for the bottom of the deck. NOAA (1975) provides surge elevations (10-, 50-, 100-, and 500-year) for the South Carolina coast. The 10-year surge for South Carolina is approximately equal to the highest spring tides. From this observation, it is reasonable to use a relatively low hurricane wind to compute wave heights. Also, the hurricane return periods (Section 2.3) indicate that there have been only 14 category 2 hurricanes along the entire South Carolina coast in 113 years. Wind speeds for a Category 1 hurricane, which has maximum winds between 74 and 95 mph, should be used for this relatively frequent event. The maximum winds occur along the right side of the hurricane eye.

The methodology recommended to compute wave heights is from the Shore Protection Manual (USACE 1984). This procedure is quite lengthy but has been incorporated into a spreadsheet that is available to the Pooled Fund. The data required are wind speed, fetch length, channel flow depth, and floodplain flow depth. The spreadsheet computes wave heights in the channel and floodplain, the wave length and period, the duration of wind required to produce the waves, and the wave classification. The computed wave height is the "significant" wave height which is defined as the average height of one-third highest waves. This wave height is converted into a one percent wave, H_1, or the average of the one percent highest waves. Although the waves are assumed to be limited by the fetch, they may also be limited by the duration of the wind.

As shown in **Figure 2.17** half the computed wave height is added to the surge elevation. Also shown in this figure is the wave length. The period is time between waves, which is the wave length divided by the wave speed. Waves are classified as (1) deep water, where the wave height is virtually unaffected by the bottom and the celerity is unaffected by the water depth, (2) transitional waves, where the bottom affects the wave height and depth affects the wave celerity, and (3) shallow water waves, where celerity is only a function of depth and waves are more likely to break. The maximum wave height is approximately 0.78 times the flow depth. This limiting wave height is unlikely in the deep channel area but a reasonable estimate for shallow floodplain areas.

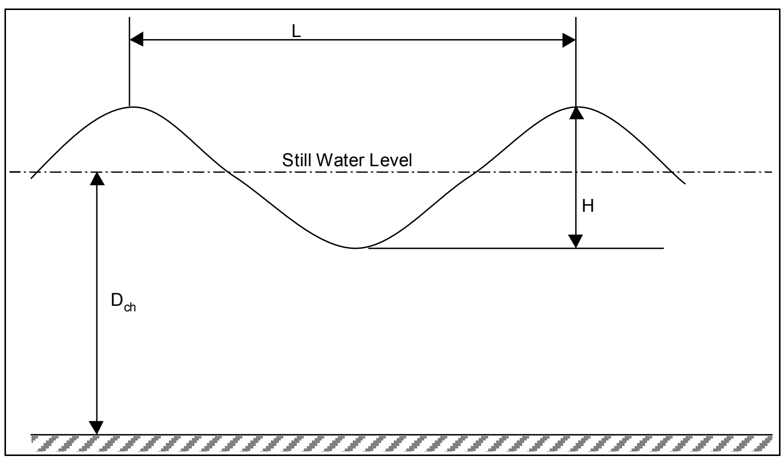

Figure 2.17. Wave height plus surge.

For the purposes of computing wave heights within a bridge opening, the definition of fetch requires the greatest judgement. Fetch is the distance of unobstructed wind with fairly uniform speed and direction. **Figure 2.18** shows a road embankment and bridge crossing a floodplain and channel. The floodplain is assumed to have some shallow depth of flooding during the 10-year surge. The wind is assumed to be oriented in the worst case direction with respect to the channel, but within a range of directions that can be reasonably produced near the peak of a 10-year storm surge. The range of directions should be limited to within 45 degrees of the storm track. Land is an absolute limit to the fetch. Because waves tend to break in shallow water, the length of deeper channel could limit the fetch. It is reasonable, however, to extend the fetch somewhat upwind of the deep channel area, perhaps by 1,000 to 2,000 feet.

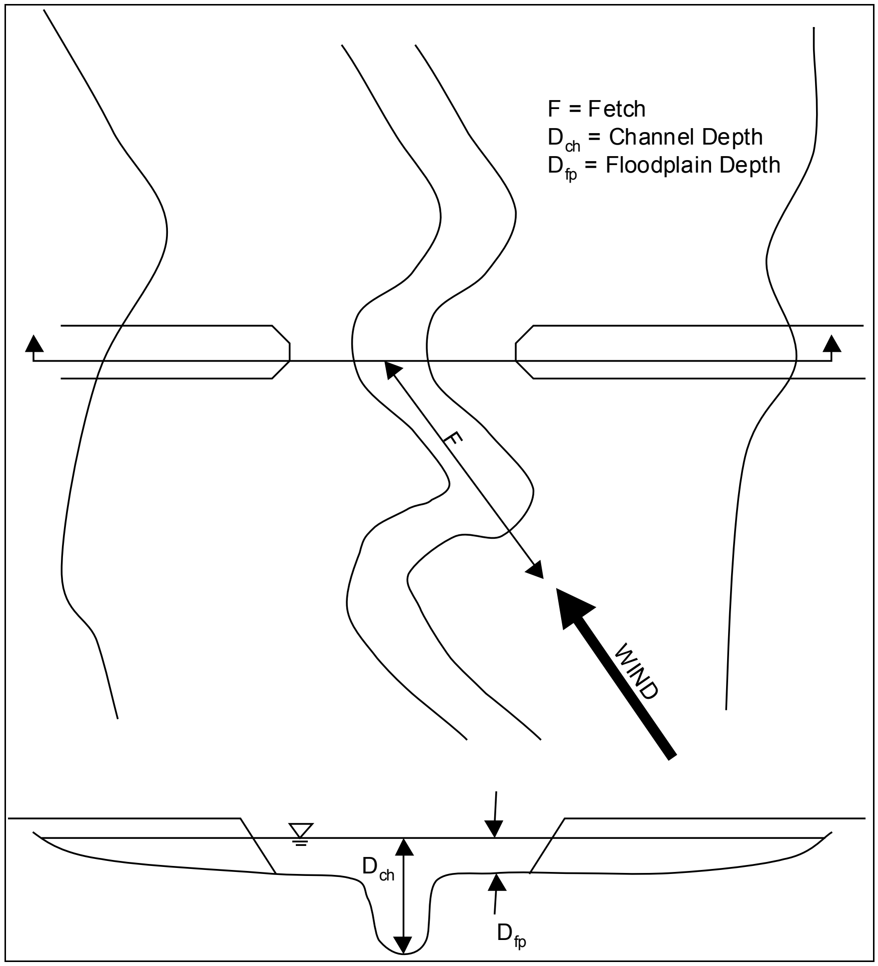

Figure 2.18. Definition sketch for wave calculations.

The tidal users manual includes all the equations and procedures for computing surge plus wave height. These equations and procedures have been incorporated into a spreadsheet for practical application.

2.7 Wind Effects

This section contains guidance on including wind stress in surge simulations. Wind is a significant component of the surge at a coastline. Although there are 1-dimensional models that include wind effects, it is most reasonable to use 2-dimensional models for simulations that include wind. The Corps Storm Surge Analysis manual (USACE 1986) indicates that wind is the greatest component of surge and that the peak surge occurs in the area of maximum winds. Each of the sources of data for surge along the open coast, (NOAA, FEMA, and ADCIRC) already have wind effects included. If wind effects are included in the analysis of an estuary, bay or other tidal waterway, then additional considerations are needed. These include (1) adjusting the maximum hurricane wind speeds to durations long enough to realistically move water within the waterway, (2) accounting for wind that has been interrupted by land as it moves from ocean to an embayment, (3) using a realistic storm path that is consistent with the simulated surge, and (4) using realistic hurricane wind fields.

Figure 2.19 shows a relationship for adjusting wind speed between different durations. Factors are related to the one hour sustained wind speed (U_{3600}). From this figure, a one-minute sustained wind speed is 1.24 times faster than a 1-hour wind speed and is 1.23 times faster than a 30-minute wind speed. Therefore, a hurricane with maximum winds of 100 mph (a speed that is maintained for short durations of a minute or less) should be modeled with a maximum wind speed of 81 mph, which is the speed that is maintained over durations of 30 minutes to one hour. If the hurricane wind speed is defined as the Fastest Mile wind speed, a 100 mph wind is maintained for a duration of only 36 seconds. The 30-minute duration wind speed that is appropriate for modeling would be 78 mph. Although these are only 20 percent reductions in wind speed, the wind shear stress acting on the water surface is proportional to wind speed squared. Therefore, the stress is reduced to 64 percent of the stress that would be associated with durations less than one minute.

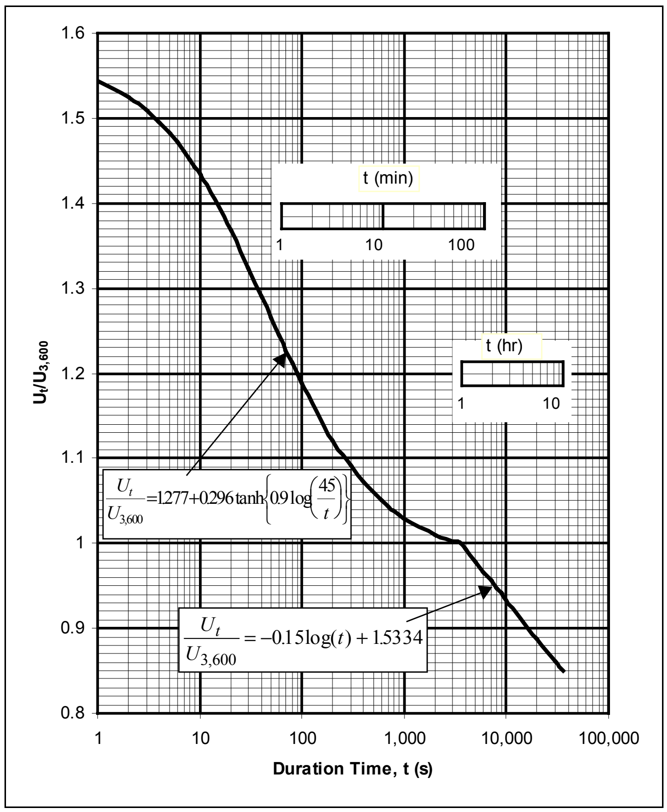

Figure 2.19. Ratio of wind speed for duration t to the 1-hour winds speed.

Another adjustment that should be made for winds over interior tidal waterways is related to the reduction in wind speed as it encounters coastal terrain. There is an immediate drop in wind speed when the wind reaches land and the speed continues to decrease as it moves over the land (**Figure 2.20**). When the wind reaches the tidal waterway, the wind speed gradually increases until it resumes the original velocity. For example if an embayment is 5 n-miles inland over rough terrain and has open water of 8 n-miles, the wind speed when it reaches water is 0.56 of the original wind speed and as it moves across the open water it regains the original speed (K=1.0). The average reduction of wind speed over the open water area, accounting for the upward curvature of the "to water" line, should be about 0.85. If this factor is combined with the factor for wind duration, a 100 mph maximum wind (Category 2 hurricane) would be modeled using a 69 mph maximum wind. The wind stress computed in the model would be only 47 percent of the stress had the model been run with the unadjusted 100 mph wind.

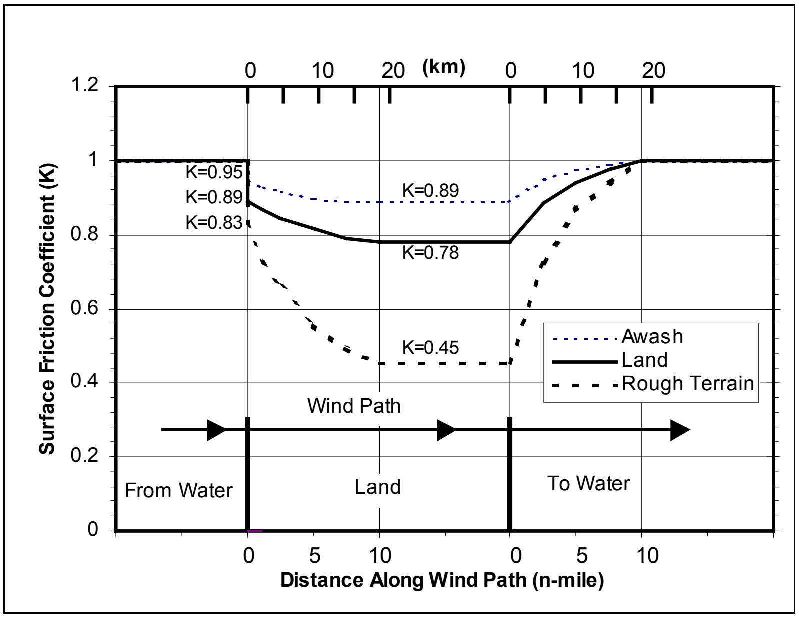

Figure 2.20. Wind speed adjustment for nearshore friction.

If wind is added to a storm surge simulation, then the modeler must select storm characteristics and a storm track that is consistent with the storm surge. The average forward speed and average radius of maximum wind is recommended. Depending on the location of the tidal waterway, a particular category hurricane should be selected. The most conservative storm path would typically place the maximum wind at the location of interest at the peak surge and the storm path would be aligned with the tidal waterway. For example, for a 100-year surge model of the Hathaway bridge at Panama City, Florida, a Category 3 hurricane (125 mph maximum wind) was included. The "without wind" model results are shown in Figures 2.5 and 2.6. For the maximum wind placed at and aligned with the bridge, maximum velocities increased by 18 percent. If the placement was maintained but the storm direction was rotated 45 degrees, the maximum velocities increase by 7 percent. Each of these storm tracks could reasonably have produced the 100-year ocean surge.

Identifying the worst case storm track is quite easy and for this reason alone could be used in the modeling. It should be recognized, however, that the results will be the most conservative and, therefore, the most unlikely to occur.

The RMA-2V model has a storm generator that can reasonably approximate storm wind fields. Although it is documented as an experimental feature, it is not supported in SMS and the storm cards must be added to the boundary condition file prior to model execution. The FESWMS model storm generator is, as yet, undocumented but is supported by SMS and produces a more realistic wind field than RMA-2V. The FESWMS storm generator produces 10-minute winds that are suitable for modeling (within approximately 4 percent of 30-minute wind speeds), but does not account for reduction of wind speed due to coastal terrain. The FESWMS storm generator uses atmospheric pressure to compute wind field. It is recommended that the user adjust the pressure until the desired wind speed is achieved. The desired wind speed should be appropriate for conditions over the embayment (not open ocean) including wind duration and terrain reduction factors.

3. PHASE III MODEL TESTING

3.1 FESWMS Version 3.0

In Phase II FESWMS (Froehlich 1996) was selected as the 2-dimensional model for training and manual development. FESMWS is a 2-dimensional finite element model that includes several hydraulic structure features useful for bridge hydraulic analysis. Version 2 of the model contained several bugs that limited its applicability to tidal and bridge hydraulics. The model could not be run in dynamic mode until shortly before the Phase II training course. Once the model was corrected to run in dynamic mode, element wetting and drying was not functional in dynamic mode. This has remained as a significant limitation in Version 2. Also, various updates of FESWMS Version 2 could not adequately analyze bridge pressure flow. The problems associated with bridge pressure flow included severe numerical instability, loss of flow continuity, and unrealistic amounts of backwater. In Phase II a work-around was developed using the depth variable Manning n routine in FESWMS (see appendix E of Zevenbergen et al. 1997b). The Manning n was adjusted to produce an appropriate amount of additional friction loss when the water depth exceeded the deck low chord. Although this resulted in an appropriate amount of energy loss through the submerged bridge opening, a considerable amount of effort was required to apply this work-around in a model.

Because of the significant potential for using FESWMS in tidal simulations the model has continued to be supported by the Pooled Fund for use in tidal applications. The features that are of significant interest to the Pooled Fund are the bridge structure features, automatic pier scour calculations, and sediment transport capabilities. Several other new or updated features have also been added to FESWMS Version 3. These include a storm feature (to generate hurricane wind fields) and storativity, which is an alternative approach to element wetting and drying. The wind field generated by the storm feature produces an additional shear stress on the surface of the water. For wetting and drying, rather than turning off an element that has a dry node, storativity treats the element as if there were a slot extending below ground. Using the storativity feature causes the element to stay wet but the slot produces only a negligible amount of additional conveyance. Therefore, the element does not actually "dry" and numerical instabilities resulting from drying and rewetting are avoided.

FESWMS Version 3 has been under development for FHWA for Dr. David Froehlich. The model has been supplied to the Research Team by FHWA although it is currently for FHWA review and not for general release. During the preparation of the Interim Report, Appendix A of the FESWMS Users Manual and an updated Version 3 have been supplied to the Research Team by FHWA. Appendix A of the FESWMS manual contains data record formats. The model testing focused on the bridge structure features, dynamic mode operation, wind features, wetting and drying, and storativity.

As with Version 2, FESWMS Version 3 works in dynamic mode. It appears, however, that the program skips the initial time step. This would typically not cause erroneous results later in the simulation. If the correct initial condition is desired, then a separate steady state model can be run for the starting conditions and the results used as an input initial condition for the dynamic run. Standard element wetting and drying works in dynamic mode. This correction alone significantly increases the model's applicability to tidal applications. The storativity feature was also tested in steady-state and dynamic modes.

Although the hydraulic computations are expected to be slightly different when this feature is used, the differences should be negligible. Only a small amount of additional conveyance is added to each element when storativity is used. This feature keeps portions of the model active when standard wetting and drying would turn off "dry" elements. In the steady-state test model, the results were virtually indistinguishable between otherwise identical models with and without the storativity feature. In the dynamic test model, differences were significant between the models run with and without storativity. In areas well away from shallow flow, velocity differences between the models were typically five percent and often greater than 25 percent. Until these differences are resolved, the storativity features should probably not be used in dynamic runs.

Bridge pressure flow continues to be problematic in FESWMS Version 3. Even for relatively simple cases, numerical instability causes the model to diverge when the water surface reaches the bottom of a bridge deck, even for relatively low discharges and flow velocities. It appears that for very simple geometries the model can converge on a solution when lower than normal relaxation factors are used. The relaxation factor controls how much of the computed change in flow will be allowed from one iteration to the next. For example, if the model computes a 1 ft/s change in velocity at a node for the next iteration (within a time step), a relaxation factor of 0.6 would then limit the change to 0.6 ft/s. When the relaxation factor is used, numerical stability can be increased. If convergence criteria are used to limit the number of iterations, these should also be reduced by the relaxation factor.

The weir, pier and culvert routines were also tested. These features appear to work as well as in Version 2. For culvert hydraulics, the model does not automatically distinguish between inlet and outlet control. If invert elevations are not input, then the model computes outlet control flow and inlet control flow is computed if invert elevations are input. For dynamic modeling it is recommended by the Research Team that outlet control be assumed for the entire run (invert elevations not input). Although the full area of the culvert is used in either case (full flow for outlet control and the total opening area for inlet control), the use of outlet control is less likely to significantly over estimate the amount of flow through the culvert.

3.2 UNET Version 4.0 and HEC-RAS Version 3.0

Formal testing of UNET Version 4.0 (Barkau 1997) was not performed as part of this phase because it was an update of previously tested versions. The update was to include metric computation. During this project, one bug has been discovered by the Research Team and the Army Corps of Engineers Hydrologic Engineering Center (HEC) has corrected the code. The bug occurred when using the normal bridge method and resulted in an occasional, and typically minor, error in computing the bridge opening area for the right overbank under a bridge.

During Phase III, HEC-RAS Version 3.0 was released by the USACE Hydrologic Engineering Center to include UNET unsteady flow routes. Testing of HEC-RAS Version 3.0 was performed by the research team when it was released as a Beta test version prior to general release. The model was compared with UNET and only minor differences were apparent in the results. The research team has subsequently used HEC-RAS successfully on a tidal bridge scour evaluation project for Florida DOT District 7. The combination of the UNET unsteady flow routines with the HEC-RAS graphical user interface produces an excellent platform for 1-dimensional tidal modeling.

4. PHASE III MANUAL AND TRAINING COURSE DEVELOPMENT

4.1 Manual Organization

In Phase I, several models were identified as suitable for tidal modeling. These included UNET (Barkau 1997), DYNLET (Cialone and Amein 1993), FESWMS (Froehlich 1996) and RMA-2V (USACE 1997). UNET and FESWMS were incorporated into the training and users manual (Zevenbergen et al. 1997b) developed in Phase II. However, all the models listed above and other dynamic models could be used for tidal applications. For a model to be applicable to tidal hydraulics, it must have dynamic flow routing capabilities, include variable flow and stage boundary conditions, allow reversible flow computations and incorporate adequate structure hydraulics. In view of the fact that several models could be used, the manual provides general information on tidal modeling for bridge hydraulics. It also includes specific information on the use of HEC-RAS as this model is widely available and is an excellent platform for tidal hydraulic modeling.

The manual, entitled "Tidal Hydraulic Modeling for Bridges," provides background on tides, storm surges, boundary condition development, model selection and data sources. Sections are also included on the related topics of tidal scour and wave height prediction. The manual also includes a chapter on the use of HEC-RAS for tidal modeling. **Figure 4.1** is the outline for this manual. The manual incorporates and expands on the material from the manual developed in Phase II.

4.2 Training Course Organization

Training was presented at South Carolina DOT computer training facilities. The training included lessons on tide and storm surge processes and tidal waterway hydraulics using HEC-RAS. A participants workbook was developed for the course. The manual outlined above was the reference manual for the course. The lessons used PowerPoint presentations, overheads, and flipcharts and the workshops applied the techniques to develop geometric and boundary condition input for a simple estuary problem. **Figure 4.2** is the lesson schedule and learning objectives for the UNET tidal hydraulics training course.

TIDAL HYDRAULIC MODELING FOR BRIDGES

1. Introduction

1.1 Purpose
1.2 Manual Organization
1.3 Acknowledgements

2. Tidal Terminology and Theory

2.1 Tidal Waterways
2.2 Tides
2.3 Tidal Currents
2.3 Hurricanes and Storm Surges
2.4 Waves
2.5 Apparent Ocean Level Rise
2.6 Datums

3. Boundary Conditions

3.1 Tides
3.2 Storm Surges
3.3 Combining Tides and Storm Surges
3.4 Upland Runoff
3.5 Wind Effects

4. Available Tidal Hydraulic Modeling Techniques

4.1 Overview
4.2 Tidal Prism Approach
4.3 Orifice Approach
4.4 Routing Approach
4.5 Dynamic Modeling
4.6 Method Selection
4.7 Model Extents

5. Available Tidal Hydraulic Modeling Techniques

5.1 Definitions
5.2 Methodology
5.3 Network Layout
5.4 Geometric Input
5.5 Bridges, Culverts, and Other Hydraulic Structures
5.6 Boundary Conditions and Run Control
5.7 Trouble Shooting

6. Tidal Scour

7. Wave Height Computations

8. Data Sources

Figure 4.1. Tidal Manual Outline.

TIDAL HYDRAULICS TRAINING COURSE

Course Objectives: Upon completion of this course, Participant will be able to:

1. Describe tide and storm surge processes
2. Identify appropriate modeling approaches for bridges in tidal waterways
3. Prepare networks and geometry input for HEC-RAS models
4. Generate upstream and downstream boundary conditions for tides and storm surges
5. Incorporate bridge, culvert and embankment geometry into the models.
6. Compare results from different model simulations
7. Compute time dependent scour
8. Calculate wave heights in bridge openings

Day 1

Lesson 1: Course Introduction (PowerPoint, Flip-Chart)

During this lesson, the instructors and participants will identify their goals and objectives for the course and will become familiar with the course materials.

Lesson 2: Tidal Processes (Lecture, PowerPoint)

At the end of this lesson, the participants will be able to identify the causes of tides and the causes of tide variability.

Lesson 3: Hurricane Processes (Lecture, PowerPoint, Workshop)

Upon completion of this lesson, the Participants will be able to generate storm surge hydrographs and combine them with daily tide hydrographs.

Lesson 4: Introduction to Dynamic Modeling with HEC-RAS (Lecture, Hands-On Computer session)

Upon completion of this lesson, the participants will able to review input and output files in HEC-RAS.

Figure 4.2. Training Course Lessons.

Day 2

Lesson 5: HEC-RAS network and Geometric Input (Lecture, Powerpoint)
Upon completion of this lesson, the participants will be able to prepare network layouts for multi-reach channels and enter cross-section data.

Workshop 5: Geometry File Workshop Problem (Hands-on Computer session)
Participants will demonstrate the skills learned in Lesson 5 with a simple multi-reach channel system.

Lesson 6: Boundary Conditions (Lecture, Powerpoint)
Upon completion of this lesson, participants will be able to prepare boundary condition files for HEC-RAS.

Workshop 6: Boundary Condition Workshop Problem (Hands-on Computer Session)
Participants will apply the information in Lesson 6 by preparing boundary condition files for the network developed in Workshop 5. The participants will run the model, review the results, and compare the results for different input conditions.

Day 3

Lesson 7: Structures (Lecture, Powerpoint)
Upon completion of this lesson, participants will be able to identify the various structure methods for bridge and culvert geometry input data.

Workshop 7: Structure Workshop Problem (Hands-on Computer Session)
Participants will apply the information in Lesson 7 by inserting a structure into the files developed in Workshop 7. The participants will run the model and review the output.

Lesson 8: Scour Computations Workshop (Lecture, Hands-on Spreadsheet Application)
Upon Completion of this lesson, participants will be able to identify the data required for time dependent scour calculations and enter the data into the scour prediction spreadsheet.

Lesson 9: Wind Waves Workshop (Lecture, Hands-on Spreadsheet Application)
Upon Completion of this lesson, participants will be able to identify the data required for wave height calculations and enter the data into the wave height prediction spreadsheet.

Lesson 10: Course Summary and Critique

Figure 4.2. Training Course Lessons (continued).

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