PDHonline Course C420 (4 PDH)

Coastal Highways – Planning & Design Issues

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Chapter 6 - Coastal Revetments for Wave Attack

This section addresses the design of revetments on embankments for protection from wave attack. The design of an earthen highway embankment is primarily a geotechnical engineering problem with rock or rip-rap revetments sometimes employed as slope protection. Revetments can be used for protection from four different types of hydraulic situations: direct rainfall impacts, overland flow, stream or river currents, and waves. This section addresses only wave attack.

HEC-11 (Brown and Clyde 1989) provides procedures for the design of riprap revetments for channel bank protection on larger streams and rivers where the active force of the flowing water exceeds the bank material's ability to resist movement. Flow in a stream or river is unidirectional and typically aligned parallel to the banks. Waves produce oscillatory velocities and accelerations that can be in almost any direction on a revetment. HEC-11 recommends Hudson's equation to estimate stone size for revetments subject to wave action.

This section recommends an approach based on determining a design wave and using Hudson's equation to size the stones in the outer layer of a rock revetment. This approach can lead to designs with larger stones and narrower stone gradations than designs for non-wave situations. The difference is due to the higher forces caused by waves. Situations where riverine and wave flows are significant, the design engineer should consider both design approaches and develop a conservative design.

6.1 Types of Revetments and Seawalls

Figure 6.1 shows a revetment along a bay shoreline designed to protect a local road from erosion by waves during storms. This design has a stone revetment extending from below the water surface up to a sheet pile wall and pile cap near the roadway shoulder. Storm surges can exceed the pavement elevation here.

The distinction between revetments, seawalls, and bulkheads is one of functional purpose (USACE 1984). Revetments are layers of protection on the top of a sloped surface to protect the underlying soil. Seawalls are walls designed to protect against large wave forces. Bulkheads are designed primarily to retain the soil behind a vertical wall in locations with less wave action. Design issues such as tie-backs, depth of sheets are primarily controlled by geotechnical issues. Given the relationship between wave height and fetch (distance across the water body) Figure 6.2 provides a conceptual distinction between the three types of coastal protection. Bulkheads are most common where fetches and wave heights are very small. Seawalls are most common where fetches and wave heights are very large. Revetments are often common in intermediate situations such as on bay or lake shorelines.

Seawalls can be rigid structures or rubble-mound structures specifically designed to withstand large waves. Two very large, rigid, concrete seawalls with recurved tops to minimize overtopping are the Galveston Seawall (Figure 6.3) and San Francisco's Great Highway Seawall (Figure 6.4). Such massive structures are not commonly constructed in the US. Vertical sheet pile seawalls with concrete caps are common but require extensive marine structural design. A more common seawall design type in the United States is a rubble-mound that looks very much like a revetment with larger stones to withstand the design wave height. Thus, the two terms, seawalls and revetments, can be used interchangeably with the former typically used for the larger wave environments. Figure 6.5, Figure 6.6, Figure 6.7, and Figure 6.8 are examples of rubble-mound seawalls protecting coastal roads exposed to open-coast storm waves.

Figure 6.1. A revetment protecting a coastal highway. Bayfront Road, Mobile, Alabama (2001)

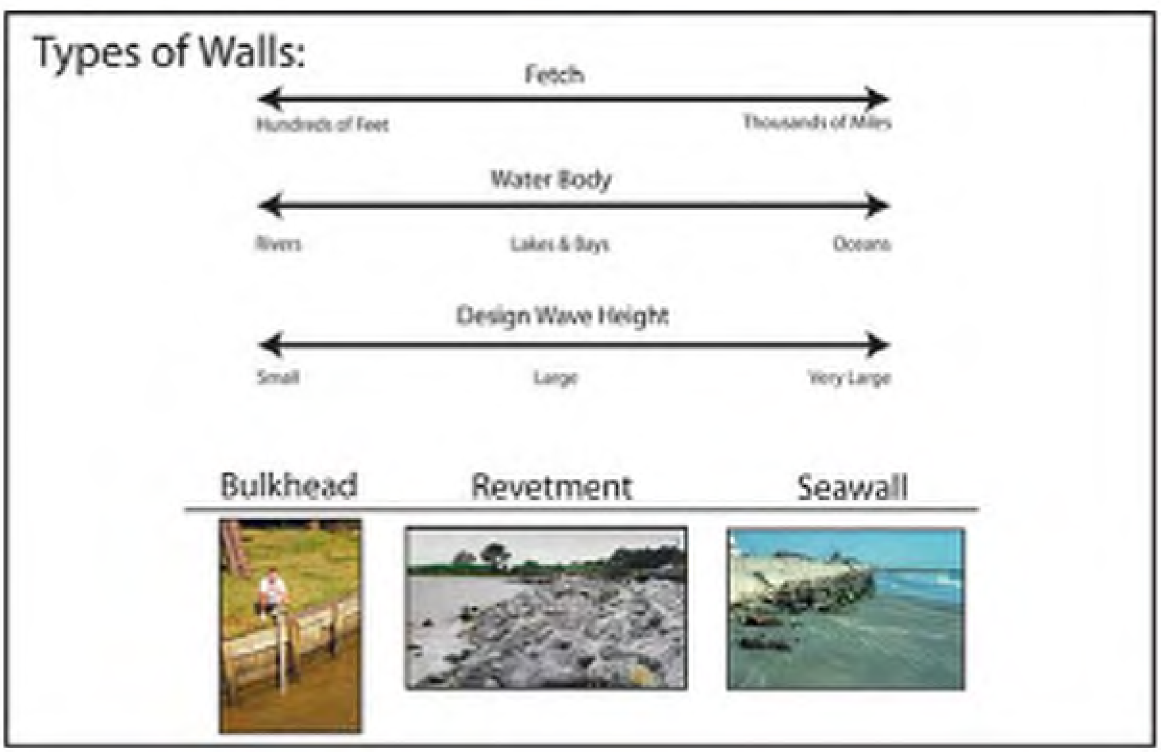

Figure 6.2. Types of shore protection walls.

Figure 6.3. Galveston Seawall. Seawall Boulevard (1983)

Figure 6.4. San Francisco's Great Highway Seawall. California Highway 35 (1991)

Figure 6.5. Seawall protecting a coastal highway. Venice, Florida (2001)

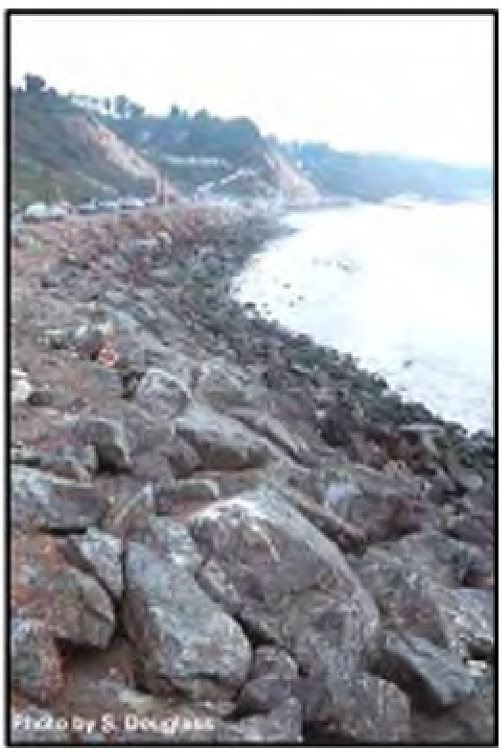

Figure 6.6. Seawall protecting a coastal highway. Pacific Coast Highway, Pacific Palisades, California (2003)

Figure 6.7. Seawall protecting a coastal highway. Florida Highway A1A, Flagler Beach, FL

Figure 6.8. Seawall protecting a coastal highway. US 101, Curry County, Oregon (2001)

Figure 6.9 and Figure 6.10 (as well as Figure 6.1) are examples of rubble-mound revetments protecting highways along coastal bays. Revetments are common on bay or lake shorelines where design waves are short-period, fetch-limited, locally-generated storm waves.

Figure 6.9. Revetment protecting a highway along a bay shoreline. Florida Highway 60, Tampa Bay, Florida (2003)

Figure 6.10. Revetment protecting a highway along a bay shoreline. Washington State Route 105, Willapa Bay, Washington (2003)

Revetments have been criticized for a variety of reasons, including their aesthetics. Figure 6.11 and Figure 6.12 show two different types of protection designed for local roads that were threatened by bluff erosion. Figure 6.11 shows a rock revetment and Figure 6.12 shows a concrete wall that has been designed to look much like the natural bluff. The engineered seawall is in the middle of the Figure 6.12 image. The more aesthetically pleasing seawall (Figure 6.12) was designed more recently than the rock revetment. This is an example of the evolving nature of seawall design in the United States.

Figure 6.11. Seawall protecting a local road. West Cliff Drive, Santa Cruz, California

Figure 6.12. Concrete seawall designed to look like the natural rock formation built on an eroding sea cliff to protect a local road. East Cliff Drive, Santa Cruz, California

6.2 Hudson's Equation for Armor Stone Size

A well-designed and constructed rubble-mound revetment can protect embankments from waves. The underlying philosophy of the rubble-mound is that a pile of stones is efficient at absorbing wave energy and robust in design in that damage is often not catastrophic. It also can be relatively inexpensive. Some of the oldest coastal structures in the world are rubble-mounds. They have the inherent ability to survive storms in excess of their design storm. In the words of an old advertisement for a brand of watches, rubble-mound revetments "can take a licking and keep on ticking." This ability to continue to provide some function even after experiencing storms that are more severe than their design storm is valuable in a coastal environment where costs often preclude selection of extremely rare design storms.

Hudson's equation (USACE 1984) provides a basis for estimating the required stone size in a sloped revetment. The required median weight for the outer, or armor layer, stones is:

$$W_{50} = \frac{w_r H^3}{K_D (S_r - 1)^3 \cot\theta} \tag{6.1}$$

where:

W_{50}	=	median weight of armor stone
w_r	=	unit weight of stone (~165 lb/ft^3)
H	=	design wave height
K_D	=	empirical coefficient (=2.2 for rip-rap gradations)
S_r	=	specific gravity of stone (~2.65)
θ	=	slope

Hudson's equation accounts for the most important variables including design wave height, different structure slopes, different stone densities and angularities. Steeper slopes require larger stones. However, the range of recommended slopes here is up to 2:1 (horizontal:vertical). Note that, by definition, the $\cot\theta=2$ for a 2:1 slope and $\cot\theta=3$ for a 3:1 slope, etc. Revetment structure slopes greater than 1½:1 (horizontal:vertical) are not recommended (USACE 1984).

The empirical coefficient in Hudson's Equation, K_D, is based on laboratory tests and varies to include the effect of stone angularity/roundness, number of layers of armor stone, distribution of individual stone sizes about the median size, and interlocking characteristics. The value suggested here, $K_D = 2.2$, is for a layer of rough-angular quarrystone at least two stones thick. The stones have a gradation of weights that varies between $0.125\ W_{50} < W < 4W_{50}$. Other values of K_D for other situations, including artificial concrete armor units, are discussed in USACE (1984) and USACE (2002).

For typical conditions of specific gravity of stone (S_r=2.65 for granite) and unit weight of stone (w_r=165 lb/ft^3), with the empirical coefficient set to K_D=2.2, Equation 6.1 can be written as:

$$W_{50} = \frac{16.7 H^3}{\cot\theta} \tag{6.2}$$

where:

W_{50}	=	median weight of armor stone (lbs)
H	=	design wave height (feet)
θ	=	slope

Figure 6.13 shows a typical revetment design cross-section. The armor layer stones have a median weight given by Hudson's equation. One component of the design is a filter cloth geotextile or composite geotextile/geogrid between the rocks and the underlying soil. A geotextile that provides rapid transfer of water through the material while holding soil particles and is strong enough to survive the construction process without puncturing by the overlying rocks is recommended. The modern use of a plastic grid integrally welded to the geotextile can provide some additional strength to bridge soft underlying soils. The geotextile should be designed to not allow the rocks to slide down the surface. The use of an underlayer of stones between the armor layer and the geotextile/grid is common except when the stone size is less than 200 lb. The underlayer should have a median weight no smaller than one-tenth that of the armor layer stones (USACE 1984). Smaller underlayer stones can be pulled out between the gaps of the armor stones.

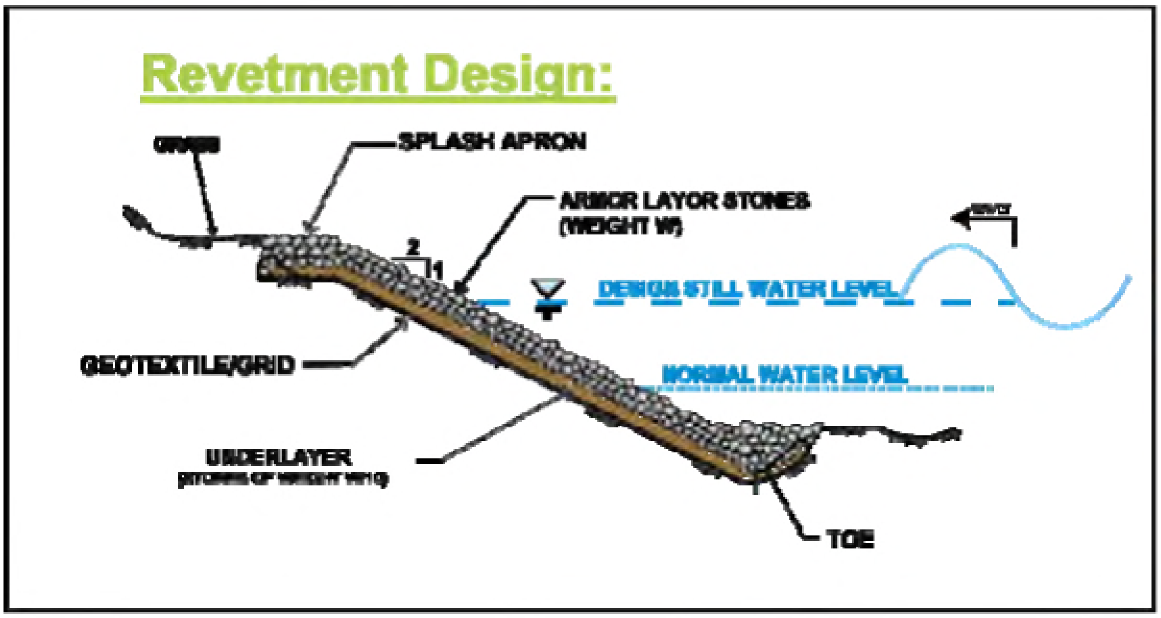

Figure 6.13. Typical coastal revetment design cross-section

6.3 Design Wave Heights for Revetment Design

The estimate of the required armor stone size from Hudson's equation is sensitive to wave height. The proper wave height for Hudson's equation above for coastal revetment design is either the depth-limited maximum wave height or the average of the highest 10% of all wave heights in the design sea-state ($\overline{H}_{10}$) whichever is lesser (USACE 1984).

This recommendation is based on interpretation considering the origin of the equation. Hudson's equation was originally derived based on monochromatic laboratory tests. Thus, the proper selection of a corresponding wave height statistic from an irregular sea-state is not obvious. Experience has found that the use of the significant wave height, H_s, in Hudson's equation is not conservative and can lead to undesired levels of damage to the revetment.

Some researchers have suggested that the proper irregular wave height statistic for use in Hudson's equation is $\overline{H}_{10}$. To be conservative, some engineers use the average of the highest

5% of all wave heights in the design sea-state ($\overline{H_5}$). The relationships (see Table 4.1) between significant wave height and these other statistics are $\overline{H_{10}}$ = 1.27 H_s and $\overline{H_5}$ = 1.38 H_s.

Coastal revetments are often located where the design sea-state is depth-limited, i.e. the depths are so shallow immediately offshore of the location of the revetment that the storm waves have broken and the largest waves are on flat offshore slopes,

$$H_b = 0.8\,d_s \tag{6.3}$$

where:

H_b	=	maximum breaking wave height
d_s	=	design depth at the toe of the structure

To account for the distance over which waves travel as they break, a depth some distance offshore of the toe (say one wavelength) sometimes is used in Equation 6.3. For non flat slopes see USACE (1984) and USACE (2002).

A depth-limited design wave height used in Hudson's equation should account for any long-term erosion that may change the depths immediately offshore. The construction of a revetment, while it protects the upland, does not address the underlying cause of erosion. The depths at the toe of the revetment will likely increase if the erosion process continues. The presence of a revetment or seawall can increase the vertical erosion at its base. The revetment or seawall does not allow the material in the bluff to naturally nourish the beach.

Hudson's equation has no factor-of-safety. Hudson established the K_D values such that there was some small level of damage to the structure. The damage level was defined as the level where 5% of the rocks on the revetment structure armor layer face moved. Thus, it is entirely appropriate for some conservatism or factor of safety to be added to the design process based on engineering judgment. The factor of safety could be included through the selection of a conservative design wave height used (such as $\overline{H_5}$) in Hudson's equation or it could be through an increase in the specified design median rock weight.

Applications of Hudson's equation in situations with a design significant wave height of H = 5 feet or less have performed well. This range of design wave heights encompasses many coastal revetments along highway embankments. When design wave heights get very large and the design water depths get very large, problems with the performance of rubble-mound structures can occur. These problems relate in part to wave groupiness (back to back large waves) , design sea-state specification, constructability and other issues. Seawalls with design wave heights much greater than H=5 feet require more judgment and more experience and input from a trained, experienced coastal engineer. Other details about the design of rubble-mound revetments are discussed in the Coastal Engineering Manual (USACE 2002).

One alternative to the two-layer design of Figure 6.13, is a "dynamic revetment" (or "berm revetment") which contains a significantly larger volume of smaller stones with a wider gradation. A dynamic revetment allows the stones to move in response to storm waves into an equilibrium shape much like a cobble or sand beach.

An alternative to the use of extremely large stones in the armor layer is to use concrete armor units. These typically are lighter since they interlock better than quarrystone and thus have higher K_D values. They can be cast on site. There are a number of shapes of artificial concrete armor units including several patented shapes requiring the payment of license fees.

6.4 Practical Issues for Coastal Revetment Design

The stone gradations recommended above for coastal revetments are much narrower than those typically used for highways. For example, FHWA's Standard Specifications for Class 5 riprap call for a median weight of W_{50} = 770 lbs, with 10% of the stones weighing 0 to 55 lbs., 40% weighing 55 to 770 lbs, 30% weighing 770 to 1540 lbs, and 20% weighing 1540 to 2200 lbs (USDOT 2003).

A footnote to the FHWA specification table says "furnish spalls and rock fragments graded to provide a stable dense mass." However, the gradation recommended above for Hudson's equation for coastal revetment for the same median weight of W_{50} = 770 lbs, calls for all stones to weigh between 100 and 3000 lbs. Thus, the recommended coastal revetment gradation precludes the smaller stones and allows for some larger stones. These smaller stones are typically not included in coastal revetments because of their tendency to move in response to wave action. If there is a potential for the smaller stones that are removed from the revetment during storms causing other damage as projectiles, then the narrower gradation, without the smaller rocks, should be required. This typically results in higher unit costs for the stone.

There are five typical failure mechanisms for coastal revetments:

1. inadequate armor layer design for wave action,
2. inadequate under layer,
3. flanking,
4. toe scour, and
5. overtopping splash.

A revetment's strength depends on the underlying soil. If wave action can remove that soil via any mechanism, the revetment will collapse. Each of the four typical failure mechanisms involves failure to protect that underlying soil. Each can be prevented by careful design by an experienced engineer.

Figure 6.14 shows a failed attempt to protect an embankment. The slope protection used concrete slab panels. The concrete panels were available from some other project and were set on the surface of the eroding bluff. Although the panels were heavy enough to withstand the wave action itself, wave action during storms likely pulled, or pumped, the underlying soil out from between the gaps in the slabs. Consequently, the panels collapsed. The second photograph shows the panels after collapse. A rock revetment was subsequently placed farther back on the bluff. The original panel design did not adequately protect the underlying soil and did not have the flexibility of a rubble-mound revetment.

Figure 6.14. Example of a failed attempt at embankment protection (USACE archives photo)

Hudson's equation can usually be used to select the stone size in the outer layer of a revetment subjected to wave attack and it was specifically developed for that situation. However, careful engineering judgment based on experience should be used when the design cross-section varies from that in Figure 6.13. Figure 6.15 shows a revetment protecting a highway that has a small, vertical bulkhead with stones on the seaward side and an almost flat stone section landward. This cross-section design essentially "trips" breaking waves when storm surge raises the water level and begins to inundate the highway. Thus, breaking waves can plunge directly on the stones and move them onto and across the roadway during major storms. For very mild slopes, Hudson's equation estimates very small armor stone and adjustments may be needed. A larger stone weight would prevent this type of failure.

Figure 6.15. A revetment with rocks too small to withstand wave attack

Flanking occurs when adjacent, unprotected shorelines continue to recede. Erosion at the end of the wall allows wave action to remove the soil from behind the wall starting at the ends, then progressing along the walls it fails. Flanking can be avoided by extending the revetment or wall to meet an existing revetment or a wall or natural rock outcropping, or by using a return wall. A return wall is aligned perpendicular to the shoreline. The length of the return wall should exceed the expected long-term and storm-induced recession of the adjacent shorelines.

Vertical scour at the toe of a revetment or seawall can cause the underlying soil to be exposed to waves. One solution to toe scour problems is shown in the recommended revetment cross-section in Figure 6.13. A significant volume of stones is placed at the toe. This toe is designed to collapse into any toe scour hole that develops without loss of the stones on the slope. For very erosive areas, more stones can be used in the toe.

Overtopping splash at the top of a revetment or seawall can also lead to failure by exposing the underlying soil to waves. If the wall does not extend to a high enough elevation, waves will overtop the wall. Figure 6.16 shows indications of overtopping splash damage at the top of rock seawall.

Figure 6.16. An example of splash damage behind seawall

A solution to overtopping splash problems is to provide a splash apron as is shown in the revetment cross-section in Figure 6.13. The rocks extend some distance back from the break in slope. The width of the splash apron varies depending on the severity of the expected overtopping. A minimal splash apron width is 5 to 10 feet.

The elevation of the top of the revetment in Figure 6.13 was based on the elevation of the top of an existing embankment. It was assumed that wave runup would allow some limited overtopping at the design conditions. The splash apron was thus included. For situations where the embankment elevation is much higher than the expected level of wave runup during design conditions, a decision regarding the height of the revetment is required. The height of wave runup (R_u) is shown in Figure 6.17. It can be estimated using:

$$\frac{R_{u,2\%}}{H_s} = 1.6\ r\,\xi_{op} \qquad \text{with a maximum of } 3.2\ r \qquad (6.4)$$

where:

$R_{u,2\%}$	=	runup level exceeded by 2% of the runups in an irregular sea
H_s	=	significant wave height near the toe of slope
r	=	a roughness coefficient (r = 0.55 for the stone revetments)
ξ_{op}	=	the surf similarity parameter as defined below

$$\xi_{op} = \frac{\tan\theta}{\sqrt{\frac{2\,\pi\,H_s}{g\,T_p^2}}} \tag{6.5}$$

where:

θ	=	angle of slope of structure (see Figure 6.17)
H_s	=	significant wave height
T_p	=	wave period, peak period
g	=	acceleration of gravity

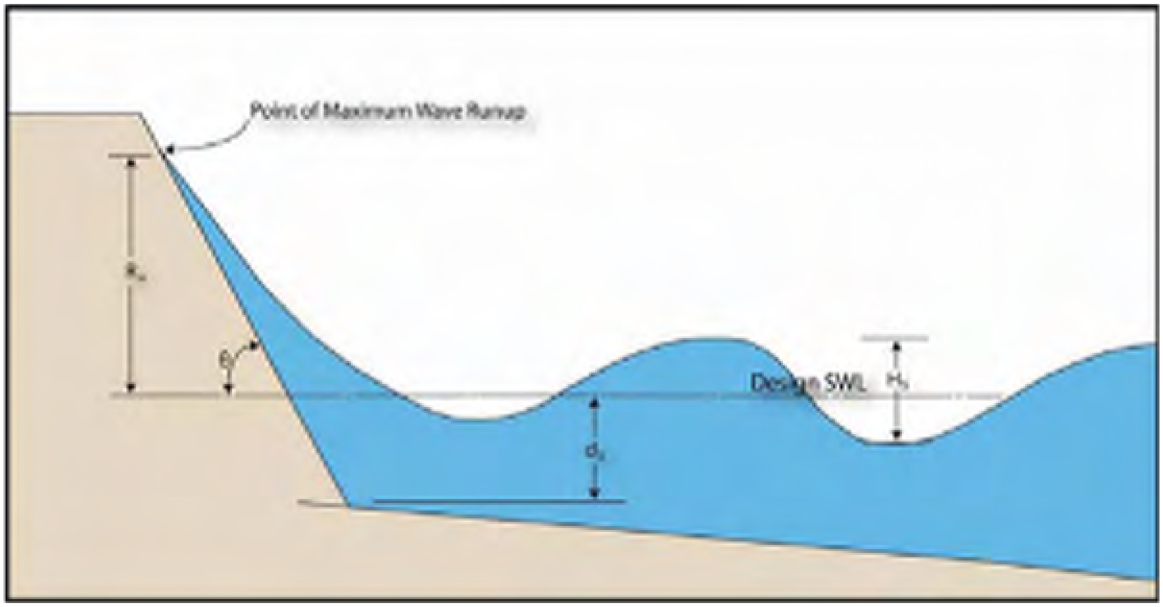

Figure 6.17. Wave runup definition sketch

The level given by Equation 6.4 is for the 2% runup level. This runup level is defined as the runup level exceeded by 2% of the incoming waves. Thus, 2% of the waves will run up higher than this level. The roughness coefficient (r) accounts for the roughness of the surface of the revetment with r = 1 for smooth slopes. For rock revetments such as shown in Figure 6.11, the recommended value for r is 0.55. For r=0.55, Equation 6.4 has an upper limit of 3.2r = 1.76. Thus, the 2% level of runup is $\leq$ 1.76H_s. Equation 6.4 is adapted from a methodology developed by Van der Meer and summarized by Pilarczyk (1999). More detail including other structure geometries can be found in that reference.

Wave overtopping of revetments and seawalls occurs when runup exceeds the top or crest of the structure. Building seawalls high enough to completely prevent overtopping is often unacceptable because of aesthetics and costs. Wave overtopping onto coastal roads is fairly common in some parts of the country. Two aspects of overtopping of interest to the design engineer are the time-averaged volumetric rate of overtopping and the intensity or force of a single wave overtopping event. Accurately estimating volumetric overtopping rates can be vital to design of seawall crest elevations if inland flooding is caused. Unfortunately, accurately estimating overtopping rates can be very difficult for many situations and input to the design team from a trained coastal engineer is likely appropriate. Guidance on estimating overtopping can be found in Goda (1985) and USACE (2002).

A commonly proposed alternative to rubble mound revetments is a concrete block revetment. Some of these have some physical interlocking between individual blocks and others do not. The performance of interlocking blocks in severe coastal environments has not been good. One problem is that minor damage can lead to failure of a large portion of the revetment. Two examples are shown in Figure 6.18 and Figure 6.19. The failed revetment in Figure 6.18 has been covered by a sand beach through beach nourishment (see Figure 7.16). The failed revetment in Figure 6.19 has been replaced by a sand beach through beach nourishment and stabilized by offshore segmented breakwaters (see Figure 7.18). Problems with concrete block revetments in coastal situations often develop at the ends of the revetment where the blocks abut a more rigid structure. Even a small amount of settlement can affect the aesthetics of block revetments.

Figure 6.18. Example of rigid concrete-block revetment failure (Florida Highway A1A, Delray Beach, circa 1972; University of Florida and USACE archive photos)

Figure 6.19. Example of failed block revetment (Louisiana Highway 87, circa 1980, USACE archives photos)

Another commonly proposed alternative to rubble mound revetments are rigid concrete panel designs. Performance of rigid concrete panels in severe coastal environments also has not been good. A concrete panel revetment on a bridge approach that suffered damage in a hurricane is shown in Figure 6.20. The underlying soil was not adequately protected from wave attack. Neither interlocking blocks nor concrete panels match the performance and flexibility of stone revetments. The Florida DOT does not allow rigid revetments in wave situations.

Figure 6.20. Example of rigid revetment failure on a coastal highway bridge approach

Other revetment systems include articulated concrete mats, flexible rock-filled marine mattresses, gabions, and sand-filled geotextile tubes or bags. Articulated concrete mats have concrete blocks interconnected by strong cables. The size and weight of the blocks are a function of the wave height, slope, currents, etc. Proper installation requires adequate filtration material and secure anchoring at the top of the slope. The toe is sometimes unsecured to allow it to settle (scour). Flexible rock-filled marine mattresses are used as foundations and for scour control underneath marine structures; but they are not generally recommended for slope protection by themselves. Gabions are rock-filled "baskets" composed of steel wire or polypropylene grid which are stacked for embankment protection. Their use in energetic coastal environments, where wave heights may routinely exceed 1 to 3 feet, is not generally recommended. Sand-filled geotextile containers (tubes or bags) are typically only used for temporary, interim embankment protection in the coastal zone. Where used, they are best buried within the existing grade and become exposed only during storm erosion (an example is illustrated in Figure 7.3). The structures are prone to damage or failure by vandalism, rolling, and natural deterioration when exposed.

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Chapter 7 - Roads in Areas of Receding Shorelines

Much of the American coastline is experiencing long-term recession. When a highway is near one of these receding shorelines, it can eventually be subjected to wave attack and erosion. This section outlines how long-term shoreline changes can be quantified and used to estimate future shoreline positions, ways to evaluate the vulnerability of coastal highways, the general options available for roadway relocation, and alternative shoreline stabilization techniques available for protecting a highway in place.

7.1 Examples of Issues

Figure 7.1 shows a roadway with a rubble mound revetment seawall protecting it from waves. In the 1970's this road was located over 300 feet landward of the shoreline. The beach here is eroding at a high rate so that the shoreline has been moving toward the road at an average rate of 15 feet per year for the past 35 years. Shoreline recession progressively narrowed the beach until an emergency rock revetment/seawall was constructed. The revetment has not slowed the recession of the adjacent beaches. There are exposed tree stumps in the surf and on the beach face as a result of the recession. Shoreline recession has continued on both sides of the revetment and the road is extending farther out into the sea. The revetment is now protecting the road and functioning like an artificial headland.

Figure 7.1. A road initially built inland of a receding shoreline is now in the sea. Stump Hole area of Cape San Blas, Florida (2005 FDOT photo)

Figure 7.2 through Figure 7.5 show other examples of roads threatened by long-term shoreline recession. The problem occurs in a variety of coastal settings including coastal bluffs and low-lying barrier islands.

Figure 7.2 shows Cape Shoalwater area of Washington State Road 105 built along a rapidly receding shoreline. At the time of this photograph (April, 2003) there was a rock revetment at the base of the bluff and a groin in the background. There had been some limited beach nourishment.

Figure 7.2. A highway initially built inland now threatened by long-term shoreline erosion. Cape Shoalwater area of Washington State Road 105 (2003)

Figure 7.3 shows a road on a narrow, low-lying barrier island in New Jersey. The road parallels the beach on one side and a back-bay wetland on the other. The shoreline here has been receding for decades and the road is threatened. Several shoreline stabilization and roadway protection projects have been attempted. The sand-filled geotextile tube was built and covered with a sand dune to protect the highway is being repaired after a storm in 2003.

Figure 7.3. A local road threatened by long-term shoreline recession. Ocean Drive, Whale Beach area of Cape May County Road 619, Ludlam Island, New Jersey (2003)

Figure 7.4 shows a local road undermined by bluff erosion on Lake Erie. This road used to continue straight ahead until bluff erosion undermined the pavement. The bluff erosion has been exacerbated by sand starvation of the beaches at the base of the bluff by an updrift jetty system.

Figure 7.4. A local road being undermined by bluff erosion and long-term shoreline recession on the Great Lakes. Painesville, Ohio (2001)

Figure 7.5 shows Texas Highway 87 along the east Texas coast of the Gulf of Mexico destroyed by shoreline recession. A twenty-mile stretch of this highway along the coast is now closed. It has been closed since 1989 when a storm caused significant pavement damage. Four-wheel drive access is permitted now but is not feasible when tides are high.

Figure 7.5. Road destroyed by shoreline recession: a) broken pavement on the beach at the old location; b) south end of the closed section; c) location map. Texas Highway 87, Jefferson County (2002)

7.2 Quantifying Shoreline Change Rates

Coastal erosion rates are often given in terms of the change in average annual shoreline position with time, e.g. 2 feet per year. These are actually shoreline change rates rather than erosion rates. The terms "recession" and "accretion" are typically used to describe the direction of shoreline movements. A beach that is widening in response to sand deposition has an accreting shoreline. A beach that is narrowing in response to erosion has a receding shoreline.

Shoreline change rates typically vary with location and time. Shoreline change rates should be looked over as long a time period as possible with as many observations as possible. "Long-term" shoreline change usually refers to multi-decadal time scales. Many observations in a single year can give some estimate of the seasonal variability in shoreline position as sand moves cross-shore on the profile. Typically these data are not useful for developing "long-term" shoreline change trends.

Historical shoreline data are available from a variety of sources including state coastal resource agencies, federal agencies that deal with the coast, and universities.

One example is the USGS results for the Atlantic and Gulf coasts (http://coastal.er.usgs.gov/shoreline-change/ 2006). A state resource agency example is the State of Florida's Department of Beaches and Coastal Systems database and analyses results (http://www.dep.state.fl.us/beaches/ 2006).

There is no accepted national standard for shoreline change analyses. The quantity and quality of shoreline change data vary significantly. Each location has different types of historical data and analyses. The most problematic shoreline recession areas in the United States have likely been studied by a variety of agencies and researchers. Developing a clear understanding of historic shoreline changes for a project can require new analysis of existing data.

Historical shoreline positions can be measured by repetitive surveys or by remote sensing such as air photograph interpretation. Historical and current vertical air photographs can provide the basis for shoreline location data with proper interpretation and positioning analysis. One source for estimates of older historic shoreline locations is NOAA's National Ocean Survey surveys and the surveys of their predecessor organization, the US Coast & Geodetic Survey (USC&GS). One example of the variability of historic shoreline positions from these surveys is shown in Figure 7.6. High-quality estimates of shoreline position can extend as far back as the 1850's. The USC&GS significantly improved the accuracy of coastal surveys at about that time. Pre-1840 estimates of shoreline position done by the USC&GS are typically not as accurate as those done after 1850. USC&GS "t-sheets" and "h-sheets" are the summary plots of specific surveys and correspond with the dates of the actual survey. Navigation charts, however, are updated continuously and the date of the chart does not correspond with the date of all of the information shown on it. Accuracy of these historical shoreline estimates often can be adequate for the purpose of shoreline change analysis (Crowell, et al. 1991).

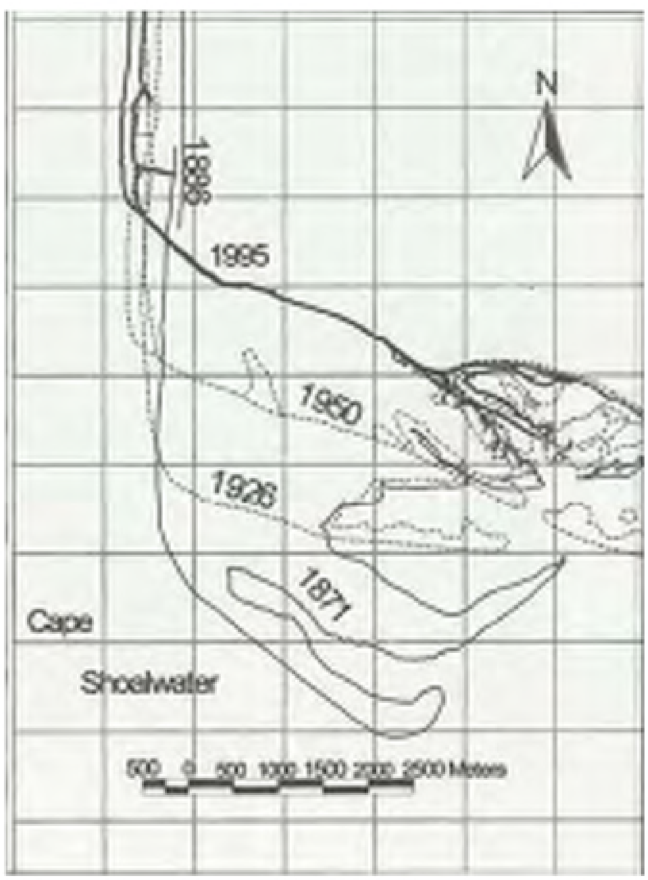

Figure 7.6. An example of historical shorelines based on USC&GS/NOS surveys updated with modern technology. Cape Shoalwater, Washington (Kaminsky, et al. 1999)

An example of a shoreline change analysis is shown in Figure 7.7. The plot is for five locations, spaced 1000 feet apart, centered on the location where the road in Figure 7.1 extends into the sea. The plot shows the measured shoreline locations through time and the lines are splines fit to the data for visual convenience. A recessional (negative) trend is obvious at all five locations and is very consistent at four of the five locations. There is some variability in the overall trend at station R-106 that may be explained by effect of the revetment protecting the road (the nomenclature and designations of the stations are those of the Florida Department of Environmental Protection). Given the natural temporal variability of shoreline location, the strong trends shown in Figure 7.7 are not typical. Similar plots often show much more variability through time and the trend is not always clear. The site analyzed in Figure 7.7 has a very clearly recessional shoreline. Figure 7.7 shows a non-linear trend in shoreline position through time. The recession rate appears to be greatest in the most recent years. A relatively large number of major storms have impacted this coast since 1997.

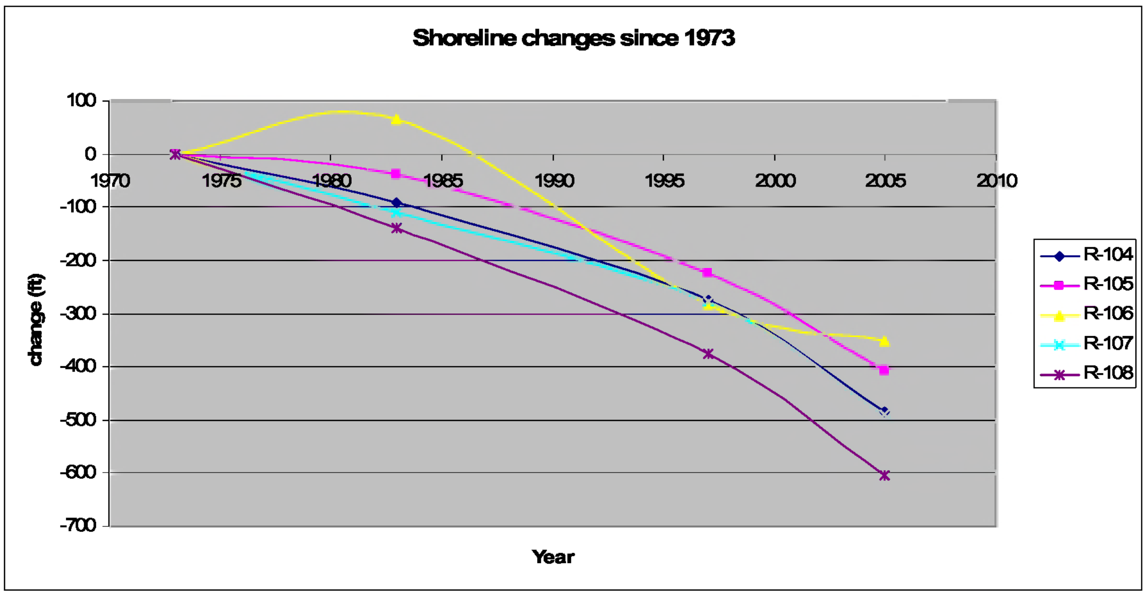

Figure 7.7. An example of shoreline position changes through time. Stump Hole area of St. Joseph's Peninsula, Florida

More results from the same shoreline change analysis are shown in Figure 7.8. The average annual recession rate along 30,000 feet of shoreline on the west-facing shoreline of St. Joseph Peninsula is shown. The average annual rate depends on location and the time period over which the average is taken. Clearly the recession rate is much greater to the south (higher R-monument numbers).

Recession rates shown in Figure 7.8 have been calculated by the "end-point method" which averages the change in shoreline position from the beginning to the end of the time period. An alternative to the end-point method is linear regression (Crowell, et al. 1997). Linear regression is typically preferred to the end-point method because it uses all the available data and is less sensitive to one spurious or aberrant value.

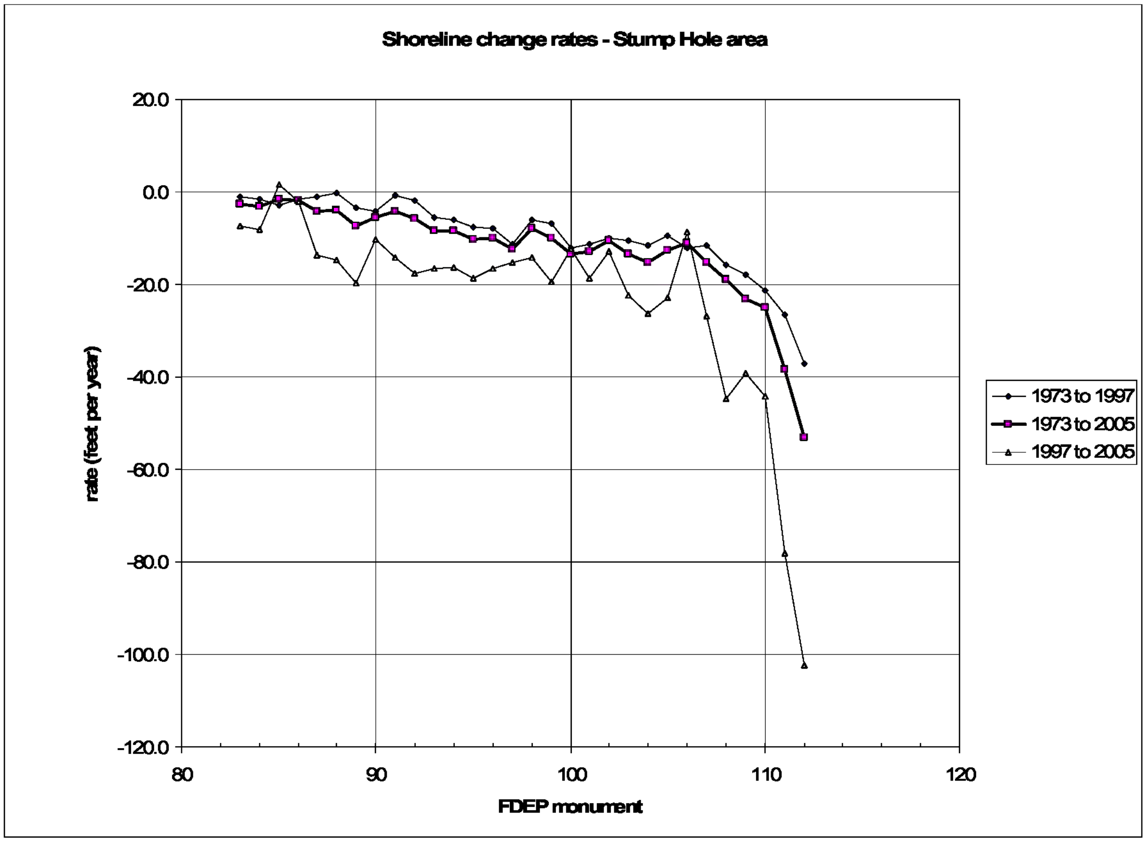

Figure 7.8. An example of shoreline change rates along 30,000 feet of coast showing temporal and spatial variations but a significant recessional trend. Western-facing shoreline of St. Joseph's Peninsula, Florida

7.3 Estimating Future Shoreline Positions

An estimate of future shoreline locations can be valuable in planning highways near areas of receding shorelines. The most common method for estimating future shoreline positions is direct extrapolation of historic shoreline change rates to the present shoreline (Crowell, et al. 1997). Figure 7.9 shows some historic shoreline positions as well as projected future positions at one location. The historic shoreline data was obtained from the FDEP on-line database. Florida originally obtained the older (1868 and 1934) data from the USC&GS, made appropriate datum corrections and added their own data from beach profile surveys. The projected shorelines are extrapolations from the 2005 shoreline location, at 1000 foot intervals along the coast based on the average annual rate of shoreline recession. The average annual rate of shoreline recession was based on the most recent 32 years (1973 to 2005). The result shows that more and more of the highway will be threatened by recession in the coming decades. This information and its graphical presentation, can be valuable in planning alternative responses.

Figure 7.9. Example of projected future shoreline positions at Stump Hole (FDOT figure)

7.3.1 Shortcomings of Shoreline Change Assumptions

There are theoretical and practical shortcomings with the underlying assumptions in using historic shoreline change rates to estimate future shoreline position. They include:

1. Natural shoreline change processes are often not linear in time.
2. Engineering may have influenced historic shoreline changes.
3. Engineering may influence future shoreline changes.

It has long been recognized that shoreline change can be episodic. An individual storm may cause significant erosion or even trigger the beginning of an erosional period. The natural dynamic equilibrium on some beaches involves years of recovery after major storms. Large storms on low-lying barrier islands can cause island rollover and migration. Large storms on some coasts may remove large amounts of sand from the beach, via longshore and cross-shore sand transport and cause bluff erosion. Subsequent times of lesser storm activity can result in the replacement of much of that sand by similar processes.

Shoreline position in many US locations has been influenced either positively or negatively by engineering works. Engineering works can include seawalls, groins, breakwaters, inlet jetties, dams (on the US West Coast), dredging of ship channels, and beach nourishment. For example, a groin that traps sand will often widen an updrift beach while narrowing a downdrift beach. Over 1 billion cubic yards of sand have been trapped or removed from US beaches by the works of man (Douglass, et al. 2003). Beach nourishment projects can widen beaches significantly. Roughly 0.5 billion cubic yards of sand have been placed on 200 areas along the US coast (Campbell and Benedet 2004).

7.3.2 Sediment Budgets

Sediment budgets can be used to estimate future shoreline positions. Sediment budgets are estimates of the rate at which sand is entering, leaving a specific reach along the coast. The difference between the volume entering and the volume leaving an area yields the volume gained or lost by that area. Sediment budgets typically require much more data and analysis than simple shoreline change extrapolation. An example coastal sediment budget for Florida's St. Joseph's Peninsula is shown in Figure 7.10. Sediment budgets are often developed to understand a specific erosion problem and to develop alternative solutions. Input data usually include historic shoreline change rates or beach profile data. The sediment budget shown in Figure 7.10 was based on volumetric changes between 1973 and 1997. The sediment budget shows that the "Stump Hole" area just north of R-110 is losing an average of 185,000 cubic yards of sand per year. This is the cause of the shoreline recession threatening the road in Figure 7.1.

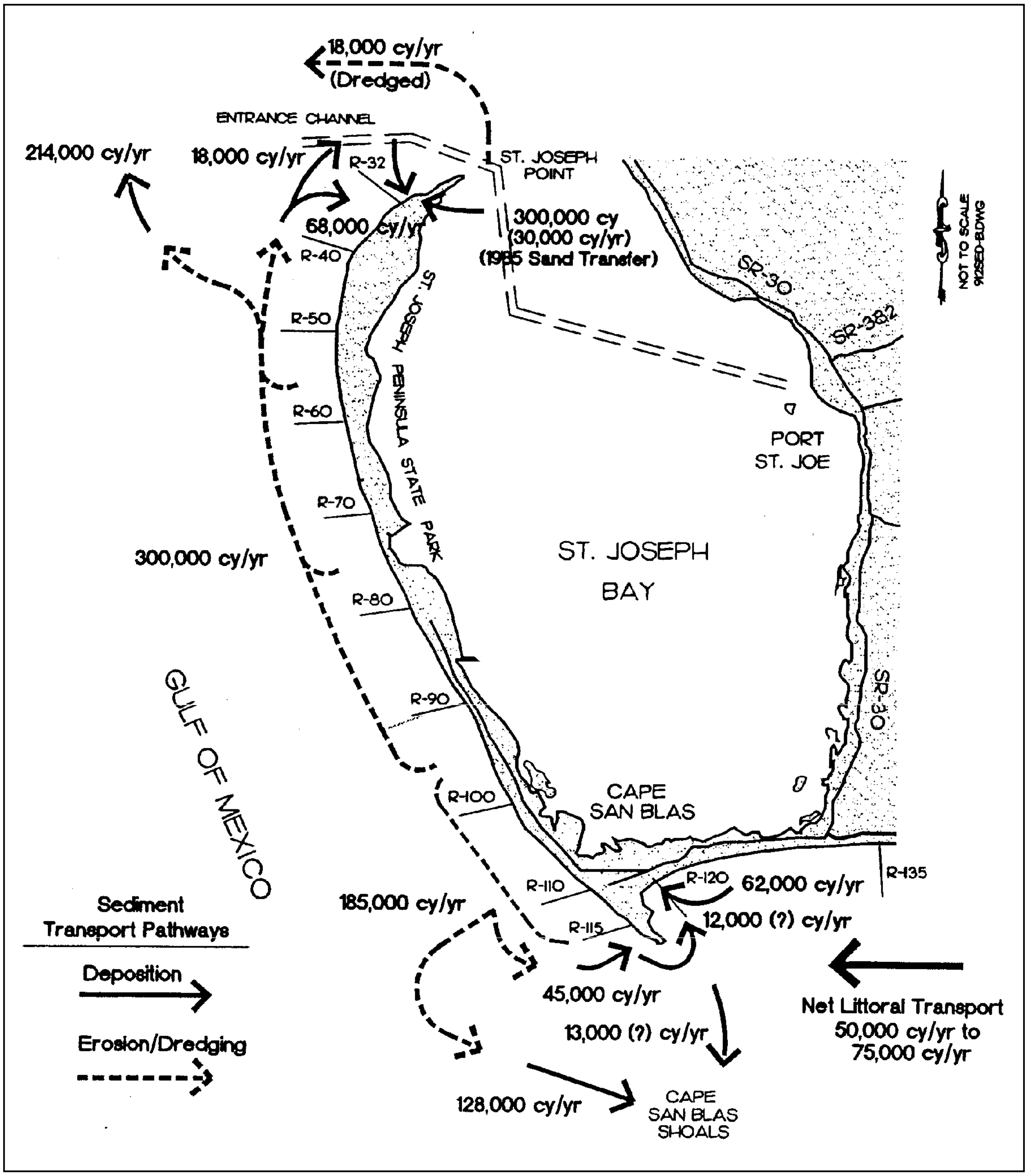

Figure 7.10. Example of a coastal sediment budget (Coastal Tech and Preble-Rish, Inc. 1998)

7.4 Vulnerability Studies for Coastal Roads and Bridges

Some fraction of the over 60,000 highway miles in the United States that are occasionally exposed to coastal waves and surge have already been damaged and will be damaged in the future. "The long-term expectation of continued highway damage requires comprehensive and continuing studies of highway vulnerability" (AASHTO Highway Drainage Guidelines 1999). Clearly, some of these coastal road miles are more vulnerable than others. Planning decisions related to repair, protect, or relocate these highways may be accomplished in a cost-effective manner based on a vulnerability study.

The decision to repair, protect, or relocate coastal highways requires an assessment of many variables including shoreline recession rates, protection afforded by existing and projected beach width, dune size, bluff geology, present and future transportation needs, and costs. A systematic method to anticipate future erosion problems along coastal highways and to evaluate responses for their repair and protection needs to be developed. The following objectives should be addressed by such studies (AASHTO Highway Drainage Guidelines):

- Identify the relative vulnerability of highway actions in the coastal zone to long-term erosion including the effects of storms and hurricanes
- Evaluate feasible engineering solutions for protecting and repairing coastal highways.
- Review and document prior highway damage, causes, remedial actions, costs, and effectiveness of solutions.
- Develop and test a methodology for matching repair and protection strategies to highway sections for different vulnerability scenarios.
- Use the model to estimate the location of all vulnerable sections and identify protection actions and costs for a predefined planning period.

Details of the model depend on the local coastal processes threatening the highway. In areas where dunes protect highways, available dune erosion models can be used to evaluate the level of protection. Vulnerability means that the coastal highway is susceptible to excessive overwash or undermining of the highway base. Transportation officials usually perceive a vulnerability problem when maintenance crews are required to make repairs several times per year (AASHTO Highway Drainage Guidelines).

Coastal highway vulnerability models are built from two databases:

1. A digitized map with elevations and shoreline position
2. An estimate of long-term shoreline recession rates.

This data can be integrated and organized for presentation on base maps and spreadsheets. When completed, this data will identify specific locations of vulnerable highways (AASHTO Highway Drainage Guidelines). For example, Figure 7.11 shows transects evaluated for vulnerability along a portion of North Carolina Highway 12. Each transect was evaluated using a model that incorporated both long-term shoreline change rates and storm-induced dune erosion (Moffat & Nichol 2005).

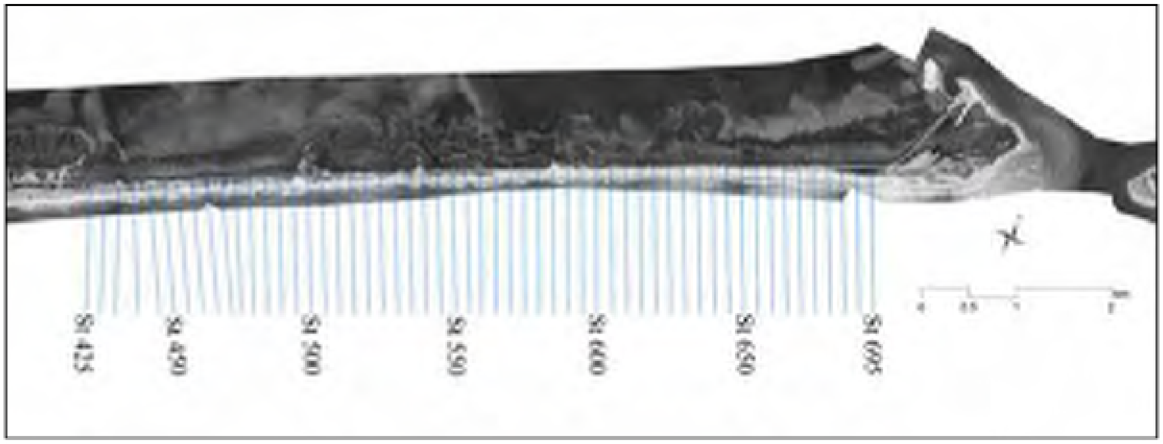

Figure 7.11. Example of coverage for a vulnerability study (North Carolina DOT)

Highway vulnerability studies based on the dominant, local coastal processes have proven to be an effective planning tool. For example, much of the damage to North Carolina Highway 12 caused by Hurricane Isabel in 2003 occurred in areas previously designated as highly vulnerable “hot spots” (Overton and Fisher 2004a). The models used in North Carolina have been developed for that coast using some of the principles and tools (SBEACH, ADCIRC, Kriebel’s dune erosion model, etc.) outlined earlier (Judge, et al. 2003, Krynock, et al. 2005).

Vulnerability study methodology should evolve as part of a permanent highway assessment program. For example, North Carolina’s methodology has been refined through time with the inclusion of modern research results (Stone, et al. 1991, Overton and Fisher 2004b). As another example, Florida DOT began a process of evaluating the vulnerability of all their coastal bridges after the hurricane of 2004 and 2005 proved that some were vulnerable to waves on storm surge.

7.5 Relocation Considerations

One obvious solution to the problem of a roadway threatened by shoreline recession is to relocate the road. Roads have been moved or abandoned at different locations along the US coast for decades. One example is Washington State Road 105 in the Cape Shoalwater area (see Figure 7.1). This road was moved several times since this area has experienced some of the highest, long-term shoreline recession rates in the nation. In 1998, a rock groin and revetment were built to protect the existing highway. Relocation of the road had again been considered but not selected as the preferred alternative.

One example of an abandoned road is Texas Highway 87 between High Island and Sabine (see Figure 7.5). The road was closed indefinitely due to damage by Hurricane Jerry in 1989. Prior to that, the road had been damaged repeatedly by coastal storms. A road in that location had been there for over a century and the local government in Jefferson County is working to re-open the road.

A primary issue when considering road relocation is the new route. The logical location is farther inland from a receding shoreline. However, those areas are often already occupied by private property or wetlands. Developing private property is extremely expensive due to its location near the coast. Wetlands maybe productive coastal wetlands protected for their habitat value.

The stretch of Texas Highway 87 that is closed today is in front of wetlands that are part of the McFadden National Wildlife Refuge. Relocating the road landward would require filling the wetlands. Likewise, relocation of CR 30E in the Stump Hole area of Cape San Blas (see Figure 7.1) would require the filling of wetlands currently managed by the state as an aquatic preserve. Alternative relocation options considered for Washington Highway 105 in the Cape Shoalwater area included private cranberry bog farms.

7.5.1 Shoreline Stabilization Options

An option along a receding shoreline is some form of shore stabilization or protection. Stabilization is essentially holding the line and resisting the recession. The shore protection generally is in one of two forms. One, some form of "hard" structural shoreline protection such as a seawall or groins or breakwaters. Two, some form of "soft" sand shoreline protection such as beach nourishment. There are many combinations of structures with nourishment.

7.6 Coastal Structures

Coastal structures can be categorized in terms of their primary function as follows:

1. seawalls, revetments, bulkheads – shore-parallel structures on the shoreline designed to protect upland property from waves
2. groins – shore perpendicular structures designed to control longshore sand transport
3. breakwaters – shore-parallel structures located seaward of the shoreline to reduce the wave energy in their lee and to control longshore sand transport,
4. hybrid structures – some functional combination of groins and breakwaters including "t-head groins" or "headland breakwaters"

Groins were probably the most common shoreline stabilization technique in the first half of the 20^{th} century. Figure 7.12 and Figure 7.13 shows two groin fields. Groins are typically placed as shown in groups or "fields." They are often called "jetties" but that term is typically reserved by the US coastal engineering community for structures that stabilize inlets. Groins can stabilize a shoreline via two mechanisms if there is adequate sand in the littoral system:

- Groins can locally realign the shoreline (shown in Figure 7.12) to reduce the longshore sand transport rate.
- Groins can shelter the area adjacent to them from the wave energy especially when waves approach the shore at an angle.

Figure 7.12. Groin field in Long Beach, New York (New York Sea Grant photo)

Figure 7.13. Groin field in Long Branch, New Jersey (2006)

Groins can trap sand on one side while causing erosion on the other. The shoreline on the updrift side of a groin accretes while the shoreline on the downdrift side recedes. Thus, groins are often built in groin fields so the one just downdrift stabilizes the next portion of the shore. Shoreline recession downdrift of the last groin at the end of a groin field can be severe (see Figure 7.14). Groins are much less acceptable today as a shoreline stabilization technique than they were prior to the 1960's. New groins are discouraged or prohibited in many states today because of their potential downdrift negative impacts.

Figure 7.14. Severe shoreline recession and beach erosion downdrift of a groin field (West Hampton, New York, circa 1985, New York Sea Grant photo)

7.6.1 Beach Nourishment

Beach nourishment is the placement of large volumes of good quality sand to widen a beach. Sand dunes can be constructed at the back of a nourished beach. "Beach nourishment is a viable engineering alternative for shore protection" (National Research Council 1995). Nourishment also has become the principal technique for beach restoration.

Figure 7.15 shows a beach nourishment project under construction. Sand is being pumped from an offshore dredge (not shown) to the beach and then down the beach to where the sand-water slurry discharges from the pipe. The beach is then shaped by bulldozers. As the new beach extends farther down the beach, the dredge pipe is extended.

Figure 7.15. A beach nourishment project under construction. Gulf Shores, Alabama (2001)

Beach nourishment projects usually need to be maintained through subsequent renourishment as the sand moves out of the project limits. Many of the policy, management, and engineering issues related to beach nourishment projects are qualitatively described in Douglass (2002). Many of the quantitative engineering tools used in beach nourishment planning and design are presented in Dean (2002). The available quantitative tools for beach nourishment engineering for shoreline stabilization include methods for evaluating the performance of potential nourishment sands, estimating the short-term performance and the long-term renourishment intervals, and evaluating the ability of structures (if desired) to extend the renourishment interval. Each of these can be critical aspects of beach nourishment planning and design.

Beach nourishment projects protect a number of roads in the US. Two examples are shown in Figure 7.16 and Figure 7.17. The beach and dune in Figure 7.16 was constructed by the City of Delray Beach on top of the failed seawall shown in Figure 6.18. The sidewalk and parapet wall on the crest of the seawall in Figure 6.18 is the same as the sidewalk and bench shown in Figure 7.16. Since originally constructed in 1973 this beach nourishment project has protected the road while providing a beach. The site has been renourished four times since 1973.

Figure 7.16. Beach nourishment project with constructed dune on top of old, failed revetment protecting road. Florida Highway A1A, Delray Beach (2001)

The nourishment project at Sea Bright, New Jersey, shown in Figure 7.17 is a federal shore protection project funded through the USACE's shore protection authority. The beach was constructed in 1994 directly seaward of the seawall. Nourishment was the preferred alternative to further seawall repairs.

Proponents for beach nourishment projects have typically not been DOTs, even when the project protects a state highway. Rather, local government, a state resource management or economic development agency, the USACE, or a private entity typically sponsors beach nourishment. There have, however, been several beach nourishment projects sponsored or co-sponsored by a SDOT.

The USACE shore protection program has the authority to consider and build either beach nourishment or seawalls to protect upland property. Almost all of the USACE's federally authorized beach nourishment projects require a significant (35% to 50%) matching cost contribution from a non-federal sponsor. The USACE shore protection program typically has an annual budget of around $100 million and the program has not grown significantly during the past several decades.

Beach nourishment should be considered by transportation engineers where a road is threatened by a receding shoreline because of nourishment's effectiveness and its broader societal benefits of aesthetics, recreation and environmental enhancement.

Figure 7.17. Beach nourishment seaward of a seawall protecting a road. New Jersey State Highway 35, Sea Bright, New Jersey (2001)

7.6.2 Combining Beach Nourishment with Structures

Modern coastal engineering shoreline stabilization solutions often combine beach nourishment with coastal structures. The purpose of the structures is to extend the interval between periodic renourishment. Some of these "hybrid" soft-hard solutions attempt to emulate natural geomorphological features such as pocket beaches and tombolos. The names of these "hybrid" solutions are still evolving.

Figure 7.18 shows a nearshore segmented breakwater with beach nourishment protecting a highway. This is the highway once protected by the concrete block revetment in Figure 6.19. The nourishment extends out to the nearshore breakwaters as tombolos forming a series of small pocket beaches.

Figure 7.18. Offshore segmented breakwaters with tombolos in beach nourishment protecting a highway (Louisiana Highway 82, Holly Beach) (American Shore and Beach Preservation Association photo, circa 2003).

Figure 7.19 and Figure 7.20 show another system that uses nearshore segmented breakwaters and nourishment sand. In this system, tombolos do not form, the beach does not extend out to the breakwaters. The bulges in the shoreline in the lee of the breakwaters are called “salients.” This system reduces longshore sand transport in the lee of the breakwaters. The tombolos of the headland breakwater system shown in Figure 7.18 eliminate longshore sand transport, inside the breakwaters during normal conditions.

The formation of salients or tombolos is controlled by the geometry of the breakwater system as shown in Figure 7.21. The Coastal Engineering Manual (USACE 2002) provides more guidance on the functional design of nearshore segmented breakwaters.

Figure 7.19. Offshore segmented breakwaters with salients in beach nourishment (USACE archive photo, circa 1980).

Figure 7.22 shows a nearshore segmented breakwater system with terminal groins used to build a small recreation beach. The beach was created with nourishment on the bay side of a long seawall that protected a road but did not have any sandy beach. The breakwater and groin structure system were designed to retain the nourishment sand. The beach was built to provide access to the bay for wind surfers and others.

Figure 7.23 and Figure 7.24 show headland breakwater-pocket beach systems designed to retain beach nourishment sands on bay shorelines. Both were constructed in front of seawalls that had previously been damaged by erosion. These headland breakwater-pocket beach systems use structures to retain sand by providing artificial headlands. Figure 7.23 shows a headland breakwater that incorporates a “t-head groin” in the middle. The structures in Figure 7.24 do not include the stem of the “t” because tombolos were expected to form. The State DOT was a partial sponsor of the project in Figure 7.24 since the system protected a short stretch of road.

Figure 7.20. Offshore segmented breakwater system at Presque Isle, Pennsylvania

Functional design parameters for the design of headland breakwater-pocket beach systems include the distance offshore as well as the gap spacing. The functional goal is the creation of a pocket beach with the sand fill. The shorelines as shown in Figure 7.23 and Figure 7.24 are curved because they have responded to the wave energy coming through the gaps in the artificial headland structures. Wave heights and directions are modified as waves diffract through the gaps. The structure layout can essentially be "tuned" to the local, site-specific wave climate to produce a beach with a desired curved shape and width (Bodge 1998). More guidance for the design of these systems including methods for estimating the final equilibrium shoreline shape and location are given in Silvester and Hsu (1993) and Hardaway and Gunn (2000). An experienced, qualified coastal engineer is recommended for the design of these solutions which combine nourishment and structures.

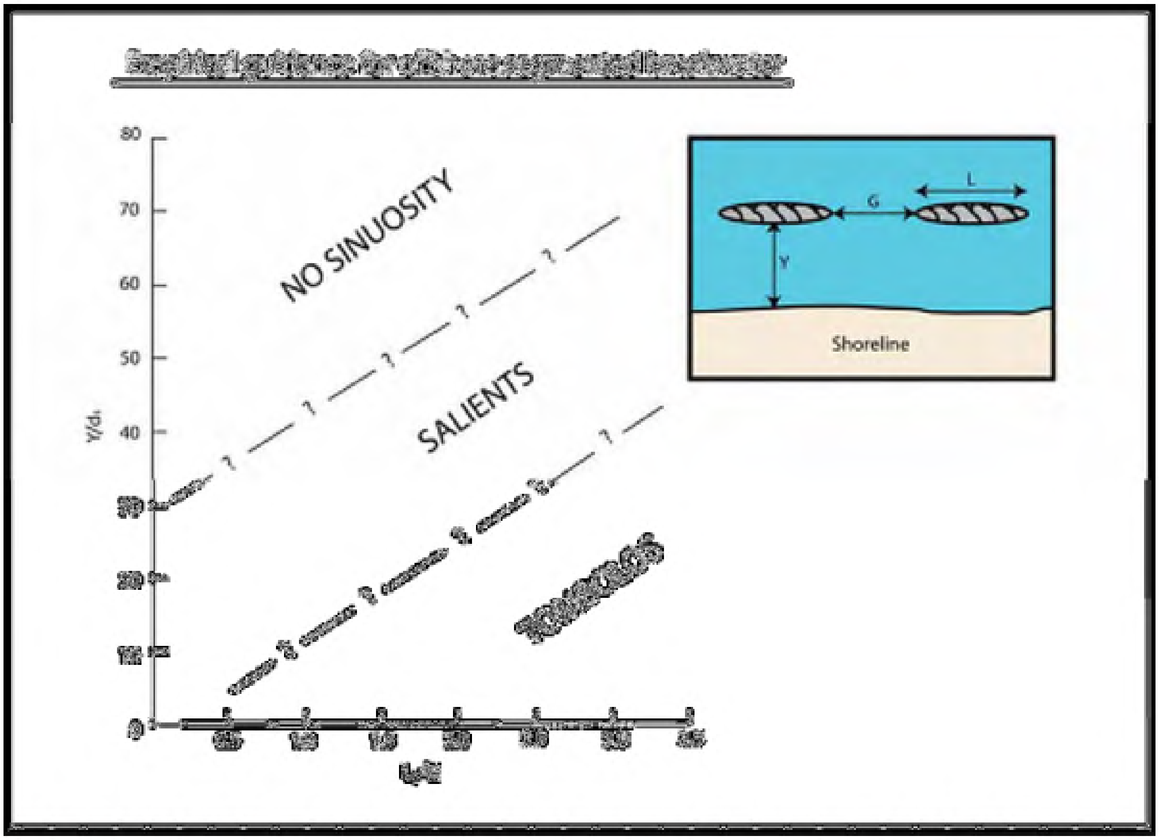

Figure 7.21. Empirical guidance for shoreline effect of offshore segmented breakwaters. (after Pope and Dean 1986, and USACE 2002).

Figure 7.22. Offshore segmented breakwaters with groins and beach nourishment on Corpus Christi Bay (Ocean Drive, Corpus Christi, Texas).

Figure 7.23. Constructed pocket beach stabilized with a T-head groin breakwater system (Point Clear, Alabama).

Figure 7.24. Beach nourishment project stabilized as pocket beaches with a headland breakwater system protecting a road (Water Street, Yorktown, Virginia)

7.6.3 Non-traditional/Innovative Solutions

The history of coastal engineering has seen many innovative attempts at shore protection and stabilization solutions. These have included many expensive, patented devices and systems. Each of these innovative approaches functions differently but all must follow the same general principles of physics including mass (sand) conservation. If the placement of a device or apparatus in the surf causes beach sands to deposit, it functions much like the more traditional structures described above.

Some of the innovative solutions to beach erosion that have been tried are artificial seaweed, used tire breakwaters, different types and shapes of rigid submerged and emergent devices and beach dewatering. Most innovative solutions are serious attempts to address a challenging problem but some are unproven and highly questionable. Unproven, innovative shore protection solutions for highway applications should be pursued very judiciously.

While the evaluation of new innovative solutions to beach erosion problems should continue in the research and development community, prudent engineering planning and design should focus on proven solutions: relocation, nourishment, structures or some combination of those approaches.

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Chapter 8 - Highway Overwashing

8.1 Description of Issue

Some roads are flooded and damaged by coastal storm surges because of their nearshore location and low elevation. An example is shown in Figure 8.1, North Carolina Highway 12, which provides access along the Outer Banks barrier island chain.

During a Thanksgiving Day 2006 storm a portion of NC 12 was being overwashed due to storm surge and waves. NCDOT personnel attempting to keep the road open are visible to the right of the photo. A small, recently constructed sand dune is shown at the left of the photo. Individual waves are washing across the road in the center of the photo and a new deposit of sand is visible on the barrier island.

Figure 8.1. A coastal road being overwashed during a storm. North Carolina Highway 12, November 23, 2006 (North Carolina DOT photo)

Post-storm damage from overwashing during Hurricane Ivan (2004) is shown in Figure 8.2. The road pavement elevation was about +8 feet (NAVD) and the storm surge peak from Hurricane Ivan was roughly 11 feet (NAVD). This chapter outlines mechanisms causing damage to pavements due to overwash and suggests strategies to minimize damage.

Figure 8.2. Example of pavement damage due to storm surge. Florida 292 on Perdido Key, Florida after Hurricane Ivan (Sept. 2004)

8.2 The Coastal Weir-Flow-Damage Mechanism

There are several mechanisms that damage pavements subject to overwash. One is direct wave attack on the seaward shoulder of the road. Another is flow across the road and down the landward shoulder. This is a “weir-flow” damage mechanism. A third mechanism is flow parallel to the road as water moves to “breaches” or lower spots in the road as the storm surge recedes.

Paradoxically, much of the damage to road pavements observed after Hurricane Ivan (2004) was on the landward side of the road. The Gulf of Mexico is to the right side of Figure 8.2 (behind the buildings). Figure 8.3 shows another example of similar damage. There was partial pavement undermining on the landward side of the road. Hurricane Ivan damaged over 50 miles of roads with partial damage as shown or complete damage. It is speculated that weir-flow was the primary cause of the failure mode with contributions from parallel flow.

Figure 8.3. Example of pavement damaged by Hurricane Ivan. (photo looking west on Florida 399, J. Earle Bowden Way, Gulf Islands National Seashore, September 2005)

The specifics of the damage mechanism are: the road embankment acts like a broad-crested weir to the incoming storm surge and the pavement is essentially the crest of the broad-crested weir. As the surge elevation exceeds the elevation of the crown of the road, water flows across the road. Flow across a broad-crested weir passes through the critical flow. Flow down the landward shoulder is super-critical. Supercritical flows scour the shoulder material. If the scour reaches the edge of the pavement, water continuing to flow over the edge of the pavement forms a hydraulic jump and undermines the pavement. The same mechanism is scour caused by flow down the seaward shoulder later in the storm as the surge returns to the sea.

The same general mechanism is responsible for damage to road embankments in a riverine environment (Chen and Anderson 1987, Clopper and Chen 1988). Figure 8.4 shows the general flow regimes that are established when a roadway embankment is overtopped. Damage can occur with or without tailwater (see Figure 8.5).

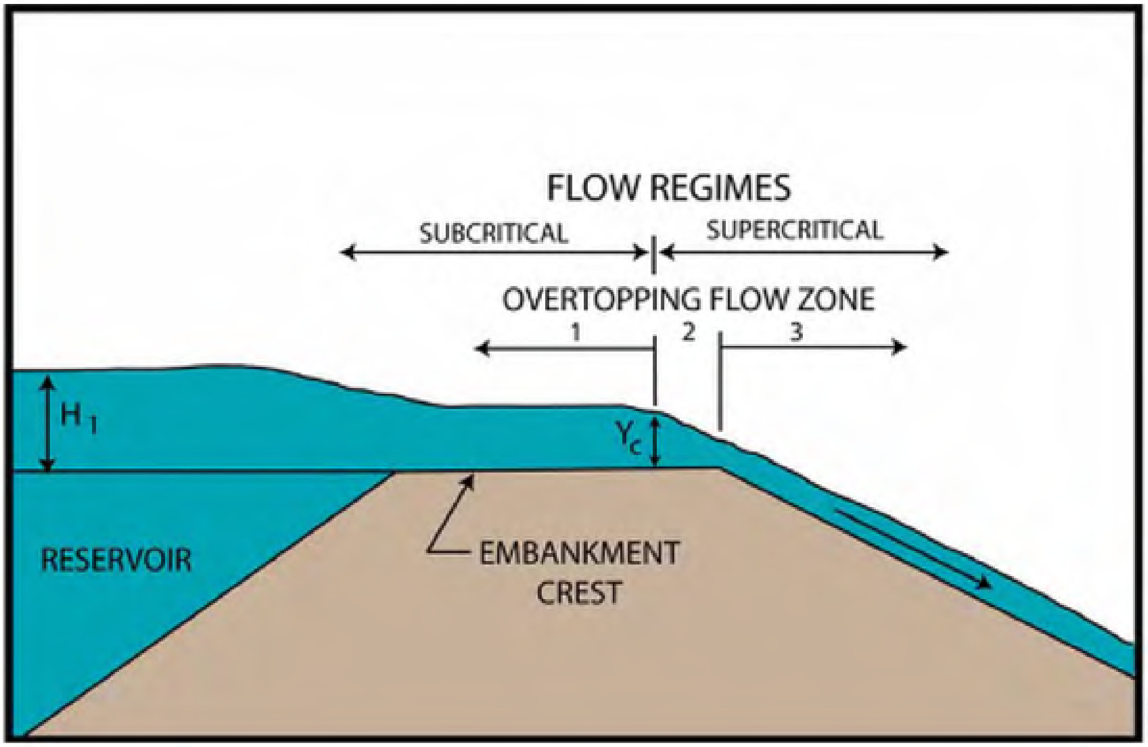

Figure 8.4. Flow regimes leading to failure of embankments in riverine flooding situations (after Clopper and Chen 1988).

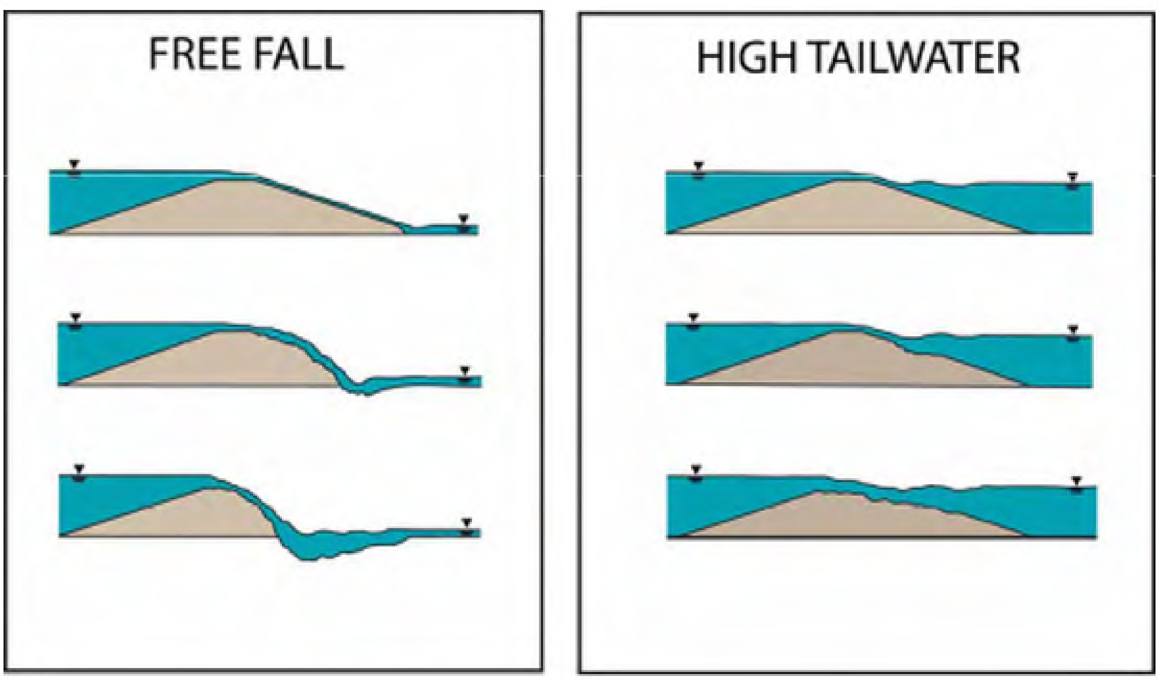

Figure 8.5. Embankment failure mechanisms (after Clopper and Chen 1988)

Figure 8.6 shows a road destroyed during Tropical Storm Arlene (June 2005). This road was under construction after having been destroyed the previous year by Hurricane Ivan (September 2004). Hurricane Ivan removed all the sand dunes and allowed this portion of the barrier island to overwash during smaller storms.

Figure 8.6. Pavement destroyed by the weir-flow mechanism (Ft. Pickens Road, Gulf Islands National Seashore, near Pensacola, Florida).

During several small storms in 2005, weir-flow was observed. Prior to those storms, the barrier islands were typically evacuated during major storms and the islands had sand dunes that prevented overwash during minor storms. Figure 8.7 and Figure 8.8 show the mechanism at two different locations during Tropical Storm Cindy (July 2005). The storm surge flow direction is from the ocean to a bay in both pictures. Flow is from right to left across the pavement in Figure 8.7 and in the opposite direction in Figure 8.8. There is a small hydraulic jump on the downstream side in each picture due to the elevation drop across the edge of the pavement.

Figure 8.7. Weir-flow damage beginning. (Florida 399, Fort Pickens Road, Gulf Islands National Seashore; July 2005; FHWA photo).

Figure 8.8. Weir-flow damage occurring. (Florida 399, Fort Pickens Road, Gulf Islands National Seashore; July 2005; FHWA photo).

8.2.1 Coastal Weir-Flow Damage Mechanism Investigations

The coastal weir-flow damage mechanism has been investigated at prototype-scale in a laboratory in an FHWA-funded study conducted jointly by the University of South Alabama (USA) and Texas A&M University (TAMU). Figure 8.9 shows a schematic of the laboratory set-up and Figure 8.10 and Figure 8.11 show schematics of the results from tests conducted in June 2005 at the Haynes Coastal Engineering Laboratory at TAMU. The experiment was conducted in a 12-foot wide and 10-foot deep flume. A sandy road embankment was constructed in the flume with a roadway on its crest consisting of 12, 2-foot wide concrete slabs. The sand shoulders were unconsolidated typical of many coastal highways. Water was pumped across the road section until failure as shown in Figure 8.10 and Figure 8.11. Figure 8.12 and Figure 8.13 show the failure. The USA/TAMU tests showed that the weir-flow is the likely cause of pavement damage observed in post-storm damage assessments. The damage can occur with only a little depth of water flowing across the road.

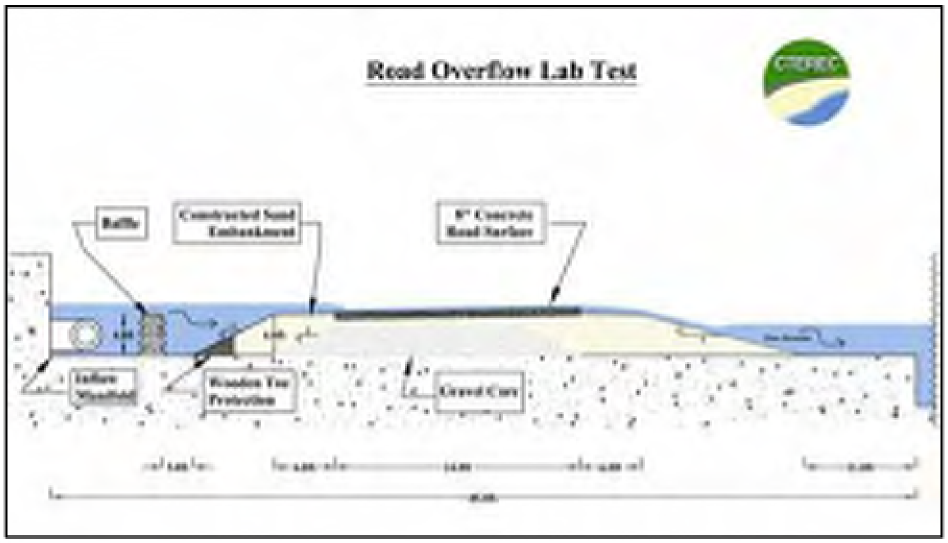

Figure 8.9. USA/TAMU laboratory experiment model setup schematic

Figure 8.10. Schematic of USA/TAMU laboratory experiments test run one result

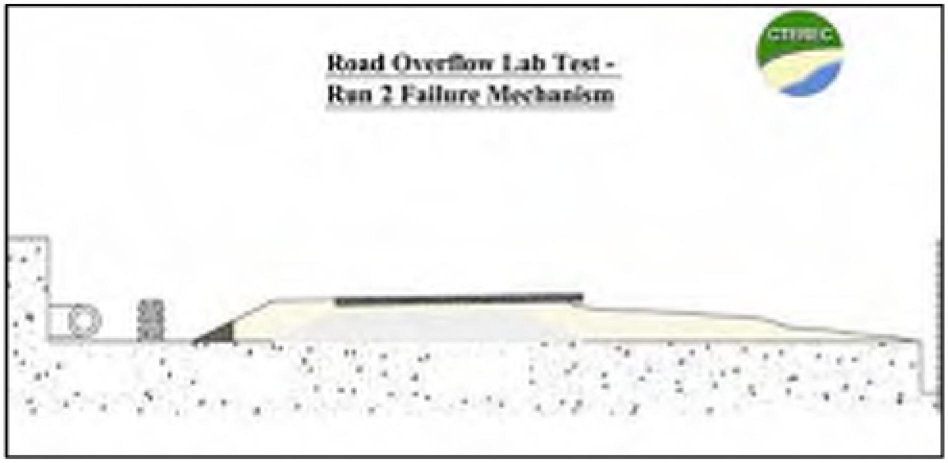

Figure 8.11. Schematic of USA/TAMU laboratory experiments test run two result

Figure 8.12. Laboratory tests of the weir-flow damage mechanism showing scour destroying the downstream shoulder and beginning to undermine the edge of pavement. (USA/TAMU flume tests, June 2005).

Figure 8.13. Laboratory tests of the weir-flow damage mechanism showing scour has continued to point of undermining failure of 3 sections (6 feet) of roadway surface. (USA/TAMU flume tests, June 2005).

It is likely that waves exacerbate the weir-flow damage mechanism. Waves moving across the pavement on the storm surge will increase the instantaneous flow velocities on the downstream shoulder which lead to more scour. No guidance is available to estimate scour due to this phenomenon at this time. Some levee failures in the greater New Orleans area during Hurricane Katrina were also due to downstream erosion due to overtopping waves.

Clopper and Chen (1988) discuss uplift on overtopped pavements on a riverine embankment. Uplifting may be an even greater problem in the coastal environment because of the easier transmittal of pore-pressure under the pavement due to the sandy nature of the coastal road bases. There was some evidence of pavement lifting during Hurricane Ivan as shown in Figure 8.14.

The same weir-flow mechanism that can damage the landward shoulder of a coastal road can damage the seaward shoulder too. Later in the storm, as the storm surge recedes, the water elevation on the landward side of the road embankment may be higher than the elevation on the seaward side. Flow is back to the sea and the downstream shoulder is now the seaward shoulder. Figure 8.15 shows pavement damage likely due to return flow.

Another related damage mechanism is parallel flow (parallel to the road direction) along the landward side of the coastal highway embankment as the storm surge recedes. Late in the storm, the embankment can begin to act like a dam holding the flood waters on the barrier island. If a portion of the embankment is lower due to failure or breaching, then water will flow laterally toward the low spot in the embankment. This flow scours the foundation material along the shoulder and contributes to its damage or failure. Lateral flow along the shoulders during coastal storms has been observed by Florida DOT personnel at US 98 near Destin, Florida. There is post-storm evidence of this flow in many locations (including the photo shown in Figure 8.2).

Figure 8.14. Pavement moved landward by overwash processes. (Gulf Islands National Seashore, Perdido Key, Florida after Hurricane Ivan; 2004).

Figure 8.15. Evidence of weir-flow damage to the seaward edge of pavement due to return flow late in the storm (West Beach Blvd., Alabama 182, Hurricane Ivan, Gulf Shores, Alabama).

8.3 Strategies for Roads that Overwash

Four strategies for minimizing pavement damage due to overwash have been successful for coast-parallel roads on barrier islands. They can be used in combination with each other:

1. re-locating the road to a portion of the barrier island where sand will likely bury the road during overwash,
2. lowering the elevation of the road to be at or below much of the existing grade to encourage burial by sand during overwash,
3. constructing a sand dune seaward of the road to reduce the likelihood of overwashing and to provide a reservoir of sand to bury the pavement when overwashing occurs.
4. armoring of the shoulders of the road to resist erosion during overwashing.

8.3.1 Road Location Considerations

Storm overwash on barrier islands often naturally erodes elevation from the front portion of the island and deposits sand on the landward portion of the island. This process is shown schematically in Figure 8.16. Frontal dunes are often the highest elevations on a barrier island. These dunes and the beach berm seaward of them often erode in major storms through dune erosion and overwash processes. Sand is pulled offshore until the dune crest is breached or overtopped by the storm surge. Then sand moves landward and is deposited in lower elevations on the back of the island. These deposits, called overwash fans, can extend back into the bay.

Figure 8.16. Schematic of sand erosion and deposition on a barrier island resulting from overwash.

If the road way is located where cross-section erodes, it will be subjected to severe wave attack and scour. If, however, it is located in the deposition zone, it can be buried by sand early in the overwashing event. Some roads, found under this layer of sand after a coastal storm, have been undamaged. A bulldozer blade can scrape the sand off the road and the road can be opened to traffic shortly after the storm.

8.3.2 Road Elevation Considerations

Another approach to reducing damage due to the weir-flow is to lower the elevation of the road to at or below adjacent ground elevations. This can prevent the weir flow from occurring since the crest of the pavement is not the highest portion of the grade. Figure 8.17 shows a road buried by overwash sand that survived a major hurricane. The piles of sand along the road were scraped off the road as part of the post-storm maintenance. There is some practical limit to lowering the road which depends on drainage and safety. Lower roads may also require more maintenance such as sand sweeping. Installation of sand fencing and vegetation can significantly reduce drifting sand and the frequency of sweeping requirements. Experience in west Florida suggests that constructing a typical road embankment elevated above the adjacent ground elevations can result in significant damage even if the road is relocated away from the ocean.

Figure 8.17. Example of road buried by overwash and opened by plowing sand off

8.3.3 Construction of Sand Dunes

Sand dunes can be encouraged or constructed seaward of roads to reduce the likelihood of overwashing and to provide a reservoir of sand to bury the pavement when overwashing occurs. Many states and local government have attempted to construct sand dunes seaward of roads to protect against storm surge and waves. North Carolina has used this approach to protect portions of North Carolina Highway 12 along the Outer Banks. Figure 8.18 shows a portion of that highway north of Buxton, North Carolina, where a large, artificial sand dune has been constructed on the seaward side of the highway.

Figure 8.18. Artificial sand dune constructed seaward of a highway to protect the highway (North Carolina Highway 12)

Dune erosion modeling tools can be used to design the size and shape of the dune. Construction of a healthy sand dune usually requires vegetative plantings to stabilize the dune and to establish a dune that functions like a natural dune.

All three of the above approaches to reducing damage to pavements during overwashing can be implemented together. The schematic of Figure 8.19 shows a new road located as far from the ocean as practical, built at a low elevation, with small dunes constructed near it. The dune vegetation also acts to reduce wind-blown sand from covering the road during normal (non-storm) conditions.

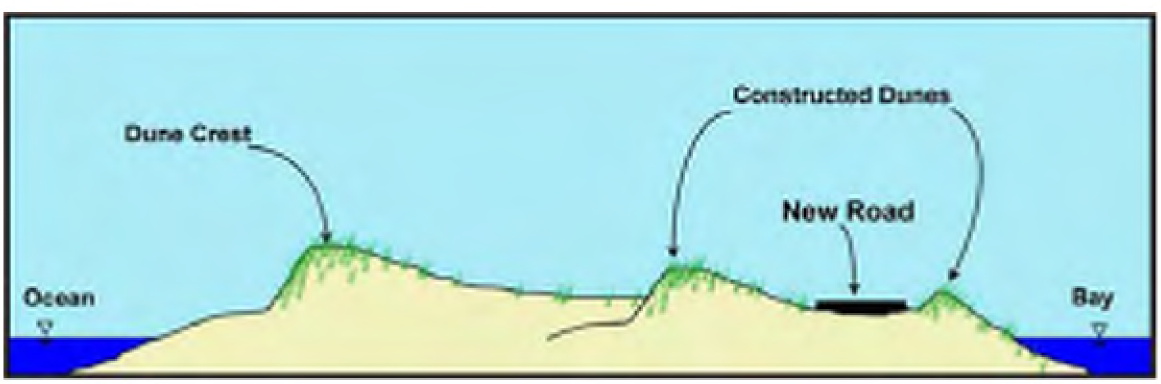

Figure 8.19. Schematic summarizing three approaches (bayward location, low elevation, constructed sand dunes near road) to minimize damage to roads that overwash.

8.3.4 Armoring of Shoulders

The downstream shoulder of roads that experience overwashing damage can be armored to withstand high velocity flows. This approach has been adopted to protect a section of US Highway 98 along the Florida coast west of Destin. The armoring includes sheet piling (Figure 8.20) and gabions (Figure 8.21). The sheet piling is located on the shoulder of the pavement farthest from the sea. This is the edge of pavement that has suffered the most damage due to the overwash mechanism in past hurricanes. Buried gabions are used where the overwashing flow may be lower but parallel to the road during the storm is expected to be strong enough to cause damage. This design was constructed in 2005, after Hurricane Ivan, and had not been tested by a major overwashing event at the time this document was written.

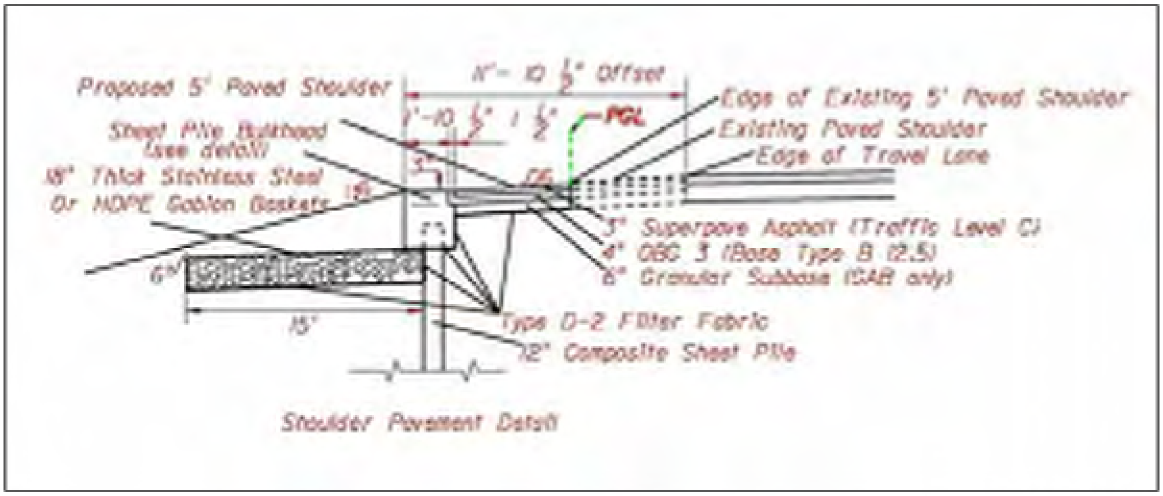

Figure 8.20. Sheet pile, with buried gabions for scour protection, at edge of pavement to resist pavement damage due to coastal storm surge overwash. (Florida DOT figure).

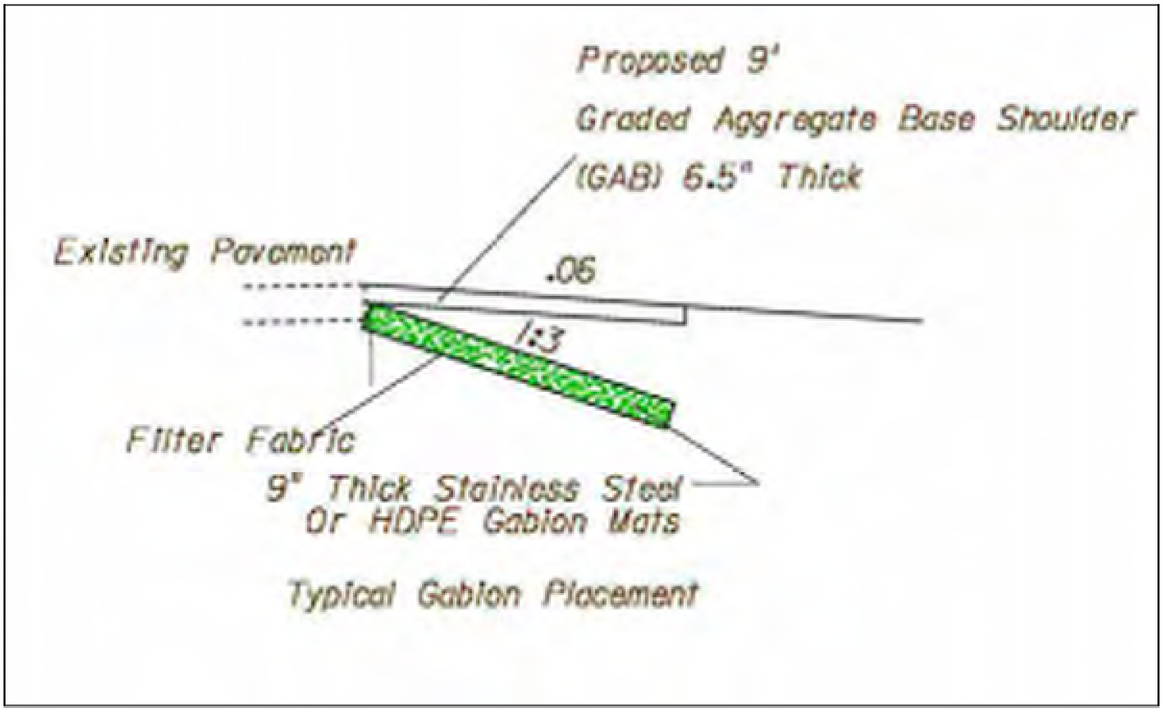

Figure 8.21. Gabions at edge of pavement to resist pavement damage due to coastal storm surge overwash. (Florida DOT figure)

Clopper and Chen (1988) provide guidance for armoring shoulders that might be applicable to the coastal problem. They conducted laboratory experiments on different possible countermeasures to resist the flow of water across a highway embankment. Their tests were based on riverine overflow situations and focused on soil types not as sandy as those typically found at the coast. They only considered current flow forces and not wave forces. However, Clopper and Chen (1988) found that a concrete block revetment system with relatively heavy blocks, horizontal and vertical interlocking cables, and anchors was able to resist the hydraulic forces due to overtopping better than a number of other alternatives. They tested flow rates generated by up to 4 feet of differential head over the embankment. Figure 8.22 is a sketch of how that concept could be implemented as a retrofit to protect a coastal highway. The capabilities of interlocking blocks to withstand the overtopping condition was confirmed by laboratory tests by Clopper (1989).

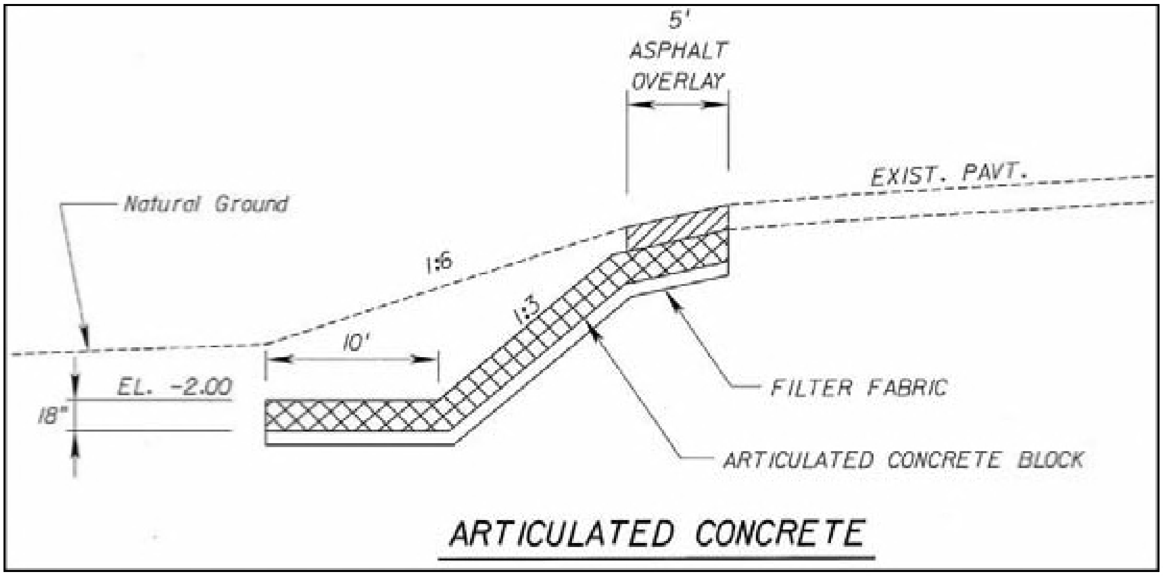

Figure 8.22. Conceptual design to resist pavement damage due to coastal storm surge overwash

Chapter 9 - Coastal Bridges

The FHWA estimates that there are over 36,000 bridges located within 15 miles of coastal waters of the United States (FHWA 2007). While a notable number of structures, this only represents 6 percent of the approximately 600,000 bridges contained within the National Bridge Inventory (NBI) (FHWA 2007)[3].

For perhaps this reason, many SDOT drainage manuals apply riverine based hydraulic design concepts and approaches to these coastal bridges. For example, flow and water surface elevations at riverine bridges can frequently be fairly well represented by assuming steady uniform flow with reasonably long flow durations. This justifies use of relatively straight forward hydrologic approaches (regression equations, rainfall/runoff models) to develop peak design flows. Likewise, given these peak flows, practitioners generally use steady flow, one-dimensional models to estimate velocities, backwater, and other hydraulic design constituents.

However, the complicated hydrologic and hydraulic processes in the coastal environment may render such assumptions inappropriate for coastal bridges. Astronomical tides have reversing flows and may also have substantial ranges. These result in associated depths and velocities that vary significantly over a relatively short period of time. In addition to tidal fluctuations, hydraulic analyses need to consider and determine design storm surge and design wave heights, increasing the complexity.

Typical modeling assumptions and approaches (i.e., use of steady flow, one-dimensional models) usually do not apply to coastal bridges and may lead to problematic results and interpretations. For example, some analyses attempt to equate design flow and design surge elevation. This is a faulty assumption. During a flood event in a riverine system, the channel cross sections defining the floodplain also provide the limits of flow conveyance and thus the associated flow depth of that flood (i.e., flood quantity determines water elevation). During a design surge event, the water levels extend over a much larger geographical area with water depths limited by those factors described in section 3.2, "Storm Surge." Therefore, at any particular location, the water elevation (head) determines flow quantity (i.e., water elevation determines flow). Additionally, as described earlier, the highly time dependent nature of coastal hydrologic and hydraulic processes (described above) preclude steady flow approaches, adding intricacy to the modeling effort.

Coastal bridge complexities are not just related to hydrologic and hydraulic processes. The orientation of the coastal bridge to "flow" direction may be quite different than a typical riverine bridge. At such riverine bridges the goal is to place the bridge as perpendicular as possible to the natural design flood flows direction. In many cases coastal bridges are not transverse the stream thalweg, but are in-line with the direction of the surge. Do such surges induce velocities sufficient for scour formation? Or a bridge located within an embayment may be, depending on storm direction, be subject to wave scour or wave loads, whereas for other storm directions, the bridge could be reasonably safe.

Therefore, even more so than riverine bridges, the level of engineering for coastal bridges requires consideration of forces and processes unique to the coastal environment including tidal bridge scour potential and hydrodynamic loads from waves and tidal currents. Wave and current

[3] The 36,000 bridge estimate also does not include bridges and culvert systems with less than a 20 foot span (nor are these smaller spans included in the NBI).

loads on the sub-structure components of coastal bridges such as piles, pile caps, etc. are unavoidable and require investigation.

This Chapter provides an overview on several related hydraulic aspects of bridges in the coastal environment. These include the location of the bridge within the coastal floodplain, coastal bridge scour, coastal wave loads, and other important issues.

9.1 Locations of Coastal Bridges

Coastal bridges can be found at four general locations within the coastal environment: inlets, causeways, tidal arms/embayments, and river mouth crossings (Figure 9.1). Each type of location presents different issues and challenges for the hydraulic and coastal practitioner.

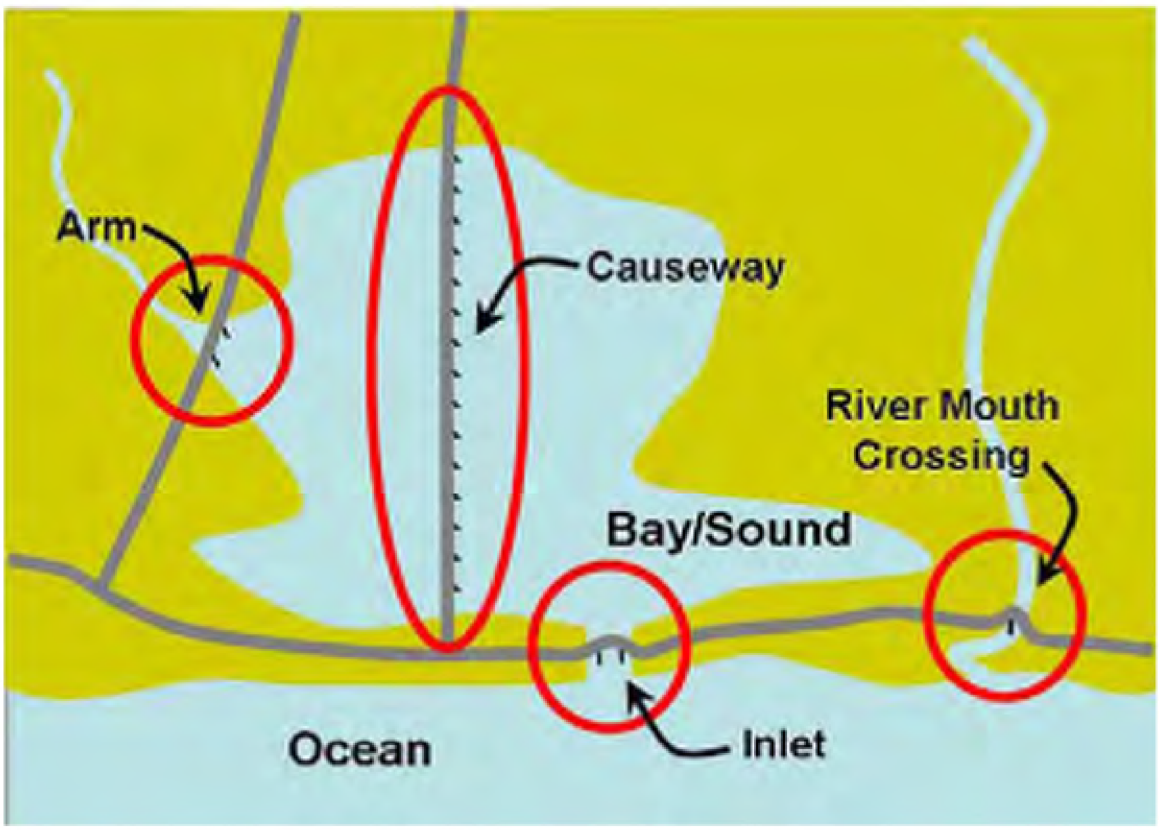

Figure 9.1. Conceptual schematic of four typical bridge locations within the coastal environment.

9.1.1 Bridges at Inlets

Inlets are where the tides move between the ocean and a bay (see Section 5.6, "Tidal Inlets"). Inlets are the entrance to many estuaries and other water bodies of ecological importance. These interior bays and estuaries can store significant volumes of water. Inlets experience complex hydrodynamics, some with extremely intricate interactions between currents and sands. Most shoreline change is near inlets; many are "evolving" geologically in response to engineering and natural changes. There can be multiple inlets to the interior water body (bay, sound, etc).

The United States has over 600 tidal inlets, of which many have bridges across their throat. These can range from very large structures (e.g., Golden Gate Bridge) to relatively small spans (Gulf Shores, Alabama; seen in Figure 9.2). Depending on the exact configuration of the bridge and tidal inlet, the bridge will exhibit varying levels of hydraulic control between the ocean and interior water body. Even bridges "spanning" the inlet during daily astronomical tides fluctuations may exhibit some hydraulic control when surge and wave levels reach a certain point.

Figure 9.2. Bridge spanning small inlet (SH 182 in Gulf Shores, Alabama).

9.1.2 Bridge Causeways

Causeway bridges typically link coastal and barrier islands and peninsulas to the mainland. They can consist of bridges or a combination of bridges and elevated embankments. The floodplain crossing can be some combination of open water and wetlands (or other low-lying crossing floodplain system. For example, Figure 9.3 depicts the Ben Sawyer causeway bridge near Charleston, South Carolina. The causeway bridge leads from Mount Pleasant (mainland) to Sullivan's Island. The causeway bridge (like many such structures) serves as an evacuation route during storm events. The NBI records the actual bridge length as 1150 feet, straddling the Intercoastal Waterway (FHWA 2007)[4]. However, the embankment portion of the causeway extends many thousands of feet (arrows differentiate between bridge and embankment).

This causeway bridge also illustrates some of the hydrologic and hydraulic complexities affiliated with such structures - direction of surge relative to the bridge and orientation. The FEMA FIS flood insurance rate map (FIRM) describes that the 100-year stillwater surge elevation will reach 14 to 15 feet (plus any additional waves on top of that stillwater) (FEMA 2004). However, the FIRM appears to indicate the surge direction is roughly perpendicular with

[4] The Ben Sawyer bridge was damaged by Hurricane Hugo in 1989.

the direction of beach orientation – surge "moves" in the same (longitudinal) direction as the bridge and embankment. While the Ben Sawyer causeway bridge crosses marsh regions, other causeway bridges may span a lake, sound or other open water body (making them occasionally prone to wind and fetch affiliated wave issues, depending on storm wind direction). Some implications related to coastal bridges will be described later in this Chapter.

Figure 9.3. Ben Sawyer causeway bridge between Mount Pleasant and Sullivan's Island, SC.

9.1.3 Bridges spanning Tidal arms / Embayments

A common location for coastal bridge crossings are found on tidal arms or embayments. As opposed to inlet bridges, these bridges are located in interior water bodies or a distance "upstream" on an open bay or estuary. These are also distinct from causeway bridges in that they are in open waters and more likely subject to wave action and wave transformations[5]. Examples can range from a small tidally influenced creek to large tidally influenced waterbodies such as Mobile Bay (Alabama), Knik Arm (Alaska), and most of the major bridges affected by Hurricanes Ivan and Katrina.

These locations might also include bridges upstream of rivers with confluence to the ocean, bay, or other large water body. Such rivers are still tidally influenced and, just as importantly, have some storage capacity. Examples of such locations are the Columbia River, Hudson River, Cooper River, etc.

[5] Wave transformation described in greater detail in section 4.2, "Wave Transformation and Breaking."

The bridges at these locations can vary in width and span length – from 2 lane, 20 foot spans over a tidal creek to multi-mile Interstate spans. As will be described later in this Chapter, the size, orientation, and potential surge and wave effects dictate the level of analyses needed at such bridge locations.

9.1.4 River Mouth Bridge Crossings

Along the West Coast of the United States are numerous bridges crossing at or near the mouths of smaller river and creeks. These rivers differ from the other locations described above because the local geographical features (mostly hills and mountains extending to the shoreline) often result in a narrower floodplain. These in turn affect the available storage and the extent of the tidal prism. Figure 9.4 depicts four of these types of crossings (Figure 2.4 also provided an example of a bridge and river mouth crossing).

Big Creek in Oregon

Pistol River in Oregon

Redwood Creek in California

Yachats River in Oregon

Figure 9.4. West Coast River Mouth Crossings.

Some of these rivers carry a notable sediment load to the littoral zone. These rivers and creeks may exhibit severe lateral migration, especially within the backshore beach zone. Breakwaters are constructed at some river mouths to control this migration and provide other stabilization measures. When encountering such situations, good practice would be to consult with a qualified coastal engineer (see section 2.6, "Coastal Engineering as a Specialty Area").

9.2 Coastal Bridge Scour

Scour is the most common cause of bridge failures in the United States. Bridge scour is the erosion caused by water of the soil surrounding a bridge piers and abutments.

Research has produced a vast body of knowledge for evaluating and estimating scour at bridges. Mostly oriented towards the riverine environment, research represents riverine conditions by assuming steady uniform flow with reasonably long flow durations.

Recommended procedures for estimating scour at these bridges rely heavily on these assumptions. The FHWA has produced the document HEC-18 "Evaluating Scour at Bridges" (fourth edition) (HEC-18) (Richardson and Davis 2001), as well as other documents and material to provide guidelines for designing new bridges to resist scour, evaluating existing bridges for vulnerability to scour, inspecting bridges for scour, and improving the state-of-practice of estimating scour at bridges.

9.2.1 Coastal Bridge Scour Policy, Guidance, and Research

Significant resources have been devoted to the bridge scour problem, yielding a growing body of knowledge and products. The FHWA uses these products to develop and provide national scour policy and guidance.

The position of the FHWA is that these policies and guidance cover both riverine and coastal situations. However, the FHWA also recognizes that conditions in the coastal environment may necessitate moving away from a "one size fits all" technical approach in certain case-by-case situations. Of vital importance when considering deviating from these national approaches is that the SDOT recognize the risk associated with the scour methods to be applied to a specific project. This risk assessment includes endorsement by the local FHWA Division Offices and, as needed, knowledgeable scour experts.

Appendix D provides some background and commentary on coastal scour related policy and guidance, including scour estimation and potential countermeasures. Appendix D also provides a brief synopsis of some relevant research efforts.

9.2.2 Coastal Bridge Scour Hydrology and Hydraulics

For coastal bridges, the applicable hydrology and hydraulics are influenced by waves, tides, storm surges, longshore sand transport, inlet dynamics and stability, and other coastal processes. Therefore, before any scour analyses occur, the practitioner needs to resolve these technical issues, including some especially relevant to bridges over coastal waters.

Hurricane storm surges often produce extreme flow conditions for time periods of only a few hours. This leads to an observation of another important difference between riverine and coastal bridge hydraulics – the distinction in analyzing coastal flood conditions and scour conditions.

Coastal flooding will manifest itself in several ways: first the effects of the storm surge (and waves) on the coastal floodplain. Since coastal areas are generally at low elevations and flat, the extent of the flooding is widespread – inundating properties, infrastructure, and open spaces. Secondly, the elevated surge acts as a downstream control for storm related rainfall runoff. Until the surge has receded, this runoff does not have anywhere else to go, increasing the backwater and flooding effects. This flooding may occur over some time – possibly more time than the storm surge duration. Additionally, the probability of exceedance of the resulting flood level may be much greater than the frequency of both the storm surge event and the rainfall event, so a storm with a 10-year surge and 15-year rainfall might combine to produce a 100-year flooding event.

These optimal flooding conditions may not necessarily be the same conditions as those that would produce the worst scour. This is because when comparing the effects of the two primary hydraulic variables associated with scour – velocity and water depth – velocity has a greater role.

Therefore, optimal coastal scour formation conditions likely occur when the velocity would be the greatest value. Specifically occurring during two situations: first, when surge is entering the inlet or embayment at the fastest; and secondly, during the recessional period, when combined surge and the storm affiliated rainfall flows back to the ocean (similar to the weir-flow damage mechanism discussed in section 8.2).

To model these conceptual conditions, for hydrologic boundary conditions, a conservative scour analysis would (1) consider a surge "hydrograph" having a short duration entering into a bay while the bay was at MLLW; (2) consider a design runoff hydrograph (including residual surge volume) returning to the ocean at MLLW. Clearly, this approach is conservative, which is why larger studies often apply more refined techniques (see section 3.2, "Storm Surge" and section 9.5, "Selection of Design Storm Surge & Design Wave Heights").

Once the design parameters have been determined it is necessary to estimate the magnitudes of flow depths and velocities (and possibly other values as well). The determination of flow parameters for coastal bridges almost always require the use of a surface water model that can analyze unsteady flows. HEC-18 describes a "three-level qualitative approach" protocol to assist in defining the amount of required analyses. Once again, consultation with a qualified coastal engineer can serve to refine this overall protocol.

9.2.2.1 Level One Approach

The use of a HEC-18 based level one qualitative approach is never suitable for coastal bridge hydraulic design or scour estimates on its own. However, a level one approach can be useful in determine the potential level of effort required for a specific project.

9.2.2.2 Level Two Approach

The use of a level two (tidal prism) approach is suitable only for smaller bridges or low ADT bridges in well protected tidal arms and embayments. The use of this approach is not recommended for bridges at inlets or causeway bridges.

The range used in the analysis should be combination of the highest daily astronomical tidal elevation (MHHW) and design event storm surge still-water-level (if not already combined).

As with the level one approach, a level two analysis can provide generally conservative estimates of potential scour. When applying a tidal prism approach, the areas of uncertainty will be area of the bay, stage-storage characteristics, and the ability to determine the hydraulics performance of the bridge section.

9.2.2.3 Level Three Approach

Level three approaches apply varying degree of analyses. Smaller bridges (or systems of bridges) at a single inlet, or embayments or river mouths can be analyzed with one-dimensional unsteady flow models. The model would apply the hydrologic boundary conditions described above.

Causeway bridges, bridges with unusual configurations, and larger and more complicated bridges (or systems of bridges) require the use of two-dimensional unsteady flow models. Generally, scour analyses of complex piers and bridges necessitate application of two-dimensional numerical hydraulic models.

The tradeoff is that the small amount of additional modeling effort produces additional confidence in the velocity and depth parameters. The potential results of these more site focused values may be smaller foundation elements (new bridges) and reduced scour countermeasure material quantities (existing bridges).

Some specific and critical bridges may require advanced numerical and physical modeling. These advanced numeric models may couple hydrodynamic, wave, and sediment transport modules while the physical model simulates the actual processes using a scaled down version of the physical feature with representative hydrodynamics, waves, and sediments.

Once the flow parameters have been properly determined, they are applied to the various scour types and methods described below to estimate the magnitudes of scour at the bridge.

9.2.3 Types of Coastal Bridge Scour

The types of scour that occur at bridges in the coastal environment include the same general categories (local (pier and abutment) and contraction) as found at riverine bridges. Additionally, coastal bridges can experience scour as a result of wave action (wave scour) and localized areas of high velocities flows. Finally, HEC-18 recognizes that sea-level rise might occur over the life of the structure, so that consideration should also be incorporated into scour analyses. As described below, even for the general categories, the practitioner must consider important caveats and differences associated with the coastal environment.

9.2.3.1 Local (Pier and Abutment) Scour

Local scour includes pier and abutment scour. In riverine local scour mechanisms, the scour hole typically forms near the upstream structure face. Some bed material deposition occurs near the downstream face. Given the flood and ebb associated with the coastal environment, sediment transport mechanisms can differ, resulting a scour hole can forming around the entire pier. Figure 9.5 depicts such an example of scour forming around entire pier. The scour is exacerbated by debris accumulation. Debris accumulation is not uncommon during coastal storm events.

9.2.3.1.1 General Approach for Local Scour

As long as the design hydraulic conditions are determined based on appropriate hydrodynamic methods, local scour equations such as those found in HEC-18 can be applied to coastal bridges. At a minimum this includes sites suitable for level one analysis and smaller coastal bridges in protected embayments.

9.2.3.1.2 Wide and Complex Pier Geometry

HEC-18 includes methods to compute pier scour for standard and complex pier geometries. The HEC-18 equations include wide pier correction factors that may be applicable to bascule piers when the pier is wide in comparison to the flow depth. HEC-18 also outlines a procedure for evaluating scour at complex piers that include a combination of pile groups, piles caps, and piers. Other local scour equations are presented in Hoffman and Verheij (1997), Melville and Coleman (2000) and Sheppard (2003).

Figure 9.5. Scour at a Coastal Bridge Pier

9.2.3.1.3 Time Dependent Local Scour

Time dependent scour equations have been suggested as more appropriate in the coastal environment. In addition to the typical physical processes that are described (Richardson and Davis 2001), the short duration of the typical design storm must be considered. Also, piers that are impacted by waves are subjected to very short duration pressure gradient fluctuations that result in a difficult to quantify shear stress variations.

The University of Florida has conducted research and developed such a set of equations (Gosselin and Sheppard 1998; Miller 2003). The Florida equations require a time-marching solution for the depth of scour adjacent to bridge piers. Input requires time-varying estimates of depth-averaged storm surge velocities at the bridge based on numerical modeling of the hydrodynamics. The Florida equations include calibration coefficients which are primarily based on laboratory investigations. Miller (2003) discusses how the equations can be used to estimate scour at prototype coastal bridges.

Gosselin and Sheppard (1998) concluded that more research is needed before meaningful relationships can be developed for time dependent local scour. 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 laboratory versus prototype 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 dependent local scour amounts. One additional complication is that the time dependent local scour amounts would have to be added to ultimate local scour amounts produced by daily tides.

9.2.3.2 Contraction Scour

In a riverine context, contraction scour involves the removal of material from the bed and banks across all or most of the channel width. This component of scour results from a contraction of the flow area at the bridge which causes an increase in velocity and shear stress on the bed at the bridge. The contraction can be caused by the bridge or from a natural narrowing of the stream channel.

Contraction scour occurs in the coastal environment, but formation can greatly depend on the location and orientation of the bridge (inlet vs. causeway vs. embayment) and embankments. For example, a bridge crossing an inlet on a barrier island may have contraction limited only by the touchdown embankment length. Surge and waves could inundate the roadway approaches and allow water passage at those locations (as well as through the bridge opening).

9.2.3.2.1 General Approach for Contraction Scour

HEC-18 contraction scour equations can be applied to coastal bridges (given similar hydraulic caveats as described for local scour). Contraction scour should be computed based on the live-bed or clear-water equations depending on the velocity of flow approaching the bridge in the un-constricted waterway. The location of the approach flow will depend on whether worst case conditions occur during the flood/ebb tide or surge/post-storm hydraulics.

If astronomical tide currents have high velocities, scour should be computed for these conditions in addition to design velocities produced by hurricane or storm surge conditions. Surges can produce extreme velocities that could produce very deep scour. The HEC-18 equations may be overly conservative for surge conditions because these equations were developed for ultimate scour conditions. While the surge may produce extreme velocity, the high velocity condition may persist for such a short duration that ultimate scour cannot be reached. Additional sediment transport analysis and judgment may be necessary for computing scour in tidal waterways.

9.2.3.2.2 Time Dependent Contraction Scour

Computing contraction scour using procedures outlined in HEC-18 will produce ultimate conditions that 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 live-bed contraction scour equation, which uses a simplification of the Laursen sediment transport equation (Larsen 1960). Sediment transport relationships could also be used directly to compute ultimate contraction scour. 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 should be used."*

A sediment transport model, such as the USACE's HEC-RAS (USACE 2008) could be used to compute ultimate contraction scour conditions for variable flow rates using a stepped hydrograph as long as sufficient simulation duration is used and the steady-state gradually-varied flow assumptions are not violated. It could also be estimated for shorter duration rapidly-varied flow conditions used the unsteady flow modeling capability of the model. Similarly, sediment transport relationships could be used directly to make estimates of the rate of sediment transport. Once the volumetric rate of sediment transport is known, contraction scour hole geometry can be assumed, and the depth of time dependent contraction scour for an assumed storm can be determined.

Figure 9.6 shows the results from a time dependent scour analysis using the approach described above. Figure 9.6 demonstrates scour development through the time required to reach ultimate conditions. It also shows the ultimate scour estimates from HEC-18 (Laursen) and a sediment transport function, and the intermediate value of scour for 3-hour duration. No specific time is associated with the HEC-18 result as it is for "ultimate" conditions.

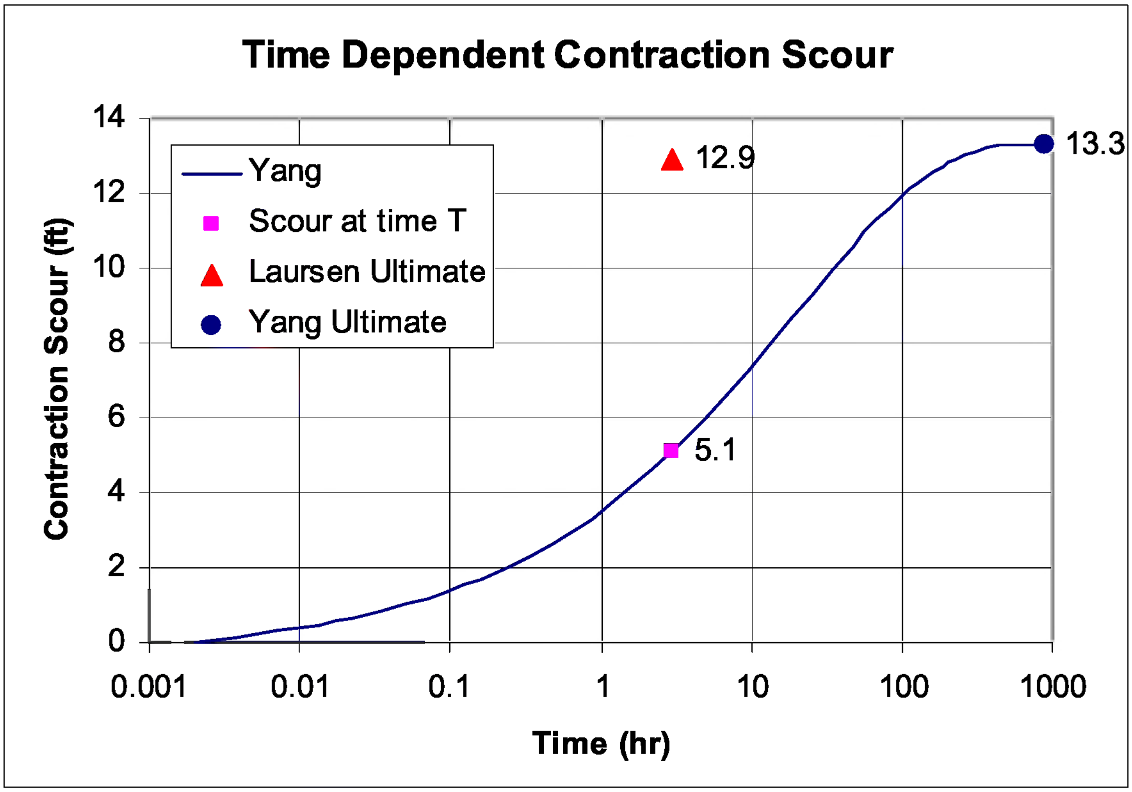

Figure 9.6. Time Dependent Contraction Scour Results (Zevenbergen, et al. 2004)

Figure 9.6 illustrates that 5.1 feet of contraction scour can occur in 3 hours and that it would require approximately 400 hours to reach ultimate contraction scour conditions. The sediment transport function predicts 13.3 feet of ultimate scour compared with 12.9 feet using the HEC-18 equation. Contraction scour for live-bed conditions is generally less extreme than equivalent clear-water conditions. However, live-bed scour reaches ultimate conditions in less time than equivalent clear-water conditions. For relatively small amounts of live-bed scour, three hours can be sufficient to reach the ultimate scour.

This approach of applying sediment transport calculations can result in a prediction of considerably less scour than the HEC-18 equation in some situations. By using the peak hydraulic conditions and steep upstream and downstream scour hole slopes, the method should produce conservative results.

9.2.4 Wave Scour

Wave scour is a phenomenon associated with coastal structures. While bridge specific research is scarce, researchers have conducted experimental investigations into the topic area for many years (Sumer and Froedsoe 1991). Many of the physical formation elements are familiar to those knowledgeable with riverine pier scour: horseshoe vortices in front of the pile, lee wake vortices behind the pile (with and without vortex shedding). However, the presence of waves adds reflection and diffraction, and the possibility of wave breaking into the overall process. Researchers acknowledge the difficulty in modeling this phenomenon, recent efforts have attempted to apply three-dimensional modeling techniques (Umeda 2006; Rouland 2005)

The research tends to suggest that wave scour is less than local scour associated with a constant current or flow (general local scour case). However, the research also indicated that the combination of wave and current might increase the scour rate and increase the total scour depth.

Breaking waves, as might occur during a storm event, would exacerbate the scour. The FHWA and SDOTs have documented several situations where significant scour occurred during severe coastal events. For example, during Hurricane Katrina, normally "dry" portions of the US-90 Biloxi Bay bridge became subject to surge and waves. As seen in Figure 9.7, a large scour hole formed in the vicinity of a pier section. While bridge failure occurred for other reasons (as described in section 9.3, "Coastal Bridge Wave Forces"), the size and extent of the scour hole was significant.

Figure 9.7. Wave scour hole formed by Hurricane Katrina.

9.2.5 Examples of Coastal Bridge Scour

While scour has been reported at all four types of bridge location classes, some of the most problematic scour problems occur at inlets that are changing shape and size as part of their evolution. Inlets are constantly evolving in response to many factors including their initial creation, stabilization with jetties, changes to their bay systems including dredging and filling and causeway construction, and changes to other inlets connected to their bays.

9.2.5.1 Indian River Inlet

Indian River Inlet, Delaware (see Figure 9.8) has experienced progressive scour since it was originally dredged and stabilized with jetties in the 1930's. Scour holes near the bridge piers exceeded depths of 100 feet in 2000. As the inlet has deepened and its minimum cross-sectional throat area increased, more tidal flow has moved thorough it. Thus, its tidal prism has increased. And as the tidal prism has increased, it has continued to scour out the throat area. Essentially, the artificially constructed and stabilized inlet has not reached its evolutionary equilibrium since its original opening in the 1930's.

Figure 9.8. Indian River Inlet, Delaware (USACE photo).

9.2.5.2 Johns Pass

Another inlet with a history of bridge scour issues is Johns Pass, Florida (see Figure 9.9). Johns Pass also is still evolving in response to engineering that occurred decades ago. Most of Florida's inlets have been artificially created, stabilized by engineering works, and have had their tidal prisms significantly affected by engineering of the bays and by other inlets connected to those bays.

Johns Pass illustrates two important lessons regarding scour and coastal bridges. First, because of its relative size, the presence of a Bascule pier will have a larger than normal effects on the resulting scour prediction. This usually requires application of HEC-18's wide or complex pier scour approaches (the Florida Department of Transportation (FDOT) has their own complex pier scour approach, see section D.2.3). Secondly, the multiple inlets into the bay illustrate an important concern about attempts to numerically model such bridges and locations. Each inlet could require a separate boundary conditions to ensure overall hydrodynamic circulation.

Additionally, the direction of the surge event could complicate the hydrodynamic, and thus adequacy of the modeling results.

Figure 9.9. Johns Pass, Florida (2002).

9.2.5.3 Jensen Beach Causeway

Bridge scour can occur at bridge locations other than across inlets. In 2005, the Jensen Beach, Florida causeway (Figure 9.10) experienced wave scour episodes. The passage of two successive tropical events[6] along similar storm tracks produced waves within the embayment. These waves struck the causeway abutments and bridge piers – producing scour to depths of over 30 feet (Figure 9.11). FDOT engineers had concerns about structural integrity of the foundations should a third storm event occur (and before installation of scour countermeasures).

In trying to determine what had occurred, FDOT expressed concerns that standard HEC-18 approaches did not predict such scour depths (even when using advanced two-dimensional hydrodynamic and wave modeling). Only when investigators also considered (and modeled) sediment transport did the simulations agree with post-event measurements.

[6] Hurricane Frances (9/5/2005) and Hurricane Jeanne (9/25/2005)

Figure 9.10. Jensen Beach Causeway bridge.

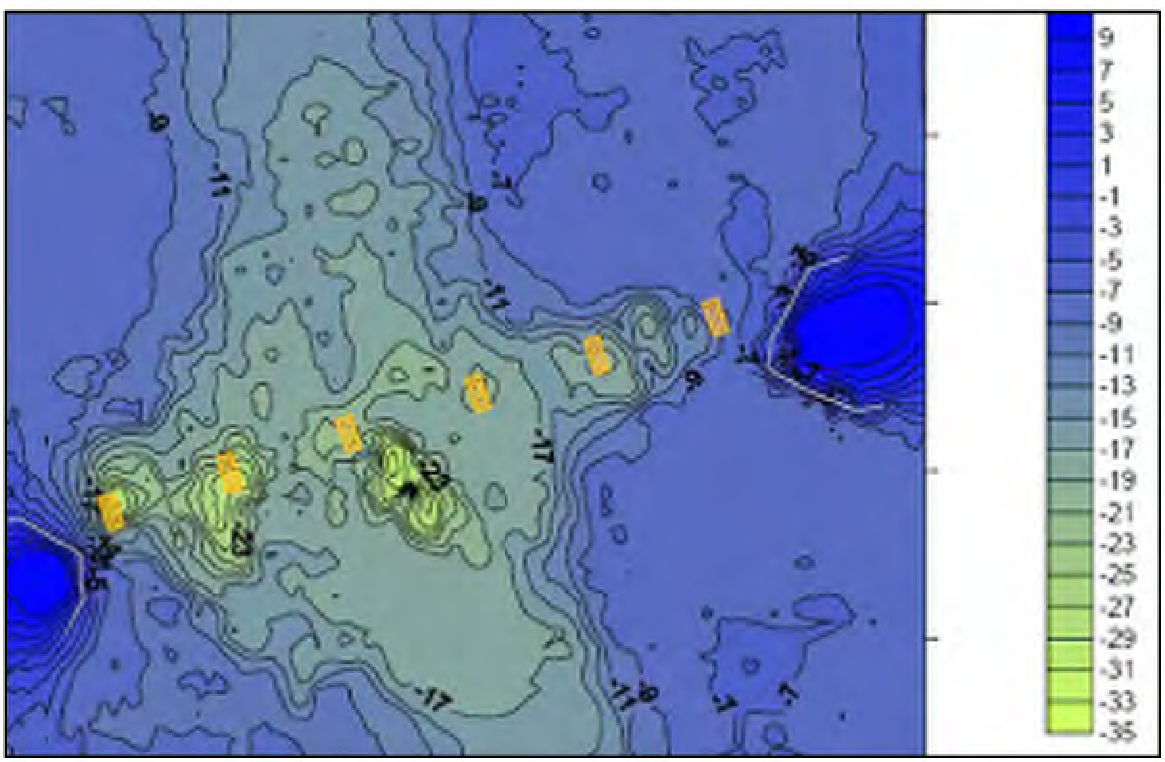

Figure 9.11. Jensen Beach Causeway bridge post event scour bathymetry (2005).

9.3 Coastal Bridge Wave Forces

Highway bridges along the north-central United States Gulf coast were damaged during landfall of Hurricanes Ivan (2004) and Katrina (2005). These include the I-10 bridge across Escambia Bay in Florida, the I-10 bridge across Lake Ponchartrain in Louisiana, the US 90 bridges across Biloxi Bay and Bay St. Louis in Mississippi, and an on-ramp to the I-10 bridge across Mobile Bay in Alabama (see location map, Figure 9.12).

Figure 9.12. Location map of some of the highway bridges damaged by hurricanes in the last 40 years along the north-central Gulf coast.

Other bridges in the region were damaged during Katrina by collisions by vessels that had broken their moorings. A comprehensive listing of bridges damaged by Hurricane Katrina can be found in the ASCE Technical Council on Lifeline Earthquake Engineering (TCLEE) report (2006).

9.3.1 Some Specific Damaged Bridges

Reviewing information related to several of these damaged bridges reveals potential failure modes and commonalities. Specifically, this document will describe the I-10 Escambia Bridge in Florida and the US-90 Biloxi Bay Bridge in Alabama. The investigations and lessons taken from these two bridges could similarly describe many of other wave load impacted bridges.

9.3.1.1 I-10 Escambia Bridge (Hurricane Ivan)

Figure 9.13 shows a photograph of damage to the I-10 bridge across Escambia Bay, Florida, as a result of Hurricane Ivan. At the time of this photo, the storm surge elevation had already dropped a few feet below its maximum.

Figure 9.13. Interstate-10 bridge across Escambia Bay, Florida, after Hurricane Ivan. Photo looking east from Pensacola at dawn September 16, 2004. (Pensacola News Journal photo)

Note that the spans in the right/center of the photograph have been moved to the left (in the direction of wave propagation) and some have fallen off the pile caps.

The spans in the foreground, which are at the same elevation as the ones in the center, have not moved. Potentially, wave heights here were slightly lower due to the partial sheltering of shore and slightly shallower water near the shore.

The spans in the background have not moved because they are elevated above the waves. The spans on the westbound bridge (left side of photo) are less damaged than the ones on the eastbound bridge because the eastbound bridge provided shelter during the peak of the storm and reduced wave heights at the westbound bridge. Indications are that the wave-induced loads were just large enough to begin to move the decks at the peak of the storm surge. Some were moved far enough to topple off the pile caps; others were just displaced a short distance by the waves.

9.3.1.2 US-90 Biloxi Bay (Hurricane Katrina)

Figure 9.14 (and Figure 2.5) show the US Highway 90 bridge across Biloxi Bay, Mississippi after Hurricane Katrina. The extreme storm surge during the hurricane raised the water level to an elevation where waves could impact and inundate the bridge superstructure. The simply supported-span bridge decks were moved off the pile caps to landward (sea is to the left in Figure 9.14). However, no pile cap movement occurred at higher deck elevations (i.e., the approach to a ship channel - shown between the deckless pile caps and an open drawbridge across that channel).

Figure 9.14. US 90 bridge over Biloxi Bay, Mississippi showing the spans at higher elevations were not removed (photo looking southwest from Ocean Springs 2/19/06.)

9.3.2 Wave Loads – A Potential Bridge Failure Mechanism

As part of a synthesis of the existing body of knowledge related to wave forces on highway bridge decks Douglass, et al. (2006) concluded that wave loads (see Figure 9.15) were the primary force causing much of the damage to coastal bridges in the north, central Gulf coast due to Hurricanes Ivan (2004) and Katrina (2005). The likely damage mechanism was waves that struck the simple-span bridge decks because the storm surge raised the water level.

The likely failure mechanism was individual waves producing both an uplift force and a horizontal force on the simple-span bridge deck. The magnitude of the maximum resultant wave force is able to overcome the weight of the decks and the small, lateral resistance provided by the connections (Douglass, et al. 2006).

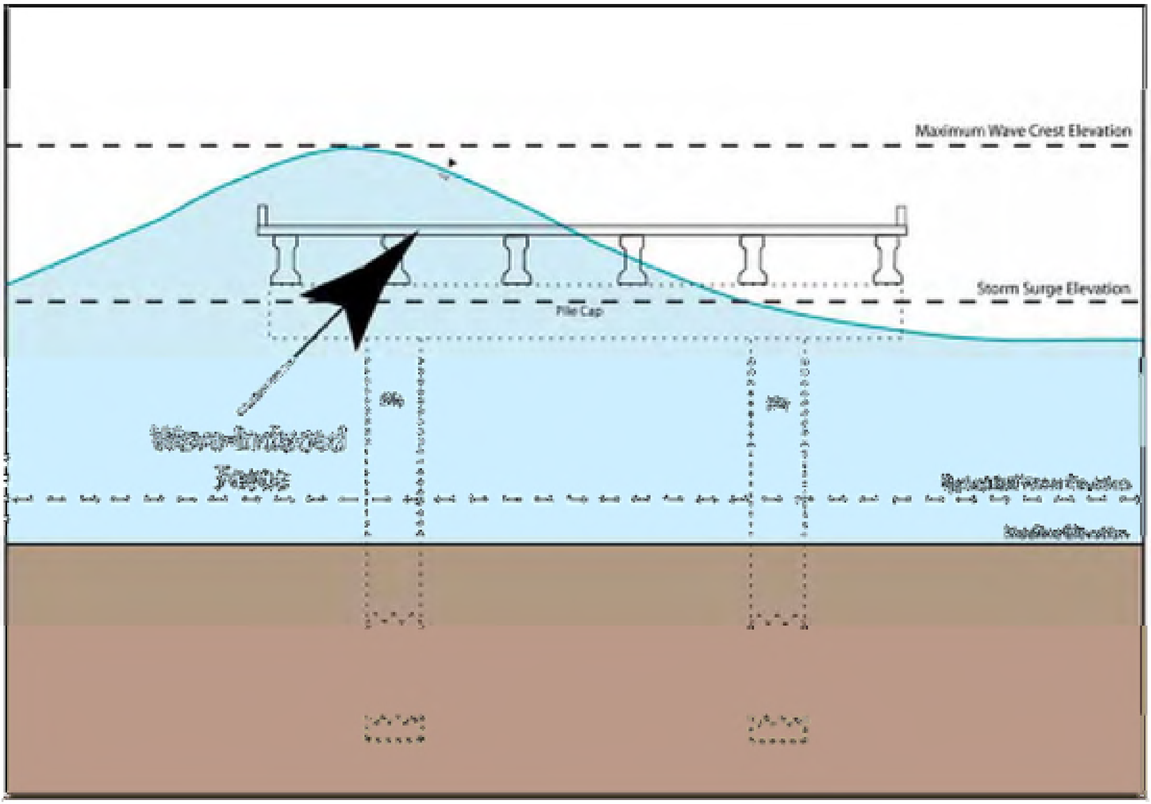

Figure 9.15. Schematic of wave-induced uplift and lateral loads on a bridge deck.

9.3.3 Available Literature on Wave Forces and Loads

The available engineering literature provides little information and limited guidance on wave forces on highway bridge decks. There is, however, a substantial body of literature on wave forces on other types of rigid structures including vertical walls, cylindrical pilings, pipelines, etc. in the ocean and coastal engineering fields. Of particular relevance are investigations of wave loads on decks of piers near or at the coast and on decks of offshore oil and gas exploration and production platforms. Some of the methods from the coastal and ocean engineering literature can be adapted to provide preliminary estimates of wave loads on highway bridge decks for the case of deck elevations at or above the storm surge elevation.

A number of investigators, in small-scale laboratory tests, have measured wave uplift loads on horizontal decks subjected to waves (e.g. El Ghamry 1963; Wang 1970; French 1970; Isaacson and Bhat 1995). Those investigators considered primarily monochromatic waves. McConnell, et al. (2004) report on more recent tests with irregular waves and present a method for estimating lateral and vertical loads on decks with underlying beams. Kaplan, et al. (1995) and Bea, et al. (1999) present methods developed for estimating lateral loads on offshore oil platforms. All three of these investigators only considered relatively high decks with significant clearance above the still-water-level which is typical of the offshore industry. The only testing of highway bridge cross-sections in the existing literature has been by Denson (1978, 1980), Cruz-Castro, et.al. (2006), and Douglass, et al (2007).

Of these existing methods in the literature, McConnell, et al. (2004) may be the most readily adaptable to the highway bridge deck problem. It is an empirical approach calibrated with laboratory results; it is based on relatively simple concepts; it is similar to and more comprehensive than Wang (1970), French (1970), or Overbeek and Klabbers (2001). The laboratory experiments were conducted with irregular waves using modern wave-generation capabilities. The weaknesses of McConnell's method for the highway bridge application were that it was not based on a highway deck geometry, it has not been repeated by other investigators or at other scales, it is perhaps overly complex in its separate treatment of internal and external beams and decks, and it was not developed for decks at or below the still-water elevation.

The two existing approaches developed for the offshore oil industry, Bea, et al. (1999) and Kaplan, et al. (1995), can be used to estimate loads on bridge decks with significant extensions and adaptations. The strengths of these two approaches include their theoretical, physics-based background with Morison's equation (discussed later in this Chapter) and their implicit inclusion of the body of knowledge developed over the past five decades of offshore rig design. Their weaknesses include the complexity of application, the substantial difference in cross-section geometry (including the fact that most offshore platforms have open-grid decks to reduce vertical loads), and that they were specifically developed and tested for structures with very high clearance between the still-water elevation and the bottom of the deck. There is another potential theoretical weakness in that the Morison's equation assumes that the structures are "thin" as compared to the wavelength which is much more questionable for coastal bridges than it is for offshore platform decks. Morison's equation assumes that the structure does not significantly affect the fluid velocities in the wave.

None of the above mentioned methods adequately estimate loads for the case where the bridge deck is completely submerged below the still-water level. The investigators did not test or consider this condition.

9.3.4 Wave Load Constituents

Figure 9.16 shows a schematic of an assumed, typical time-history of one component (either vertical or horizontal) of wave-induced loads on a rigid structure like a bridge deck. Such loading is consistent with measured laboratory loads reported in the literature by numerous investigators.

One part of the wave-induced force is a longer-duration slowly "varying" force. This "varying" force changes magnitude and direction with the phase (crest or trough) of the wave as the wave passes under or across the structure. This part of the wave-induced load has been called "quasi-static," or simply "wave" force by others in the coastal engineering literature. The duration of the "varying" load corresponds with the period of the incident waves that is typically on the order of 3 to 15 seconds. The horizontal slowly varying loads are in the landward direction (based on direction of wave propagation) for the wave crest but can reverse to the seaward direction in the wave trough. Likewise, the vertical slowly varying loads are directed up (i.e. lift) for part of the wave but can be downward for part of the wave. The downward-directed wave load can be due to both the mass and downward momentum of the portion of the wave crest above the bridge deck. The uplift loads appear to be typically greater than the downward-directed loads.

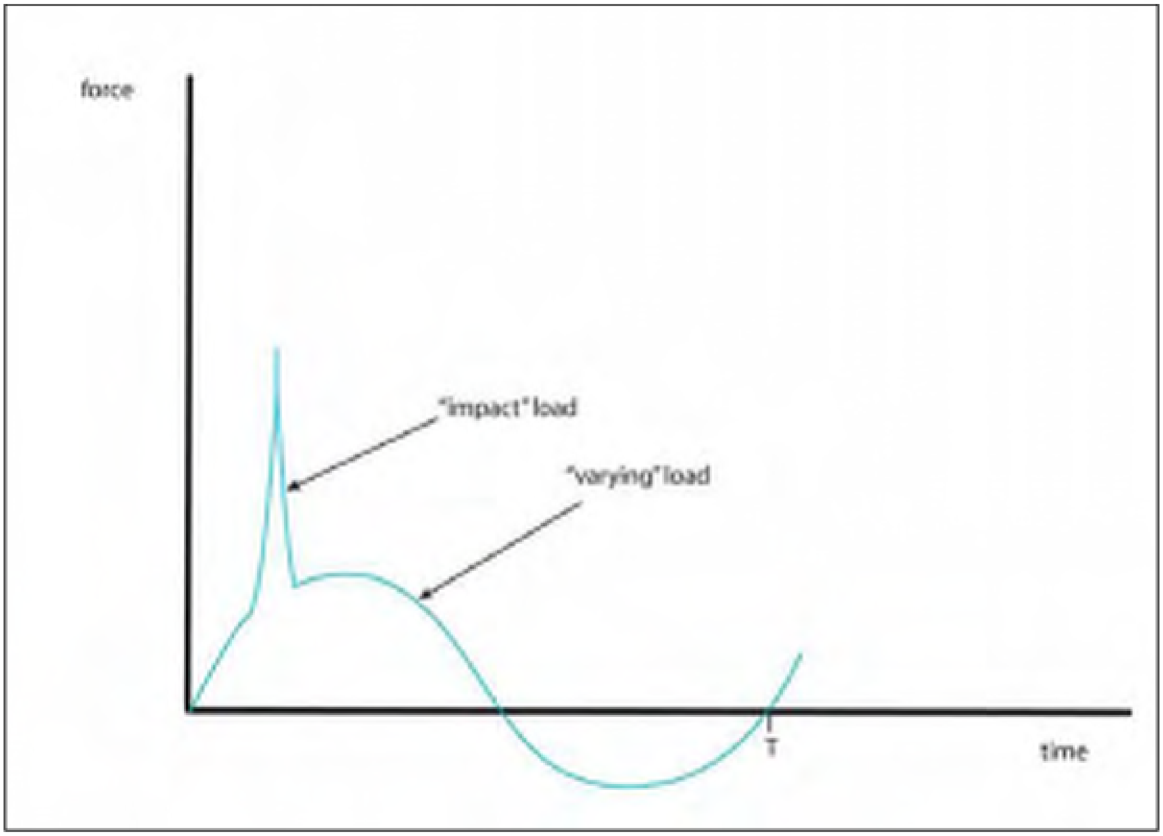

Figure 9.16. Schematic of typical time-history of wave loads on rigid structures.

The other part of the wave-induced load shown in Figure 9.16 is a very short-duration (maybe less than 0.1 to 0.001 seconds long) "impact" force as the wave crest first begins to hit the deck. This force is directed in the horizontal direction of wave propagation and in the upward vertical direction. This impact force does not typically reverse direction. The impact force is often associated with the trapping of a small pocket of air between the structure and the wave face, and is sometimes referred to as the 'slamming' force. Wave impact loads have been studied most for horizontal, wave-induced loads on vertical walls.

9.3.5 Methods for Estimating Wave Loads on Bridge Decks

Several of the methods in the literature discussed above have been used to develop estimates of wave-induced loads on bridges. These include applications of McConnell, et al. (2004), modifications of Kaplan (1995), and a method suggested in Douglass et al. (2006). The Douglass et al. (2006) method is summarized in Appendix E of this document.

At the time of the preparation of this document, a joint AASHTO/FHWA task committee was developing guidance for the design of retrofit solutions for bridges exposed to wave loads (Shelden 2007).

9.3.6 Wave Load Mitigation: Designs and Countermeasures

Concerns related to this phenomenon include the vulnerability of existing bridges, an interest in appropriate design of retrofits to existing bridges to avoid similar failures, and for the design of new bridges that span coastal waters.

9.3.6.1 Bridge Deck Elevation

The most common design approach is to avoid superstructure wave forces by elevating the bridge so that the storm waves crests pass under the low-chord of the bridge. This elevation is shown schematically in Figure 9.17.

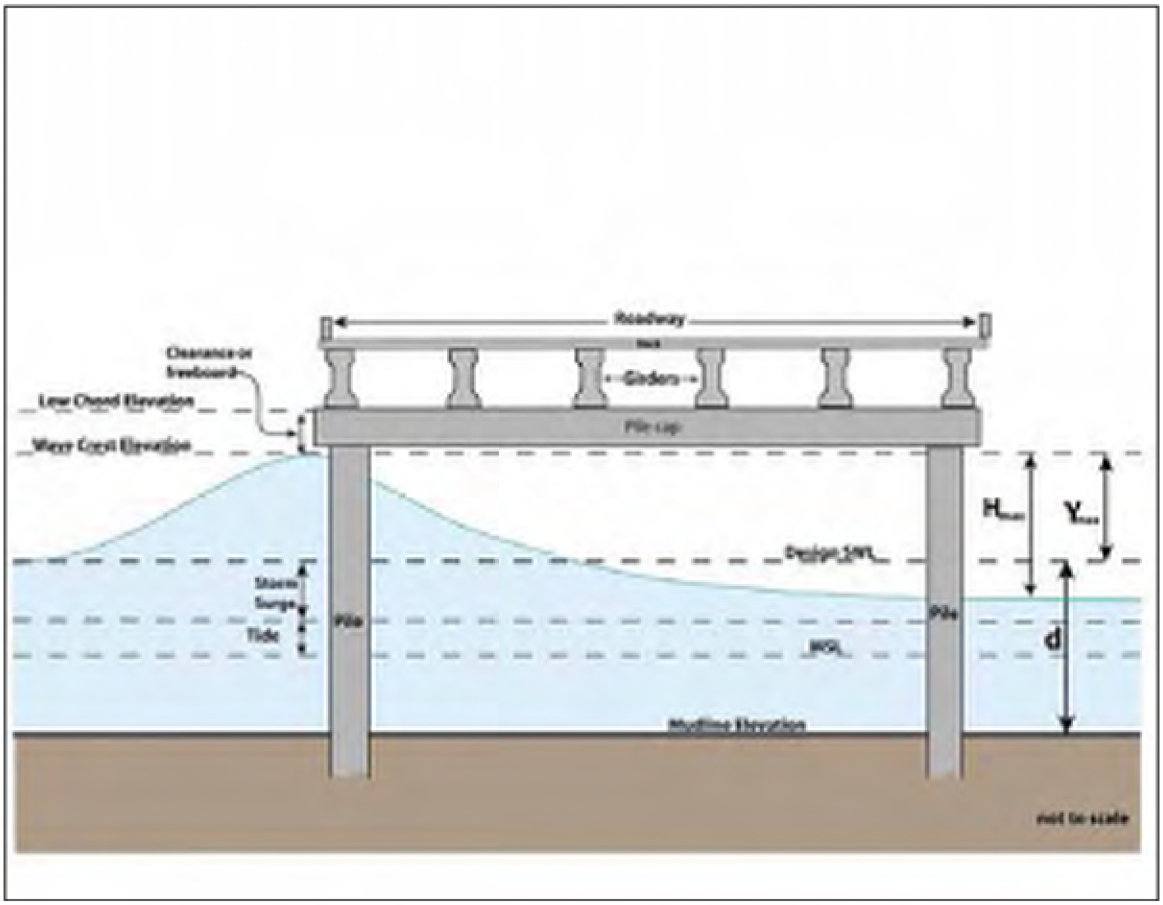

Figure 9.17. Definition sketch of wave parameters and water levels for determining elevation of bridge deck for clearance from wave crests

The elevation can be set by adding some additional clearance or freeboard above the crest of the largest wave in the design sea state:

$$(\text{low chord elevation}) = (\text{wave crest elevation})_{max} + \text{freeboard} \tag{9.1}$$

The low chord elevation is taken as the elevation of the bottom of the girders (see Figure 9.17). The maximum wave crest elevation can be calculated as:

$$(\text{wave crest elevation})_{max} = (\text{design storm surge SWL}) + Y_{max} \tag{9.2}$$

where:

SWL	=	design still water level
Y_{max}	=	difference between the SWL elevation and wave crest elevation for the maximum wave in the design sea-state (defined below)

In general, the value for Y is the portion of the wave height, H, above the SWL. A useful engineering estimate of Y for this purpose is 75% of H. Thus Y_{max} above can be estimated as:

$$Y_{max} = 0.75\, H_{max} \tag{9.3}$$

where:

H_{max} = design maximum wave height (defined below)

9.3.6.1.1 Nominal Maximum Wave Height Approach

The design maximum wave height (H_{max}) depends on the site-specific conditions. The design sea-state can be estimated using a wave generation model applied to that site for specific wind and water level conditions. Given a design significant wave height (H_s), the design maximum wave height can reasonably be set as:

$$H_{max} = 1.7\, H_s \tag{9.4}$$

The value of 1.7 given in Equation 9.4 corresponds with a wave height statistic on the Rayleigh Distribution (see Table 4.1) that is slightly higher than the average of the highest 1% of wave heights ($\overline{H}_1$). This 1.7 value corresponds with the probable maximum wave height for 200 waves. This is a reasonable number of waves for the typical durations of the peak of a storm surge and average wave periods in storm surges[7]. For example this would be roughly 24 minutes with average wave periods of T = 7 s.

Combining Equations 9.3 and 9.4 yields:

$$Y_{max} = 1.3\, H_s \tag{9.5}$$

9.3.6.1.2 Depth Limited Maximum Wave Height Approach

In some cases however, the maximum wave height might be depth-limited, i.e., very large waves in very shallow water. Larger waves in the design sea state may break farther offshore of the bridge and the largest waves will not reach the bridge. In this case, check the depth-induced breaking criterion (or similar criteria):

$$\left(\frac{H}{d}\right)_{max} \approx 0.8 \tag{9.6}$$

This can be written as:

$$H_{max} = 0.8\, d_s \tag{9.7}$$

where:

d_s = depth at bridge structure during design conditions (i.e. including the storm surge)

For the depth limited case, combining Equations 9.3 and 9.7 yields:

[7] The Longuet-Higgins (1952) equation (as presented in the Coastal Engineering Manual, USACE 2002) provides a more complex approach than Equation 9.4 for estimating H_{max}.

$$Y_{max} = 0.6\ d_s \tag{9.8}$$

9.3.6.1.3 Estimating the Maximum Wave Crest Elevation

The difference between the SWL elevation and wave crest elevation for the maximum wave in the design sea-state (Y_{max}) used in Equation 9.2 should be the lesser of the values yielded from Equation 9.5 and Equation 9.8. Therefore, considering the potential for non-depth-limited and depth-limited maximum wave heights, the primary equation estimating the elevation of the maximum wave crest (see Figure 9.17) becomes:

$$(\text{wave crest elevation})_{max} = (\text{design storm surge SWL}) + (1.3\ H_s \text{ or } 0.6\ d_s)_{min} \tag{9.9}$$

This equation can be used to set the elevation of the low-chord of bridge decks that span coastal waters. The next section discusses the use of additional freeboard above this elevation and the determination of the input surge and wave height to Equation 9.9.

9.3.6.2 Freeboard Considerations

"Freeboard" can be added to the maximum wave crest elevation found from Equation 9.9. The approach outlined above does not provide "freeboard" above the wave crests. In riverine systems, State DOTs may require one or two feet of "freeboard" to be added above the design water surface elevation to account for wave action or debris as well as for uncertainty in the analysis. This freeboard, if added in the coastal situation, will also account for higher waves in the sea-state. The uncertainties involved in coastal surge SWL analysis are likely at least as great as those in the riverine situation (if not significantly greater). Thus, some additional freeboard for the low-chord elevation of coastal bridges may be appropriate.

However, complete clearance from all wave forces may not be needed to ensure bridge integrity during major coastal storms. Post-storm inspections of damage to bridge decks along the north-central Gulf coast in 2004 and 2005 indicate that some bridge decks survived that were exposed to some wave loads. Apparently, the loads were small enough that they did not cause damage. The damage pattern suggests that there was a critical elevation at each location for that specific bridge deck design and those site-specific and storm-specific surge and wave conditions. Spans below that critical elevation were displaced off the pile caps; spans above that elevation were not. The critical elevation was below the elevation for complete wave clearance given by Equation 9.9. This is likely due to resistance to wave forces provided by the weight of the bridge spans and the limited connections.

For example, Figure 9.14 shows some simply-supported spans on the US 90 bridge across Biloxi Bay, Mississippi still in-place even after removal of other spans. These remaining spans had a higher low chord elevation than those displaced. The critical elevation for the bridge damage was a low-chord elevation of roughly 23 feet (Douglass et.al. 2006) (all bridge span elevations in this discussion are average elevations of the bottom of the outer girder relative to NGVD). There may have been damage at higher elevations that was not visible from shore.

On the east side of the drawbridge shown in Figure 9.14, the span at elevation 24.5 feet (low-chord) stayed in place and the next lower span (elevation = 22.9 feet) moved. The estimated maximum storm surge SWL elevation at this location during Hurricane Katrina was 21.5 feet (NGVD) with an estimated significant wave height of H_s = 9.8 feet (see Figure 3.5). The Equation 9.9 procedure would estimate that the crest of the maximum wave was at + 34.2 feet.

Applying this example of Katrina damage to the Biloxi bridge: the maximum wave crest elevation was + 34.2 feet, yet a bridge span as low as + 24.5 feet "survived." Thus, the bridge span with a low-chord elevation almost 10 feet lower than the maximum wave crest elevation apparently did not move. One conjecture about this observation is that the wave loads were insufficient to overcome the weight of the decks and the connection resistance.

Some researchers have suggested that simply-supported bridge decks with low chord elevations above the elevation of the crest of the significant wave survived wave attack in the hurricanes of 2004 and 2005 (Chen 2005). This would suggest the Y_{max} could be set to $0.75H_s$ and not require any additional freeboard. A preferable approach is to set the deck elevations based on an improved understanding of the wave loads. The discussion above also assumes that the pile cap design can withstand wave loads.

9.4 Other Coastal Bridge Issues

This section very briefly discusses other design and maintenance issues related to coastal bridges including increased concrete spalling due to wave splash, and lateral loads on pilings.

Some low-elevation coastal bridges have suffered increased concrete damage near their landward end just above vertical retaining walls. Wave splash during storms sprays salt water on the underside of the bridge deck concrete and, over time, these areas can become areas of concern for bridge inspectors. The use of reinforced concrete in the marine environment typically requires additional engineering considerations, including the use of air entrainment admixtures and increased minimum thickness of specified concrete cover over reinforcing bars. Newer bridges, with higher clearance requirements and longer, higher approach sections, often avoid this problem by elevating all of the bridge deck well above the elevation of splash. Wave runup and splash on existing low bridges could be reduced by placing rip-rap on the vertical walls. Clearance issues for coastal bridges over navigation channels are primarily controlled by the US Coast Guard.

Lateral loads on bridge pilings and pile groups in a coastal situation can be increased due to waves. These loads in riverine situations are well modeled by the traditional fluid mechanics approach of estimating drag as a function of the water velocity squared and an empirical drag coefficient (e.g. Standard Specifications for Highway Bridges, AASHTO 2002). However, the nature of wave motion produces loads beyond those due just to drag. The oscillatory water particle motion below waves can impart significant forces on structures due to the fluid accelerations as well as the velocities. Thus, it is neither adequate nor appropriate to just increase the velocity used in the drag equations to account for the maximum wave orbital velocity. The acceleration generated forces, also called inertia forces, should be considered.

Morison's equation from ocean engineering estimates the horizontal force per unit length of a vertical pile in waves as:

$$f_p = f_i + f_D = C_M \rho \frac{\pi D^2}{4} a_x + C_D \rho D u|u| \qquad (9.10)$$

where:

f_p	=	horizontal force per unit length of a vertical pile
f_i	=	inertial force per unit length of pile
f_D	=	drag force per unit length of pile
D	=	diameter of pile
ρ	=	density of water (1025 k/m³ for seawater)
u	=	horizontal water particle velocity at the axis of the pile (as if the pile were not there)
a_x	=	horizontal water particle acceleration at the axis of the pile (as if the pile were not there)

C_D = drag coefficient
C_M = inertia or mass coefficient

The first term in Morison's equation accounts for the dynamic force on the structure due to the acceleration in the waves. It is called the inertia term. The second term is the drag term and it is analogous to the drag load on a piling in unidirectional flow. The absolute value is used in the drag term because the load reverses direction with wave phase. In a wave, the water particle velocity, direction and acceleration at different points are constantly changing with phase. They also vary with depth below the surface and the total force on the pile is the depth-integrated sum of these changing loads. The two terms are out of phase and thus not maximum at the same time.

More information, including values for the coefficients and appropriate applications, on Morison's equation can be found in other references (Sarpkaya and Isaacson 1981; USACE 1984).

An inherent assumption in Morison's equation is the "thin piling" assumption that velocity and acceleration do not vary over the structure in the direction of wave propagation and that the piling is thin enough to not cause much of an effect on the wave. Because of the complexities involved in applications of Morison's equation, a coastal or ocean engineer should be included in the design or analysis team for estimating wave loads on pilings. Empirical consideration of these forces is described in Wiegel R. L. (1964) and NAVFAC DM 26.2 (1982). In cases of shallow water and/or wave breaking, where water particle velocities and accelerations will be significantly under-predicted by simple linear wave theory, higher-order theories, discussed in Chapter 4, are required. Dean's stream-function approach is a non-linear wave theory that was developed to predict wave kinematics and forces on structures in deep and shallow water settings (Dean 1965).

9.5 Selection of Design Storm Surge & Design Wave Heights

9.5.1 Design Storm Surge SWL

The selection of the design storm surge SWL (still-water-level) can be based on an analysis of historic storm surge elevations at the specific site or on an analysis that incorporates site-specific modeling of historical (hindcast) storm surges (see section 3.2 and specifically section 3.2.2 for additional details).

As described in section 3.3.2, FEMA FISs and FIRMs provide SWL for many coastal areas. These may be suitable sources for these data, as long as study and methodological caveats are well understood.

A nearby tide gage may provide a reasonable first approximation of surge at a site. In particular when a bridge location along a coast is between two tide gages, a reasonable estimate of the storm surge at the site might be generated by comparing the long-term statistics from the two gage locations. However, care should be taken that typical storm surges are not significantly different from those at the nearest tide gage. This could be the case for bridge crossings in areas that can magnify the storm surge due to local bathymetry and geography. Storm surge elevations can vary significantly from location to location.

Site-specific modeling of historical (i.e. hindcast) storm surges is appropriate for the design of new bridges and decisions concerning modifications to existing bridges. The potential damage justifies a comprehensive hydrodynamic surge analysis. Developing a probabilistic basis for this design storm surge elevation is consistent with both the process for riverine bridge design considerations as well as risk-based flood maps for coastal management done by FEMA and other agencies. Both approaches, historical gage analysis and historical storm modeling analysis, can be used. The historical gage analysis can be used as a check on the reasonableness of the results of the modeling approach.

9.5.2 Design Wave Heights

The design wave height (H_s) used in Equation 9.9 is the significant wave height at the bridge location during design conditions. This can be determined by using the appropriate techniques outlined in Chapter 4. For fetch-limited situations, the parametric wind-wave generation modeling method (Appendix C) may be adequate. For some situations in shallow water without much storm-surge, depth-limited wave conditions may apply. Many situations, including those exposed to open ocean storm waves, may require probabilistic oceanic wave modeling.

As a check, some FEMA FISs contain wave height estimates. However these may not report H_s, but some other wave height statistic. Apply such estimates with knowledge of these and other study caveats.

9.5.3 Coastal Engineer Involvement

Given the importance and complexity of these considerations to the integrity of the highway structure, the involvement of a qualified coastal engineer in the project's design or pre-construction review is highly recommended.

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Chapter 10 - References

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