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Tidal Bridge Scour in a Coastal River Environment: Case Study

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ABSTRACT

As part of an innovative design/build project a data-rich hydraulics and scour analysis was performed for the proposed 5 kilometer long Washington Bypass Bridge over the Tar River in Washington, NC. This analysis featured: 1) a current and stage monitoring program, 2) historical aerial photograph analysis, 3) extensive long term bed elevation study, 4) debris scour evaluation, 5) variable skew angles due to spatial and temporal changes in flow characteristics, 6) complex pier analysis, and 7) use of a two-dimensional (2D) hydrodynamic model (TABS RMA-2) to evaluate riverine flooding and hurricane surge hydraulics causing extensive wetting-and-drying.

The model was subjected to a comprehensive two-step model calibration/verification process. Low-flow conditions were calibrated and verified using project-collected Acoustic Doppler Current Profiling (ADCP) and tidal gage monitoring. Boat-mounted ADCP measurements collected by USGS during hurricane surge (Hurricane Dennis) and rain-induced flooding (Hurricane Floyd) were used for high-flow calibration and verification. This was a rare opportunity for a bridge designer to be able to evaluate scour using a sophisticated hydrodynamic model that was calibrated with field data collected during an event that represented the design conditions.

Two-dimensional hydrodynamic modeling was used to simulate complex hydraulics of the project site located at the end of the 8,300 square kilometer watershed which is tidally influenced. Due to the size of the upstream drainage area and the proximity to the open ocean both rain-induced-flow and storm-surge scenarios were considered.

INTRODUCTION

AECOM was hired by Flat Iron – United Joint Venture (FLUNJV) to perform engineering services relating to the hydraulics and scour analysis for the Washington Bypass Bridge (BIN 353-US17 over the Tar River) in Washington, NC (Figure 1). Under this agreement, AECOM developed a 2-dimensional hydrodynamic model (RMA-2) to evaluate the flow depth and velocity for the 100-, and 500-year storm events. The predicted velocity and depth information from these events were used to calculate scour depths at the bridge and assist in designing the bridge substructure units to withstand scour.

As part of this study, AECOM designed and oversaw the required data collection performed by Ocean Surveys, Inc. (OSI) which was used for the calibration and verification of the hydrodynamic model. The data collection occurred during a Spring Tide and involved three (3) tide gages, one (1) “upward looking” Acoustic Doppler Current Profiler (ADCP), and ADCP transects of cross sections at and either side of the bridge.

Figure 1. Project location.

Due to the size of the upstream drainage area (8,300 sq. km) and the proximity to the open ocean (i.e. to Atlantic Ocean through Pamlico River and Pamlico Sound), both rain-induced-flow and storm-surge scenarios were considered separately as indicated by HEC-25 (1st Edition, Section 2.8 Page 2.31).

Local pier scour was calculated for each of the 127 pile bents (piers) for 8 different conditions: 2 (rain/surge) x 2 (100-year/500-year) x 2 (debris/no-debris) = 8. Upon evaluating the results, scour calculations were determined to be sensitive to debris. Therefore, a debris accumulation potential evaluation per HEC-9 3rd Edition

(Debris Control Structures – Evaluation and Countermeasures) was performed for each bridge component. Contraction scour caused by the proposed bridge, which is scour due to contraction of the flow's conveyance area caused by the bridge structure and its approaches, was also calculated.

In addition to local and contraction scour, long-term bed elevation change was analyzed. The analysis consisted of three subtasks. The first subtask utilized historical aerial photographs of the area to digitize the shoreline and then to observe the evolution of the shoreline changes in time. The second subtask was obtaining various historical bathymetric surveys conducted at the US 17 Bridge crossing, located about 1.6 km downstream of the Washington Bypass Bridge, and calculating the vertical changes of the river bottom. The third subtask was the channel stability assessment performed by geomorphologist Prof. Stanley Riggs of Eastern Carolina University, who is an expert on the riverine geomorphology for this site.

METHODOLOGY

In order to estimate the site-specific detailed hydrodynamic characteristics at the Washington Bypass Bridge it was necessary to construct a 2-dimensional hydrodynamic model. Hydrodynamic modeling was accomplished using the Surface Water Modeling System (SMS) in conjunction with RMA-2.

Model Domain

The model domain was defined considering the area of interest, location of available data sources, and the limitation on computational resources. The area of interest was confined to the vicinity of the bridge crossing. In order to obtain an adequate solution at the area of interest, model boundaries were established distant from the area of interest and where USGS station locations were available as data sources.

For the storm surge scenario the downstream model boundary needed to be moved further downstream (35 km downstream of Pamlico USGS gage) to utilize available data stations.

The model domain was meshed using triangular and rectangular elements. The approximate number of elements in the meshes used ranges from 9,000 to 23,000.

Calibration and Verification

The RMA-2 model is calibrated primarily with two (2) parameters: the Peclet Number (Pe) and the Manning Roughness Coefficient (n), and was performed in two stages. In the first stage, the detailed information collected by project survey team was used to calibrate only the model cells representing the channel. Calibration with this detailed data was limited to the channel because during the data collection period, water was confined to the channel. In the second stage, an extreme event was used to calibrate the overbank areas (i.e. wetlands and other floodplain areas) while they are exposed to flow. The data records collected at the US 17 crossing by USGS during Hurricane Dennis in 1999 were used for this purpose.

The channel calibration of the model was performed with the available data and the data sets collected during the project's monitoring period. The point in-situ

ADCP and the project's Tide Gage No. 1 (TG 1) were used to compare the actual data with the model results.

Overbank calibration of the model was performed using data collected by the USGS at the Pamlico Gage. Input variables for the model included WSE from the Pamlico Gage and model elements account for storage in watershed (which were used beyond the Tranter's Creek and Grimesland boundaries). Calibrated channel properties were kept the same and the overbank depth varying Manning's n roughness was used for calibration.

Verification of the model was performed in two (2) ways. First, for low flow conditions flow measurements at Pamlico gage were compared to model results. Second, for high flow conditions USGS ADCP velocity transects data at the maximum velocity conditions during Hurricane Floyd at 45 meters upstream of US 17 were compared to model predictions.

Calibrated model prediction of Hurricane Dennis compared to flow measurements taken at the USGS Pamlico Gage is shown in Figure 2.

Figure 2. Calibrated model simulation of Hurricane Dennis.

RESULTS

Once the model was calibrated and verified, two different scenarios and two different return intervals totaling four (4) flow cases were developed: a rain-induced-flow scenario and a storm-surge scenario, for 100- and 500-year storms.

Rain Induced Flow Scenario

As seen during Hurricane Floyd, a hurricane causing significant rainfall over an already saturated ground can cause significant flooding in Tar/Pamlico watershed. Another observation made during Floyd was the duration of the peak flow. Unlike the tidal surge, which has a peak that passes within a few hours (i.e. dynamic), Hurricane

Floyd's peak was sustained for days (i.e. steady). The duration is important for scour in cohesive sediment (such as this case), as equilibrium scour conditions require time to develop. FEMA FIS (Reference 2) was used to determine peak flow values for the model boundaries. Tar River peak flows were estimated using the flows represented in the FIS immediately downstream of the Bear Creek confluence, and the Tranter's Creek boundary flows were also read from the FIS upstream of the confluence with Tar River. Table 1 shows the peak flow values used for the rain induced flow cases (i.e. Case 1 and 2).

Table 1. Upstream Boundary Peak Flow Conditions in Cubic Meters per Second (cms)

	Tar River	Tranter's Creek
Case 1 (100-year)	1,550	310
Case 2 (500-year)	2,200	485

The downstream boundary condition was the WSE at the USGS Pamlico gage. The WSE for both cases was calculated using a stage-flow relationship developed from the data collected at the gage. Only high flows (i.e. >283 cms) in ebb direction were considered while generating the relationship.

In general, velocities are lower on the overbanks and higher in the channel and highest around the thalweg as expected.

Hurricane Surge Scenario

Upon establishment of the typical tide, design hurricane surges were developed using the Pooled Fund Study (Reference 5) that was coordinated by the SCDOT and Boundary Conditions for Bridge Scour Analysis (Reference 6) by the NCDOT. From the latter study, a peak storm surge value (S_p) of 3.4 meters was obtained for the 100-year case at Station No. 3. In order to utilize the data from the Station No. 3, the downstream model boundary was moved to the location (latitude 35.4, longitude 76.72) that corresponds to the Station No. 3. The same data sources used to create the calibrated model (i.e. shoreline, bathymetry, topography, etc.) were also used to extend the model. Roughness and turbulence properties developed for the calibrated model were also applied to the extended section.

Similar wetting/drying issues that were encountered during the NCDOT Study (Reference 6) were also applicable to the 2D model used during this analysis. Due to the large floodplain-to-channel ratio in the area (>8), most of the numerical model cells were dry except for a short duration during the entire simulation. The abrupt wetting/drying due to the dynamic surge causes numerical instability in the 2D model. Therefore, the marsh porosity approach, available through RMA2, was utilized. Marsh porosity gradually decreases the conveyance capacity when the WSE drops below the average ground elevation, providing a smooth transition between wetting and drying, which increases the numerical stability of the model.

Storm surge scenario events, both 100-year and 500-year, were simulated using the extended model. Both synthetic storm surges (100-year 6,500 cms and 500-year 12,900 cms) are significantly greater than the observed 2,600 cms peak flow magnitude of Hurricane Dennis.

The direction of the flow for the storm-surge scenario deviated from the average direction (observed from the field data collection) as the velocities increased. The deviation was observed to be 8 degrees during ebb times (i.e. rain-induced-flow scenario) and 20 degrees during flood times (flood periods produce peak velocity/depth combinations).

Figure 3 shows peak velocity magnitude and direction for the 100-year storm-surge scenario. Color-coding indicates velocity magnitude and the vectors are scaled to magnitude. In general, velocities are lower on the overbanks and higher in the channel and highest around the thalweg as expected. One exception to that is the bents located adjacent to Kennedy Creek experience increased velocities compared to other overbank bents due to the relatively lower friction path provided by Kennedy Creek during flood stages of the surge.

Figure 3. 100-year storm surge depth averaged velocities (red indicates 3 m/s and blue indicates 0 m/s).

Scour Analysis

Scour evaluation and calculations were performed in accordance with the guidelines set forth in the most current editions of the Federal Highway Administration's Hydraulic Engineering Circular (HEC) HEC-20 3rd Edition (Stream Stability at Highway Structures), HEC-18 4th Edition (Evaluating Scour at Bridges),

and HEC-25 1st Edition (Tidal Hydrology, Hydraulics and Scour at Bridges). The hydraulic data required for the scour analysis were extracted from the 2-D model, the soil data were supplied by Mactec Inc., and the preliminary bridge geometry was taken from AECOM's preliminary bridge design plans. All equation constants and coefficients used in the analysis were taken from literature. The following four components of total scour were calculated: aggradation and degradation, general scour, local scour, and lateral stream migration.

Based on the results of the 2D modeling of peak flows caused by rainfall and peak flows due to storm surge hurricane surge conditions (landward flow) produced the worst-case design scour conditions at the proposed bridge location.

Analysis of Long-Term Bed Elevation Change

The analysis consisted of three subtasks. The first subtask involved utilizing historical aerial photographs of the area to digitize the shoreline and observe the evolution of the shoreline changes in time. The second subtask involved obtaining bathymetric surveys conducted at the US 17 Bridge crossing, located about 1.6 km downstream of the Washington Bypass Bridge, and observing the vertical changes on the river bottom. The third subtask was a channel stability assessment performed by riverine geomorphology expert Prof. Stanley Riggs of Eastern Carolina University, who has specific knowledge of the project site.

The historical aerial photograph analysis was performed by digitization of the shoreline in the vicinity of the proposed bridge crossing and then the subsequent observation of the resulting pattern and rate of the channel movement. Analysis showed that no significant changes have occurred over the past 50 years.

The channel stability assessment performed by Prof. Riggs indicated that the Tar River in the vicinity of the Washington Bypass Bridge has been stable vertically (i.e. the channel bed has not experienced significant sediment erosion or accumulation) and laterally (i.e. the channel has not migrated significantly). Considering the current rate of change, significant degradation is not expected.

The conclusion by Prof. Riggs is that the long-term scour component at the site is negligible (i.e. the long-term component of scour is 0 m). However, the analysis indicated that there may be a lateral shift of 0.3 m/year. Therefore, considering the design life of the bridge is 100 years, it was recommended that closest overbank bents to the channel be designed as channel bents.

Scour at Abutments

Per the preliminary bridge design plans, the abutments were determined to be outside of the flood elevation, and were not analyzed further for scour.

Computation of the Magnitude of Local Scour at Piers

Local scour at the proposed Washington Bypass Bridge was computed for hurricane storm surges with recurrence intervals of 100 years and 500 years using the CSU equation as presented in HEC-18. Both simple and complex pier calculations were utilized in calculation of the pier scour as explained below. The hydrodynamic model time step causing the highest scour was used.

During the field inspection, it was determined that the fallen, or eroding, vegetation located at the edge of the channel could potentially become a source of debris that could collect at the channel bents and cause increased scour. However, the established swamp forest in the overbanks acts as a debris deflector for the overbank bents. Debris, therefore, is not expected to be a critical factor for the bents located on the overbanks. The simple pier equation was found to be more suitable for bents in the channel due to its conservative assumption of a solid pier and ability to account for full blockage by debris as described in HEC-18. However, for bents in the overbanks, which have a sufficient forest buffer protecting them from debris, the complex pier equation was found to be more suitable.

A summary of the calculated scour depths is shown in Table 2. For the channel bents, considering that the thalweg in the channel can migrate, the largest scour value (lowest scour-hole bottom elevation) calculated should be used for design of each channel bent. Bents on the overbank which lie outside the potential 30 meters of channel migration should be designed to withstand the smaller calculated scour depths in these areas.

Table 2. Scour Depth Summary Table (m)

	Pile Bent Numbers		
	1 through 81	82 through 99	100 through 127
	Southern Overbank	Channel	Northern Overbank
Storm Event	Avg. (min, max)	Max.	Avg. (min, max)
100-year	3 (1.5, 5.5)	8.2	2.7 (1.5, 4.3)
500-year	4.3 (1.8, 7)	9.1	4 (1.8, 5.5)

CONCLUSION

Adequate data was a key in preparing a comprehensive scour analysis. Obtaining this data involved engaging relevant resources (NCDOT, USGS, Eastern Carolina University, etc.) at the early stages of the project. Especially, the importance of long-term ADCP data and how it can improve the confidence level of sophisticated hydrodynamic model simulations became apparent.

By limiting complex scour equations to simple scour equation, over estimation of scour at the extreme values could be minimized.

A comprehensive long-term aggradation/degradation analysis can indicate maximum expected river bed shift for the design life of the bridge, which minimizes the number of piers to be treated as a channel pier instead of an overbank pier.

Over estimation of scour due to debris accumulation can be minimized by identifying only the piers that are subject to accumulation.

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