Coastal Engineering Assessment of Storm-Induced Scour Problem for Proposed Indian River Inlet Bridge

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March 29, 2004

Table of Contents

Introduction

Figg Bridge Engineers, Inc. (2003) evaluated the scour problem associated with the proposed new bridge spanning the Indian River Inlet using a hydraulic engineering approach based on HEC-18 (Federal Highway Administration 1997). It was recommended that a coastal engineering study should be conducted to examine the scour problem because the Indian River Inlet is not a river environment. The Delaware Department of Transportation requested the Center for Applied Coastal Research, University of Delaware perform such a study. The coastal group of the Department of Civil and Environmental Engineering, University of Delaware, previously conducted coastal engineering studies of the Indian River Inlet (Lanan and Dalrymple 1977; Dalrymple et al. 1983).

The history of engineering projects for the Indian River Inlet was summarized by Lanan and Dalrymple (1977). The stabilization project for the Indian River Inlet was approved in 1937. Two jetties were constructed of steel sheet piling which was driven to a depth of 18 ft below mean low water. The base of the sheet piling was protected by a layer of riprap that was about 3 ft thick and 6 ft wide. In 1941, the steel sheet bulkheads were extended westward to stop bank erosion. In 1963, the bulkheads were extended again westward and the sheet pile was reinforced with riprap. The top portion of the sheet piling was removed and replaced with stone and capped with concrete. The first and second spans of the present bridge were constructed in 1965 and 1976, respectively.

The worst storm to affect the Indian River Inlet was the northeaster of March 6-8, 1962. This storm produced large waves and a storm surge that lasted over five consecutive high tides during a period of unusually high astronomical tides. The combined storm tide was 7.9 ft above mean sea level (approximately 8.26 ft above NGVD 1929) at Breakwater Harbor, Delaware. The offshore wave heights were 20 – 30 ft. The storm severely eroded the beach and breached the barrier beach in several places between Dewey Beach and Indian River Inlet. More recently, the 1991 Halloween Storm, whose recurrence interval was estimated to be approximately 10 yr, caused a 5.5-ft storm tide above NGVD 1929 at Ocean City, Maryland and a 4.0-ft storm tide on Little Assawoman Bay in Fenwick Island (Johnson et al. 1994).

The present bridge constructed in 1965 and 1976 did not experience the 1962 storm. Table 1 lists the encounter probability P for the last 39 years against the storm whose recurrence interval is T_r yr where P is estimated as

$$P = 1 - \left(1 - \frac{1}{T_r}\right)^{39} \qquad \text{during 39 years} \tag{1}$$

Table 1 suggests that the experiences of the present bridge may not indicate what may happen to the new bridge during future extreme storms.

Table 1. Encounter Probability P against T_r-yr Storm

T_r (yr)	10	50	100	500
P	0.98	0.55	0.32	0.08

2. Design Water Depth and Free Surface Elevation Difference

Currents in the Indian River Inlet are caused predominantly by the hydrostatic pressure difference associated with the free surface elevation difference between the Atlantic Ocean and Indian River Bay. This free surface elevation difference is induced mostly by astronomical tides under normal conditions and by combined storm surge and tide during storms. The sea level rise in the ocean and bay is approximately the same and does not increase the free surface elevation difference. However, the future sea level rise will increase the water depth in flooded areas. The sea level rise based on the tide data at Cape Henlopen was 1 ft per century as quoted by Lanan and Dalrymple (1977).

Table 2 lists the peak Stillwater elevations due to the storm tide with the recurrence interval T_r = 10, 50, 100 and 500 yr. These elevations are obtained from the tables in pages B-25 and B-26 of the scour elevation report by Figg Bridge Engineers, Inc. (2003). For the 500-yr storm, the elevation difference between the ocean and bay was regarded to be negligible because the barrier beach was assumed to be breached extensively during the storm. The peak stillwater elevations in the ocean and bay may not occur simultaneously and the elevation difference varies during a storm. Consequently, the elevation difference listed in Table 2 may not be the maximum elevation difference during the storm. On the other hand, the flood insurance rate map number 1005C0505F in page B-9 of the report by Figg Bridge Engineers, Inc. (2003) shows that the peak stillwater elevation above NGVD, 1929 is 11 ft and 9 ft in the Indian River Inlet and Indian River Bay, respectively. These elevations and the elevation difference of 2 ft are higher than those for T_r = 100 yr in Table 2.

Table 2. Peak Stillwater Elevations in Ocean and Bay

Recurrence Interval T_r (yr)	Peak Stillwater Elevations (ft) above NGVD 1929		Elevation Difference (ft)
	Atlantic Ocean between Dewey Beach and Bethany Beach	Indian River Bay Entire coastline	
10	6.4	4.7	1.7
50	8.0	6.4	1.6
100	8.9	7.5	1.4
500	10.8	10.8	0

The stillwater elevation due to the combined storm surge and tide does not include the increase of the mean water level caused by breaking waves pushing water against the sloping beach. This mean water level increase called wave setup is about 10% of the offshore significant wave height (Kobayashi et al. 2003). For the offshore significant wave height of 20 – 30 ft, which is typical for severe storms, the corresponding wave setup is about 2 – 3 ft. It may be noted that the uncertainty or error for the storm surge prediction is about 20% and of the same order of magnitude of wave setup. Furthermore, the storm tide data in flooded areas used to calibrate the storm tide numerical model includes wave setup implicitly.

The scour evaluation report by Figg Bridge Engineers, Inc. (2003) uses the datum of NAVD 88 which is 0.78 ft above the datum of NGVD 1929 as stated in page A-1 of the report. The land elevation of the arch base and bridge abutment is approximately 5 ft above NAVD 88 and 5.78 ft above NGVD 1929. Considering the uncertainties of the storm tide and wave setup, the design water depth h at the arch base and bridge abutment is taken as

$$\text{Design water depth} \quad h = 3, 5 \text{ and } 7 \text{ ft} \tag{2}$$

for the recurrence interval T_r = 50, 100 and 500 yr, respectively. For the storm with T_r = 10 yr, the land adjacent to the Indian River Inlet may barely be flooded. This is consistent with the experience during the 1991 Halloween Storm (Johnson et al. 1994).

On the other hand, the design free surface elevation difference E between the ocean and bay may change little for the different recurrence intervals in light of those shown in Table 2. The measurements using tide gauges performed by Lanan and Dalrymple (1977) in November 1975 indicated E = 1.3 ft during flood and ebb tides. The measured tidal currents were generally less than 5 ft/s but the measured maximum surface current was 7.85 ft/s in November 1975. The vertical current profiles measured on June 11, 1983 by Dalrymple et al. (1983) were approximately logarithmic and the current velocities normally decreased downward. The measured maximum current velocity was 6.1 ft/s. The design elevation difference E is taken as the upper limit of the available values discussed above

$$\text{Design elevation difference} \quad E = 2 \text{ ft} \tag{3}$$

where it will be shown later that the estimated depth-averaged or area-averaged current velocity U is proportional to $\sqrt{E}$ and is not very sensitive to E in the range of E = 1.5 – 3 ft because $\sqrt{1.5}/\sqrt{2} = 0.87$ and $\sqrt{3}/\sqrt{2} = 1.22$.

3. Currents and Waves during Storms

The current velocity U in the Indian River Inlet is estimated using the hydraulic formula for tidal inlets in the Coastal Engineering Manual (2003) which is rewritten as

$$E = \left(k_{en} + k_f + k_{ex}\right)\frac{U^2}{2g} \quad ; \quad k_f = \frac{2gn^2L}{1.49^2 R^{4/3}} \tag{4}$$

where E = head difference between the ocean and bay which is approximated by the free surface elevation difference; k_{en} = entrance energy loss coefficient of the order of 0.1; k_f = frictional energy loss coefficient; k_{ex} = exit energy loss coefficient of the order of 1.0; U = current velocity averaged over the inlet cross section; g = gravitational acceleration (g = 32.2 ft/s^2); L = inlet length associated with the bottom frictional energy loss; R = hydraulic radius of the inlet cross section; and n = Manning's roughness coefficient in English units. The Coastal Engineering Manual (CEM) indicates n = 0.02 – 0.03 but Lanan and Dalrymple (1977) obtained n =0.029 – 0.045 using the measured values of E and U at the Indian River Inlet.

Use is made of k_{en} = 0.1, k_{ex} = 1.0, n = 0.035 and L = 2,000 ft in the following calculations. The hydraulic radius R varies along the inlet but a typical value is $R \approx$ 50 ft. For these assumed values, k_f = 0.39 and $(k_{en} + k_f + k_{ex})$ = 1.49, indicating the dominance of the exit energy loss. Under normal tidal conditions, $E \approx$ 1 ft and Eq. (4) yields $U \approx$ 6.6 ft/s, which is consistent with the measured current velocities by Lanan and Dalrymple (1977). For the design condition of E = 2 ft in Eq. (3), U = 9.3 ft /s. The current velocity U does not increase significantly because U is proportional to $\sqrt{E}$.

It is necessary to predict the depth-averaged current velocity U on the flooded land where the design water depth h is specified in Eq. (2). Eq. (4) is assumed to be applicable if the hydraulic radius R is replaced by the water depth h. The roughness coefficient n on the flooded sandy land may be smaller than the value of n = 0.035 used for the inlet protected with riprap but the presence of wind waves may increase the roughness felt by the current because of the nonlinear wave and current interaction with the bottom. As a result, the only change made in Eq. (4) is the replacement of R by h. Table 3 lists the calculated values of k_f and U for h = 3, 5 and 7 ft where the frictional energy loss is dominant on the flooded area because of the small depth. The direction of the current velocity U is assumed to be parallel to the inlet alignment although the faster current in the inlet entrains the slower current on the flooded area.

Table 3. Design Current Velocity *U* and Wave Height *H*

Water depth h (ft)	3	5	7
Frictional loss coeff. k_f	16.4	8.3	5.3
Current velocity U (ft/s)	2.7	3.7	4.5
Wave height H (ft)	2.3	3.9	5.5

The design wave height H is simply taken as H = 0.78 h which was used to assess the effects of wind waves for the flood insurance zoning (e.g., Johnson et al. 1994). Kobayashi et al. (2003) showed that this simple formula might overpredict the significant wave height on a beach with a very gentle slope. It is noted that wave heights in wind waves vary statistically and the significant wave height is the average of the highest 1/3 waves. For the flooded area adjacent to the inlet, offshore waves may propagate through the deeper inlet and increase the wave height on the flooded area. Consequently, the use of H = 0.78 h does not necessarily give the upper limit of wind waves. The calculated wave height H = 0.78 h is listed in Table 3.

4. Sediment Transport and Scour

Before the assessment of scour around the arch base and bridge abutment, the rate and volume of sediment transport during the storm is estimated using the values of U and H in Table 3 which do not account for the arch base and bridge abutment. The following estimate of the sediment transport rate should be regarded as an order-of-magnitude estimate because sediment transport under combined breaking waves and currents in the surf zone on beaches is not well understood at present. The sediment transport formulas for nonbreaking waves in the Coastal Engineering Manual (2003) were extended to breaking waves in the surf zone by Madsen et al. (2003) where Madsen wrote this chapter in the CEM. The extended formulas are rearranged here to simplify the resulting equations.

The maximum near-bottom wave orbital velocity u_m and the maximum wave-associated bottom shear stress τ_m are given by

$$u_m = \frac{1}{2}\frac{H_{rms}}{h}\sqrt{gh} \quad ; \quad \tau_m = \rho C_d u_m^2 \tag{5}$$

where H_{rms} = root-mean-square wave height associated with wave energy and given by $H_{rms} = H/\sqrt{2}$ with H = significant wave height; ρ = density of seawater (ρg = 64 pounds per cubic foot); and C_d = surf zone bottom friction coefficient of the order of 0.01.

Assuming that the waves propagate in the same direction as the current, the volumetric bed load (sediment particles moving near the bed) transport rate q_b per unit width is estimated as

$$q_b = 9\sqrt{\frac{\tau_m}{\rho}}\frac{\tau_c}{\rho g(s-1)} \quad ; \quad \tau_c = \frac{\rho g n^2 U^2}{1.49^2 h^{1/3}} \tag{6}$$

where τ_c = current-associated bottom shear stress estimated here using Manning's formula to be consistent with Eq. (4); and s = specific gravity of the sediment ($s \simeq 2.6$). The suspended sediment transport rate q_s per unit width is predicted by

$$q_s = ChU \quad ; \quad C = 8.7\times10^{-6}\left[\frac{C_d u_m^2}{0.05\, g(s-1)d} - 1\right]\frac{(\tau_c/\rho)^{0.5}}{w_f} \tag{7}$$

where C = depth-averaged volumetric suspended sediment concentrations; d = median sand diameter; and w_f = sand fall velocity. The fine sand in the vicinity of the Indian River Inlet may be characterized by d = 0.2 mm and w_f = 2 cm/s.

Table 4. Predicted Sand Transport Rates

Water depth h (ft)	3	5	7
Bed load q_b (ft^2/hr)	15	31	49
Sand concentration C	0.0017	0.0036	0.0058
Suspended load q_s (ft^2/hr)	49	238	653
Total load q_t (ft^2/hr)	64	269	702

Table 4 lists the computed values of q_b, C, q_s and $q_t = (q_b + q_s)$ for h = 3, 5 and 7 ft. The total sand transport rate q_t is the sum of the bed load and suspended load transport rates. The suspended load is dominant under breaking waves. The depth-averaged suspended sand concentration is of the order of 0.004 (sediment volume = 0.4% of the volume of water and sand mixture). This sand concentration is much less than the sand concentration of 0.04 measured for wave overwash of dunes by Kobayashi et al. (1996). The total load q_t = 269 ft^2/hr corresponds to the volume of the deposited sand (porosity □ 0.35) in the area of 1-ft width and 200-ft length with the depth of 2.1 ft. If no sand is transported into this area, the vertical erosion rate is 2.1 ft/hr. The 200-ft length may be regarded as the typical length along the inlet affected by the presence of the arch base and bridge abutment. The actual erosion rate is less than 2.1 ft/hr because sand is transported into this area. For the 100-yr event considered within the flood insurance program, the dune erosion area of 538 ft^2 (50 m^2) per unit width was adopted (Hallermeier and Rhodes 1988). If this area is assumed to be removed by q_t = 269 ft^2/hr, it takes only two hours. The 2-hr duration is very short but the peak of hurricanes may last about 3 hr (Kobayashi et al. 2003). In short, the predicted sand transport rates shown in Table 4 are uncertain but appear to be realistic in terms of the sand volume transported under the combined action of breaking waves and currents during severe storms.

The rule of thumb and simplistic empirical guidance for the prediction of coastal scour problems in the Coastal Engineering Manual (2003) are not applicable directly to the present scour problem as explained in the following. For the case of normally incident breaking waves with no currents, the maximum scour depth at a vertical wall is of the order of the nonbreaking wave height that can be supported by the water depth at the structure. This rule of thumb implies that the maximum scour depth is of the order of the wave height H = 2.3 – 5.5 ft listed in Table 3. It should be noted that the incident waves approach from the Atlantic Ocean. However, wind waves generated in the flooded Indian River Bay may not be negligible during hurricanes with circular wind fields. As for scour at vertical piles, the rule of thumb suggests that the maximum depth of scour at a vertical pile is of the order of twice the pile diameter. The CEM states that this rule of thumb appears to be valid for most cases of combined waves and currents. However, Bijker and de Bruyn (1988) reported that the maximum scour depth around the piles of jack-up platforms increased to 3 times the pile diameter probably due to breaking waves.

For the bridge abutment, the scour depth of the order of H in Table 3 is the lower limit of the scour depth because currents will increase the scour depth by transporting sand particles suspended by breaking waves further downstream. For the arch base, the scour depth of twice the pile diameter is definitely the upper limit of the scour depth around the rectangular lower base of roughly 100-ft length. Figg Bridge Engineers, Inc. (2003) used the dimension of the oval upper base but it is more appropriate to consider the scour around the rectangular base which may be located only 2 – 3 ft below the land surface. This rule of thumb is not applicable to the structure whose horizontal length is much larger than the water depth and as large as the incident wavelength (approximately 130 ft). The large structure diffracts incident waves and modifies the wave pattern around the structure considerable. Kobayashi et al. (1981) computed the vertical erosion rate of about 1 ft/hr around a conical structure with a 350-ft base diameter due to the combined action of nonbreaking waves and currents.

The maximum scour depth will depend on the duration of a storm. The northeaster of March, 1962 lasted over five consecutive high tides but produced the maximum storm tide of approximately 8.26 ft above NGVD 1929, corresponding approximately to h = 3 ft in Tables 3 and 4. On the other hand, extremely severe hurricanes may cause the storm tide which corresponds to h = 5 and 7 ft in Tables 3 and 4 but may not last more than several hours.

Considering all the factors discussed in this section, the maximum scour depth at the arch base and bridge abutment is estimated to be of the order of 10 ft with the uncertainty of a factor 5. In other words, the scour depth may vary in the range of 2 – 50 ft. The uncertainty of a factor 5 may be reduced to a factor 2 if most advanced numerical models are used for the scour prediction. Such a study may cost more than $100,000 and may still recommend scour protection measures because the consequences of the scour around the arch base and bridge abutment are uncertain. For example, the combined wave and current force acting on the exposed rectangular lower base can be very large. This force will then need to be accounted for in the design of drilled shafts supporting the base. The deep scour around the arch base may eventually extend to the riprap revetment of the inlet. The steel sheet piles behind the riprap may have been installed more than 60 years ago and may collapse under the combined action of waves and currents. The lateral erosion of the unprotected bank can occur rapidly and may reach the arch base during an extremely severe storm. In short, it is prudent to mitigate the scour problem instead of dealing with the consequences of the deep scour if the mitigation measure is cheap in comparison to the cost of the entire bridge.

5. Scour Protection

A riprap blanket placed on a bedding layer or geotextile is commonly used to protect areas susceptible to erosion. The stone size needs to be selected such that the shear stress required to dislodge the stone is larger than the fluid shear stress acting on the bottom as described in the Coastal Engineering Manual (2003), which does not consider the case of combined waves and currents around a structure. The maximum bottom shear stress for the design current velocity U and wave height H listed in Table 3 is estimated here using the combined wave and current model for sediment transport discussed in Section 4.

The values of U and H in Table 3 doe not account for the modifications of the current and wave fields by the arch base and bridge abutment. For example, the current velocity becomes approximately twice the approach velocity U in Table 3 on the side of a vertical circular pile at the location of a right angle to the approach flow (e.g., Raudkivi 1990). The diffracted wave around a structure causes the spatial variation of the wave height and the local wave height becomes twice the incident wave height H in Table 3 if a large structure completely reflects the incident waves and no wave breaking occurs. For the present problem, the use of H in Table 3 is reasonable because the shallow water depth h limits the wave height. On the other hand, the use of U in Table 3 underpredicts the local current velocity on the side of a structure. However, it will be shown that the maximum bottom shear stress is determined mostly by the wave.

The maximum bottom shear stress $\tau_{\max}$ under the combined wave and current is the largest when the wave and current are in the same direction. For this case, $\tau_{\max}$ may be assumed to be the sum of the wave bottom shear stress τ_m and the current bottom shear stress (CEM, 2003).

$$\tau_{\max} = \tau_m + \tau_c \tag{8}$$

where τ_m and τ_c are given by Eqs. (5) and (6), respectively, except that H_{rms} and C_d should be replaced by the larger significant wave height H and the wave friction factor for stone armor of the order of 0.1 (Kobayashi et al. 1987). The roughness coefficient n for the current is not modified because the current does not change abruptly over the narrow stone blanket. The critical Shields parameter for the initiation of stone movement may be taken as 0.04 (CEM 2003). This criterion and Eq. (7) yields

$$0.04(S_a - 1)D = 0.015\,h + \frac{n^2 U^2}{1.49^2 h^{1/3}} \tag{9}$$

where S_a = specific gravity of the stone (S_a □ 2.6); and D = stone diameter which may be regarded as the median diameter D_{50}. The first and second terms on the right hand side of Eq. (9) express the wave and current contributions, respectively.

Table 5. Required Stone Diameter *D* for Scour Protection

Water depth *h* (ft)	3	5	7
D (ft) without current	0.70	1.17	1.64
D (ft) with current	0.75	1.24	1.73
Stability number N_s	1.91	1.97	1.99

Table 5 lists the computed values of D without and with the current for the design conditions in Table 3 where the current effect is secondary partly because $C_d = 0.1$ is assumed for the armor stone in Eq. (5) where $C_d = 0.1$ was calibrated for sloping riprap revetments by Kobayashi et al. 1987. Table 5 also lists the value of the stability number $N_s = H/[(S_a-1)D]$ with the value of D with the current. The stability number is used for the design of riprap revetments against wind waves where the stone stability requires N_s less than approximately 2 (Shore Protection Manual 1984). The stone stability on the steep slope due to waves only is not the same as the stone stability on the horizontal bottom due to waves and currents but the coincidence of N_s □ 2 is reassuring for this problem with no design guidance.

Since stone of a 1-ft diameter will not move much even if it is dislodged on a horizontal bottom, it is sufficient to use D = 1.24 ft corresponding to h = 5 ft. Assuming approximately spherical stone (CEM 2003), the required stone weight W is 166 pounds (ℓb). The stone weight distribution should conform to available guidelines (e.g., Shore Protection Manual 1984). For the median weight W_{50} = 166 ℓb, the range of the individual stone weight may be 103 – 268 ℓb for the narrow gradation and 29 – 946 ℓb for the wide gradation (Ahrens and Cox 1990). The thickness of the stone is normally taken as the two layer thickness, $2D \approx$ 2.5 ft, for the riprap revetment. The two-layer thick stone blanket must be placed such that its surface is located at the same elevation as the top of the rectangular footing of the bridge arch. The width of the stone blanket around the arch base and abutment may be taken to be at least 20 ft because the 10-ft scour at the edge of the blanket might result in a slope of 1/2 that is a typical revetment slope.

6. Conclusions and Recommendations

The scour problem associated with the arch base and abutment of the proposed bridge above the Indian River Inlet is investigated using the Coastal Engineering Manual (2003). First, available information on the storm surge and tide at the inlet is reviewed to establish the design water depth at the arch base and abutment and the free surface elevation different between the ocean and bay which drives the current through the inlet. Second, the design current velocity is estimated using a hydraulic formula for tidal inlets. The design wave height is the standard depth-limited wave height.

The sediment transport rates under the combined wave and current action are predicted to show that the design wave and current are capable of transporting the fine sand scoured from the vicinity of the arch base and abutment farther downstream. The maximum scour depth is guessed to be about 10 ft with large uncertainties. Stone blankets surrounding the rectangular footing of the arch and the abutment are recommended to mitigate the scour problem. The cost of the stone blankets is expected to be a very small fraction of the entire cost of the new bridge. The median stone weight is approximately 160 to 170 ℓb. The thickness of the blanket is approximately 2.5 ft. The top of the blanket must be at the same elevation as the top of the rectangular footing. The width of the blanket is at least 20 ft to cope with the scour at the edge of the blanket.

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