D. EXTRACT FROM DRAFT SCOUR STUDY

Following is an extract from "Protection Alternatives for Levees and Floodwalls in Southeast Louisiana: Phase One Evaluation," a report prepared by ERDC Coastal and Hydraulics Laboratory. The document is still a draft and is marked "Intended for internal Corps use only."

The extract included here is Chapter 4, "Protection for Overtopped Floodwalls." This information is provided to designers to illustrate some of the design issues to be addressed when designing scour protection. This extract is provided for information only.

4 Protection for Overtopped Floodwalls

Failure Modes of Concrete and Sheetpile Floodwalls

Floodwall failures can be broadly grouped into two categories: (a) structural failure of the vertical wall due to applied hydrodynamic pressure forces; and (b) foundation failure due to seepage and liquefaction, slip surface or shear plane failures, and loss of lateral support due to erosion. This chapter focuses only on protection from loss of foundation support due to the erosive impact of falling water that has overtopped the floodwall.

Floodwalls that might be overtopped by rising water should be designed with erosion protection on the protected (dry) side capable of resisting the force of the free-falling water jet. Figure 4.1 illustrates flow overtopping a floodwall and plunging (in this case) into standing water on the protected side of the floodwall. The plunging jet penetrates the water and creates large eddies that erode material from the unprotected soil surface. The same mechanism will scour bed material when there is not standing water on the protected side of the floodwall.

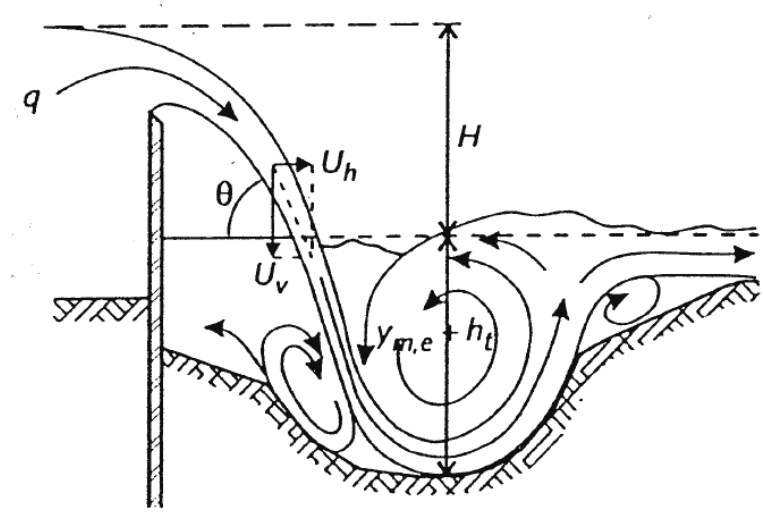

Figure 4.1. Scour hole formation by overtopping jet (from Hoffmans and Verheij 1997)

Eroded material is thrown into suspension and carried away by the turbulent flow. This scouring action removes material that may be providing critical lateral support pressure against the protected side of the vertical floodwall. Failure occurs if the remaining, undamaged portion of the foundation adjacent to the wall cannot withstand either the shear force or the overturning moment exerted on the floodwall by the elevated water on the flood side of the wall.

Total collapse of a section of the floodwall allows a large volume of water to flow into the protected region through the resulting breach, and this may cause adjacent wall sections to fail and enlarge the breach. Localized partial failure includes tilting of the floodwall so gaps open up between the dislodged section and adjacent undamaged floodwall. Provided the wall does not tilt farther, it still affords some degree of flood protection. However, the wall top elevation is deceased slightly by tilting, and the overflowing water jet will be directed on foundation soil farther away from the wall that could increase the scour hole width.

Figure 4.2 shows scour on the protected side of an I-wall adjacent to the Lakefront Airport. A deep trench was scoured by the overflowing jet, but in this case the floodwall does not appear to be affected by the loss of lateral support at the base.

Figure 4.2. Scour trench formed by overtopping flow at I-wall adjacent to the Lakefront Airport (photograph by Peter Nicholson from Seed, et al. (2005)).

Figure 4.3 shows the I-wall along the east side of the IHNC at approximate B/L Sta 11+00 (DM3 Chalmette Area Plan), looking toward the Claiborne Avenue bridge. Depth of scour was to the bottom of the I-wall concrete cap (2 ft), and scour trench width was approximately 7 ft. The I-wall top elevation was designed to a height of 15 ft above mean sea level, the bottom of the concrete cap was at elevation 7 ft, and the earthen levee

crown was at elevation 9 ft. Actual wall height was reported to be 12.5 ft when converted to local mean sea level, and the storm surge height was reported to be up to 15 ft. As an approximation of the overtopping water impact, a surge crest 2.5 ft above the floodwall impacted the earthen levee crown from a height of 6 ft. Using procedures developed in the following section, the falling jet of water was estimated from Figure 4.12 to have an impact velocity of about 23 ft/sec, and the impact force was estimated from Figure 4.13 to be about 700 lb/ft. The water impact removed a portion of the earthen levee crown, including all of the structural backfill zone adjacent to the concrete wall.

Figure 4.3. Scour trench on the east side of the IHNC

Soil scour within the structure backfill zone is also evident at other locations such as the T-wall on the north side of Gate 13E on the east side of the IHNC near Lakefront Airport at approximate W/L Sta 61+38 (DM2 Supplement 8 IHNC Remaining Levees). The top of T-wall elevation was 13.25 ft (MSL) and the existing top of ground elevation was 0.1 ft (MSL), from drawing file H-2- 24111, plate IV-20. Figure 4.4 shows a scour trench with depth of 30 in. and trench width of approximately 8 ft. Overtopping water dropped 13 ft before impacting the levee. Figures 4.12 and 4.13 were used to estimate an impact velocity of about 30 ft/sec and an impact force over 700 lb/ft.

Figure 4.4. Scour trench at a T-wall on the east side of the IHNC

Reaches along the MRGO protected by exposed sheetpile floodwalls experienced scouring on the backside, and breaches occurred at several locations. Figure 4.5 shows a section with 4300 ft of sheetpile damage along MRGO between Bayous Bienvenue and Dupre, St. Bernard Parish. The damaged sheetpile section is near utility crossings, with scour on the protected side and levee crown. B/L Sta 590+70 is centerline of the two pipelines.

Larger breaches along sheetpile reaches were evident on the north bank of the GIWW, including the Bulk Loading Facility, the Michoud Canal (Air Products plant), and pump station 15. Figure 4.6 shows the Air Products plant breach near Sta 772+00 B/L (New Orleans East Back Levee). Scour depths were 10 to 12 ft on both the floodside and protected side of the sheetpile wall. Nearest borings on either side of the failure, 5-E and 6-E (from plate 5, DM2 Supp 4, March 1971) shows CH material with sand / silt lenses in the pre-existing (1965) levee at crown elevation ~12 ft, prior to construction of the sheetpile wall. The storm surge in the GIWW was at an approximate elevation of 15 to 17 ft, and Figures 4.12 and 4.13 indicate the estimated overtopping jet impact velocity ranged up to about 23 ft/sec, and the impact force ranged up to about 700 lb/ft. Note that the breach occurred in the sheetpile reach, and not along the adjacent transitions to earthen levee on the east side and connection to the T-wall on the west side.

Figure 4.5. Overtopping scour at sheetpile floodwall along the MRGO

Figure 4.6. Sheetpile floodwall breach on the New Orleans East Back Levee

Several other vertical structures (mostly I-walls) were catastrophically breached along the 17th St. and London Avenue Canals and the IHNC (East and West sides). Investigations are ongoing, but it appears that failure modes other than erosion and scour caused by overtopping water may have played larger roles at those locations.

Design Physical Parameters

Scour protection placed on top of the foundation soil on the protected side of floodwalls must be able to withstand a free-falling jet of water that overtops the wall. This condition could persist for a prolonged period. Protection coverage must extend away from the wall sufficient distance to assure complete protection from both the direct plunging water jet, and also from the resulting ground-parallel supercritical flow and eventual hydraulic jump that forms some distance from the wall. Important design parameters related to the flow hydrodynamics are floodwall height and height of the storm surge level relative to the floodwall top elevation.

Under the assumption that robust structural foundation protection is necessary, geotechnical design parameters are somewhat limited to the requirement that foundation soil must support the overlain protection without significant differential settlement. Also important is the possibility of soil erosion at the boundaries of the overtopping protection. Geotechnical considerations related to proper foundation design to resist the applied lateral loading on the floodwall and to prevent seepage underneath the wall are not included in this chapter.

Surge Overtopping

Storm surge overtopping of a floodwall having constant top elevation along the wall is well approximated by the classic hydraulics problem of flow over a sharp-crested weir. Assuming no viscous energy dissipation occurs over the short crest width of the vertical floodwall shown in Figure 4.7, and there are no lateral contraction effects (i.e., constant wall top elevation), discharge per unit wall length is given by the expression (e.g., Henderson 1966)

$$q = C_d \frac{2}{3} \sqrt{2g}\, h_1^{3/2} \tag{4.1}$$

The discharge coefficient, C_d, is given by the expression

$$C_d = 0.611 \left[\left(1 + \frac{v_1^2}{2gh_1} \right)^{3/2} - \left(\frac{v_1^2}{2gh_1} \right)^{3/2} \right] \tag{4.2}$$

where g is the acceleration of gravity, h_1 is height of the surge above the wall, and v_1 is the upstream velocity as shown on Figure 4.7. The above discharge formulation was referred to as the "Weisbach extention of the Poleni formula" by Rouse (1961) with the addition of C_d in Eqn. 4.1 and the definition of C_d (Eqn. 4.2) being Weisbach's contribution.

Experimental work provided a simple approximation for C_d expressed as

$$C_d = 0.611 + 0.08 \left(\frac{h_1}{h} \right) \tag{4.3}$$

where h is the depth of the water as defined in Figure 4.7. For small values of h_1/h, the discharge coefficient approaches $C_d = 0.611$. Figure 4.8 presents discharge per unit length of floodwall as a function of surge elevation above the wall for values of h = 4, 6, 8, and 10 ft. For these cases the discharge curves do not have much variation until the ratio h_1/h approaches unity.

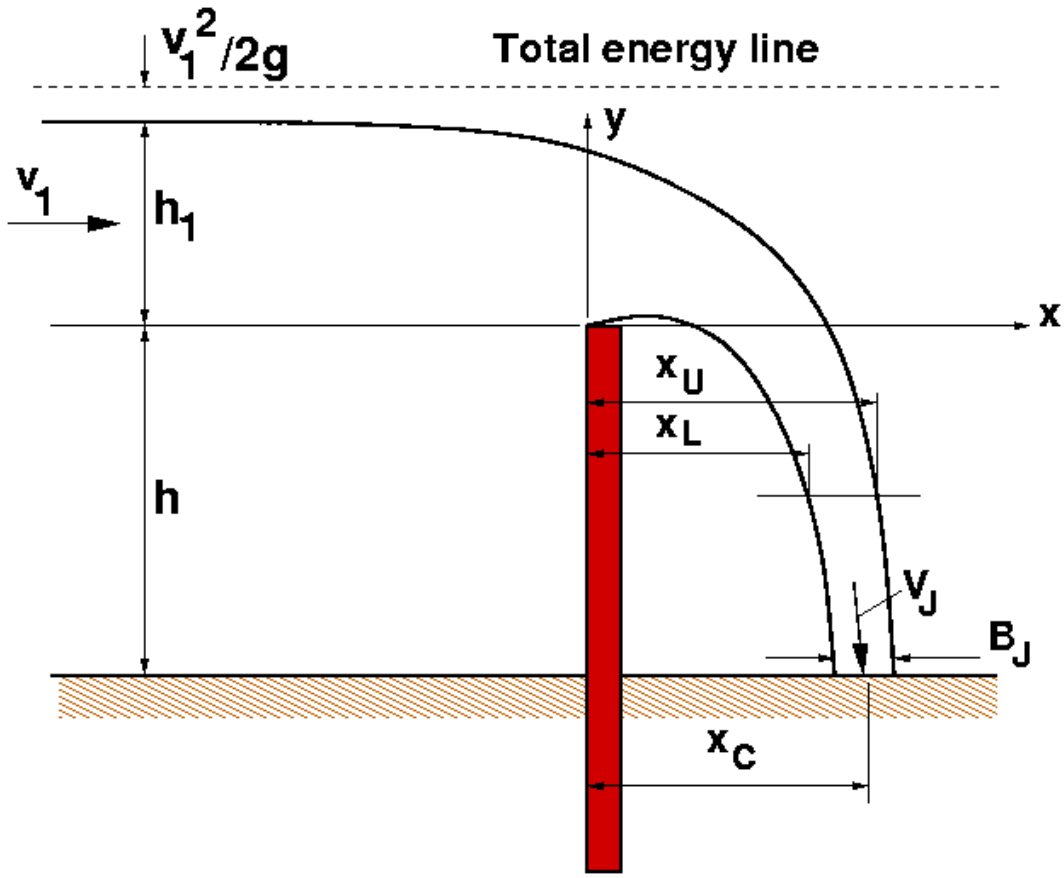

Figure 4.7. Flow over a sharp-crested weir

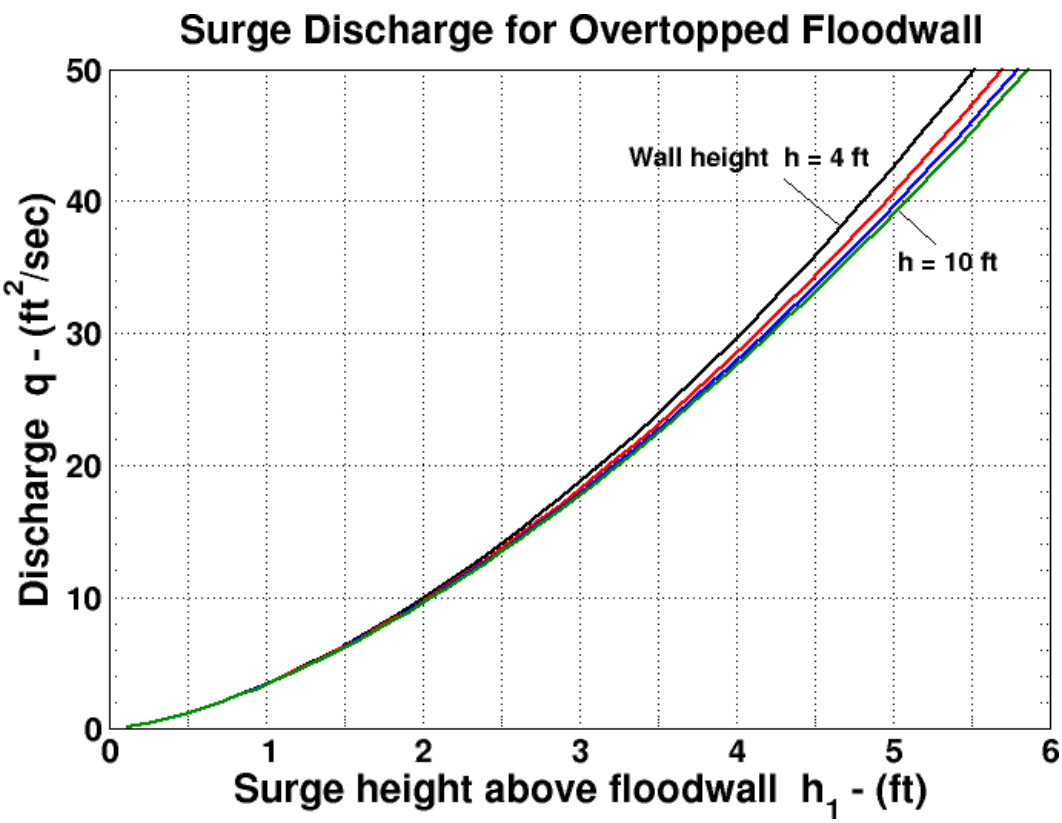

Figure 4.8. Discharge per unit floodwall length for values of *h* = 4, 6, 8, and 10 ft

The jet of water passing over the vertical floodwall has two surface profiles referred to as "nappes" (a French word meaning "a continuous surface"). The lower nappe is closest to the backside of the floodwall, and the upper nappe is the extension of the flow free surface as it spills over the wall. The trajectories of the lower and upper nappes are given in most open channel flow books (e.g., Chow 1959, Morris 1963). In dimensionless form, the equations are as follows with the *x-y* coordinate system as defined in Figure 4.7

$$\frac{y}{H} = A\left(\frac{x}{H}\right)^2 + B\left(\frac{x}{H}\right) + C \qquad \text{(lower nappe)} \qquad (4.4)$$

$$\frac{y}{H} = A\left(\frac{x}{H}\right)^2 + B\left(\frac{x}{H}\right) + C + D \qquad \text{(upper nappe)} \qquad (4.5)$$

where H is the total head above the weir crest, i.e.,

$$H = h_1 + \frac{v_1^2}{2g} \qquad (4.6)$$

and

$$A = -0.425 + 0.25\,G \qquad (4.7)$$

$$B = 0.411 - 1.603\,G - \sqrt{1.568\,G^2 - 0.892\,G + 0.127} \tag{4.8}$$

$$C = 0.150 - 0.45\,G \tag{4.9}$$

$$D = 0.57 - 0.02\left[10\left(G - 0.208\right)\right]^2 \exp\left[10\left(G - 0.208\right)\right] \tag{4.10}$$

with

$$G = \frac{v_1^2}{2gH} \tag{4.11}$$

For high weirs, $v_1 \approx 0$, and $H \approx h_1$, and the nappe equations reduce to the forms

$$\frac{y}{h_1} = A\left(\frac{x}{h_1}\right)^2 + B\left(\frac{x}{h_1}\right) + C \qquad \text{(lower nappe)} \tag{4.12}$$

$$\frac{y}{h_1} = A\left(\frac{x}{h_1}\right)^2 + B\left(\frac{x}{h_1}\right) + C + D \qquad \text{(upper nappe)} \tag{4.13}$$

with

$A = -0.425$
$B = 0.055$
$C = 0.150$
$D = 0.559$

Equations 4.12 and 4.13 are quadratic equations that can be solved to give values of the nappe profile x-values in terms of the vertical distance from the top of the floodwall. There are two solutions that satisfy each quadratic equation. The equations given below are the appropriate solutions yielding positive values of x.

$$\frac{x_L}{h_1} = \frac{-B - \sqrt{B^2 - 4A\left(C - y/h_1\right)}}{2A} \qquad \text{(lower nappe)} \tag{4.14}$$

$$\frac{x_U}{h_1} = \frac{-B - \sqrt{B^2 - 4A\left(C + D - y/h_1\right)}}{2A} \qquad \text{(upper nappe)} \tag{4.15}$$

The intersection points of the lower and upper nappes with the horizontal ground level on the protected side of the floodwall are found by setting $y = -h$ in the above equations. The horizontal width of the overtopping jet at impact is given by

$$B_X = x_U(y = -h) - x_L(y = -h) \tag{4.16}$$

and the distance from the flood side of the wall to the center of the jet at impact is given as

$$x_C = \frac{x_U(y=-h) + x_L(y=-h)}{2} \tag{4.17}$$

Figure 4.9 shows the variation of jet impact location distance, x_C , from the floodwall front face as a function of surge elevation above the wall crest and the vertical plunge distance. Horizontal width of the plunging jet at impact is given as a function of the same parameters in Figure 4.10.

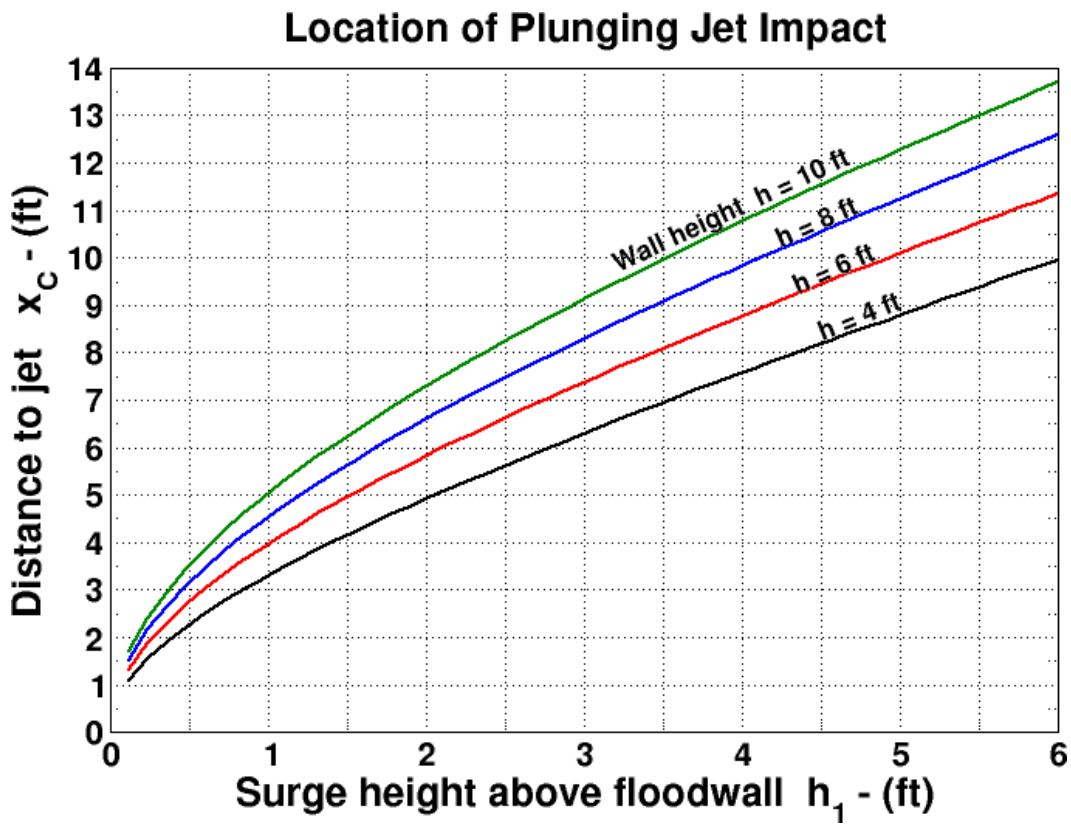

Figure 4.9. Horizontal distance between the floodwall front face and the center of the plunging jet at impact

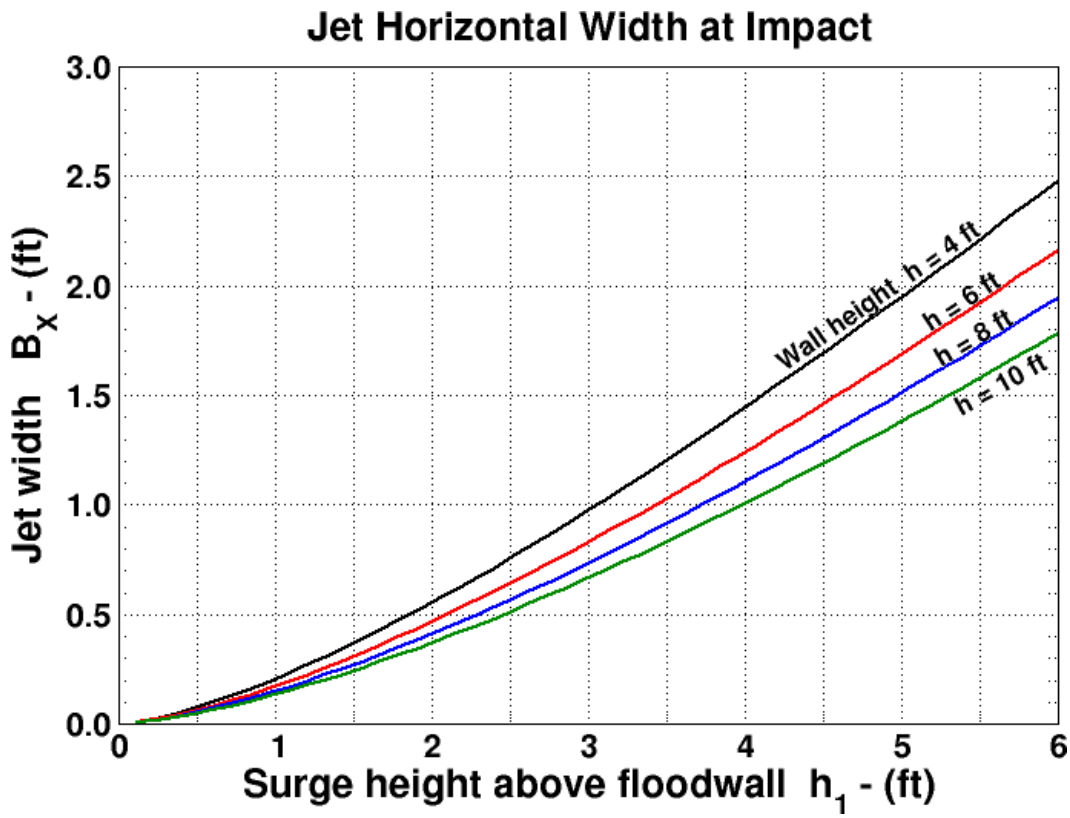

Figure 4.10. Horizontal width of the plunging jet at impact

If there is no venting, the air pressure in the space between the floodwall and lower nappe may become less than atmospheric as air is entrained into the jet during sustained overtopping. The decreased pressure will draw the plunging jet closer to the wall; however, this decrease in plunge point location away from the vertical wall is difficult to predict. This is likely not a problem because the scour protection will probably cover the entire region from the base of the wall out well past the location of jet impact.

The overtopping jet impacts the ground at an angle less than vertical (which is given by –90 deg in the coordinate system defined in Figure 4.7). The jet entry angle is well approximated by the average of the angles of the lower and upper nappe profiles when they intersect the horizontal ground level. The entry angles of the nappe profiles are found by taking the derivative of Eqns. 4.12 and 4.13 and evaluating the result at $x = x_L$ and $x = x_U$, respectively, to get

$$\theta_L = \tan^{-1}\left(\frac{dy}{dx}\right)_L = \tan^{-1}\left(\frac{2\,A\,x_L}{h_1} + B\right) \tag{4.18}$$

$$\theta_U = \tan^{-1}\left(\frac{dy}{dx}\right)_U = \tan^{-1}\left(\frac{2\,A\,x_U}{h_1} + B\right) \tag{4.19}$$

where $A = -0.425$ and $B = 0.055$. The jet entry angle is estimated as

$$\theta_J = \frac{(\theta_L + \theta_U)}{2} \tag{4.20}$$

Overtopping jet entry angles are shown on Figure 4.11 as a function of surge height above the floodwall for a variety of wall heights.

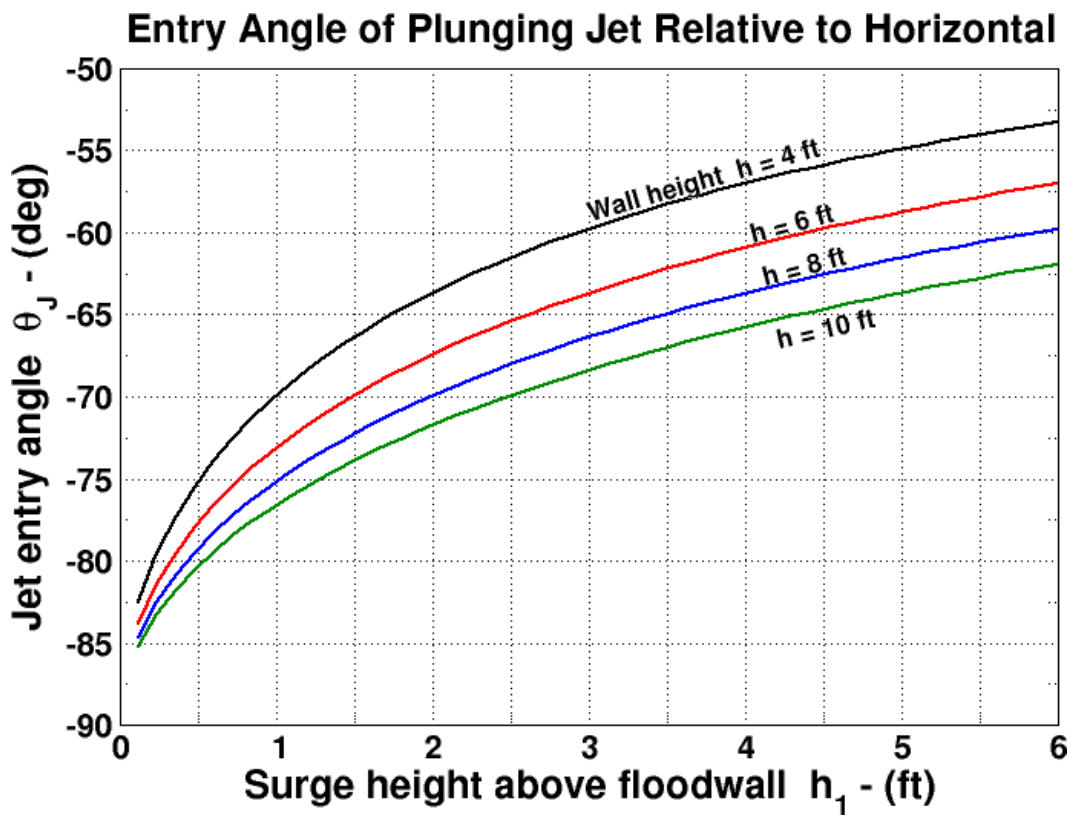

Figure 4.11. Overtopping jet entry angle relative to the horizontal ground level

From geometric considerations the width of the impinging jet normal to the flow streamlines can be estimated with reasonable accuracy by the formula

$$B_J = B_X \sin(-\theta_J) \tag{4.21}$$

Discharge over the floodwall remains constant for steady flow, and the discharge per unit length of the plunging jet at impact with the ground surface is given simply as the jet velocity parallel to the flow streamlines times the width of the jet normal to the flow. Thus, the jet entry velocity can be estimated as

$$V_J = \frac{q}{B_J} \tag{4.22}$$

Figure 4.12 shows jet impact velocities as a function of surge height above the floodwall and vertical distance to the ground level.

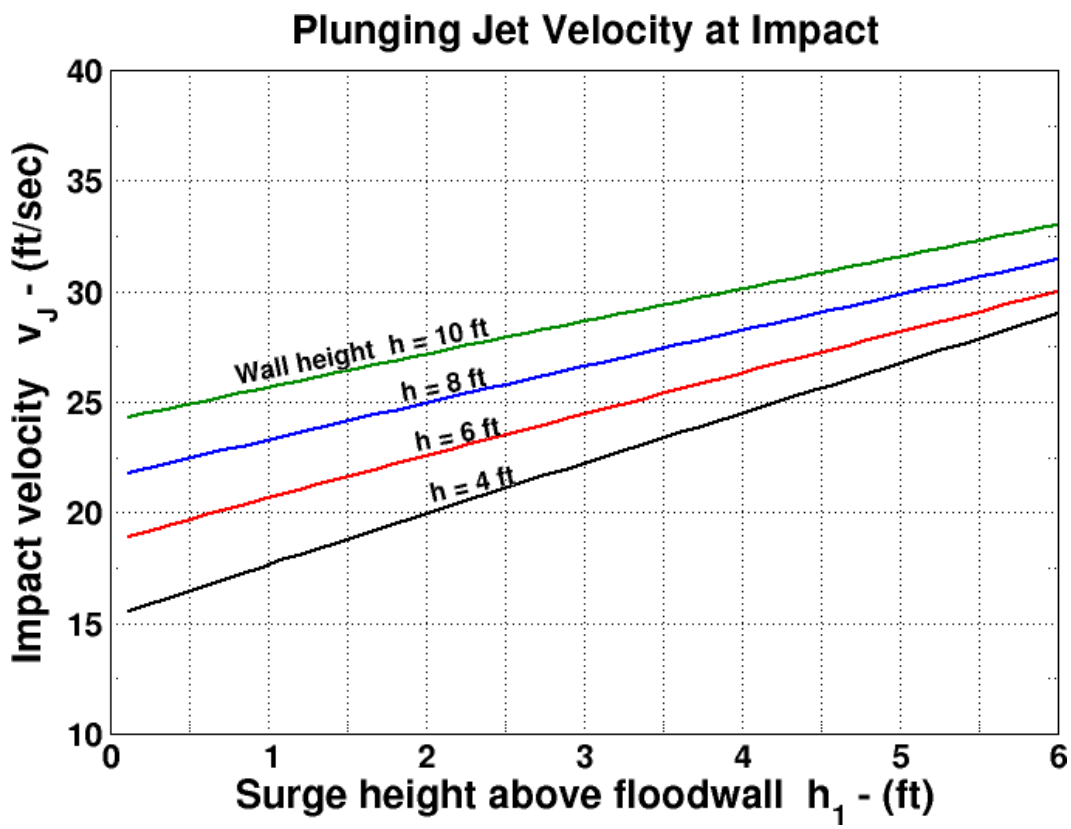

Figure 4.12. Overtopping jet velocity at impact with the ground

Finally, the total force (thrust) exerted by the overtopping jet on the scour protection per unit length along the wall is given in inviscid jet theory (e.g. Milne-Thompson 1960) as

$$F_J = \rho B_J \left(v_J\right)^2 \tag{4.23}$$

where ρ is water density. This equation is an expression of the momentum flux of the jet, and the force is directed parallel to the jet streamlines.

Figure 4.13 presents force magnitude estimates based on Eqn. 4.23. As shown on Figure 4.13, the lines for the different fall distances h are quite close because the range of fall distance is not too large. However, the impact force increases substantially with overtopping elevation h_1, that is directly related to total discharge per unit length of wall. The convergence of the lines at the higher values of h_1 is not physically correct. This convergence is most likely caused by the empirical approximations for discharge coefficient C_d (Eqn. 4.3) and jet width B_J (Eqn. 4.21).

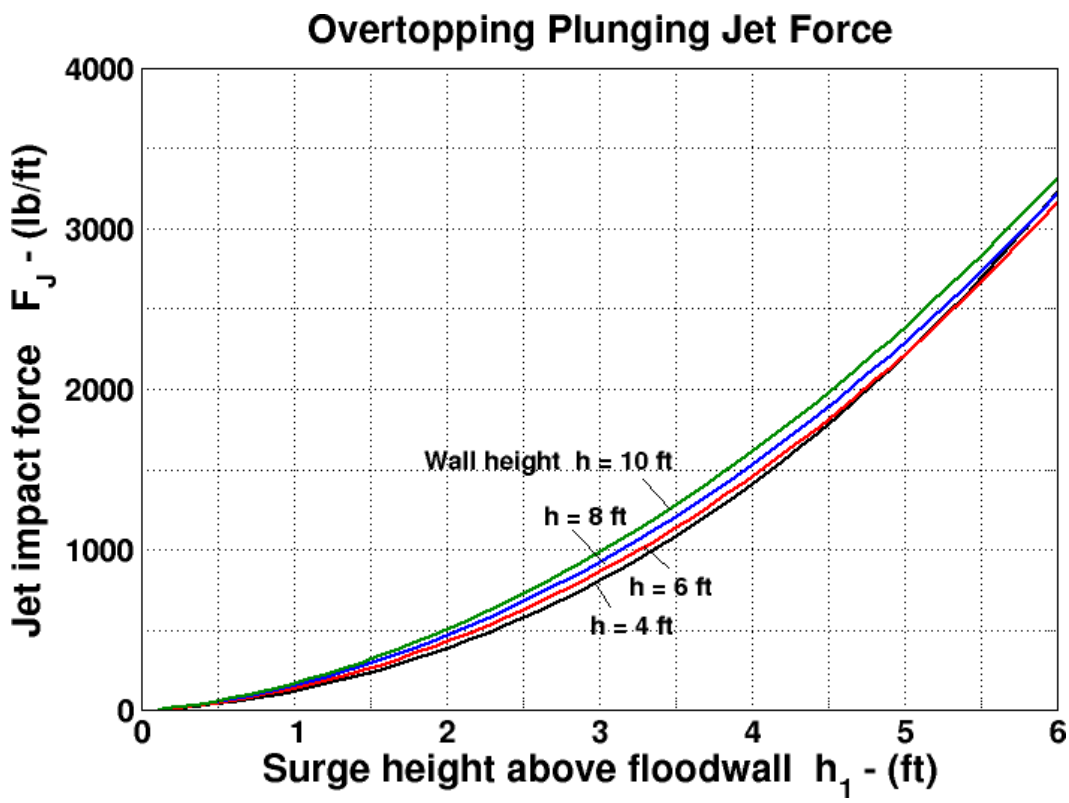

Figure 4.13. Overtopping jet impact force on the ground

The force of the overtopping jet at impact creates high pressures because the jet width is narrow (see Figure 4.10). The impact force given in Figure 4.13 can be resolved into vertical and horizontal components using the estimated jet entry angle given on Figure 4.11. Thus, the apportioning of force between vertical and horizontal components will vary with overtopping condition, and successful scour protection must be able to resist the expected range of vertical and horizontal forces. For high discharges over low walls, the jet entry angles are far from vertical, and the water after impact will retain a substantial horizontal velocity as it flows down the protected side of the earthen levee.

Depending on the elevation of the adjacent land on the protected side of the floodwall, there may be standing water at the base of the wall. The impact force of an overtopping jet will be dissipated to some degree as it enters the standing water, but it still retains sufficient force to erode unprotected foundation soil. Scour protection that relies on self-weight for stability will be less stable when submerged, and the overtopping jet may be able to dislodge submerged components of the protection.

Wave Overtopping

Waves can overtop a vertical floodwall even when the storm surge elevation is below the top elevation of the wall as illustrated by Figure 4.14. That portion of the wave above the floodwall will tumble over the wall and plunge to the ground under the force of gravity. The quantity of water will vary in time, and the unsteady discharge will be a function of wave height, wave period, and surge elevation relative to the wall. Erosion of unprotected soil will occur as the waves cascade over the wall, but the unsteadiness of the

process, coupled with the variation of impact point due to irregular waves, makes scour estimation difficult, if not impossible.

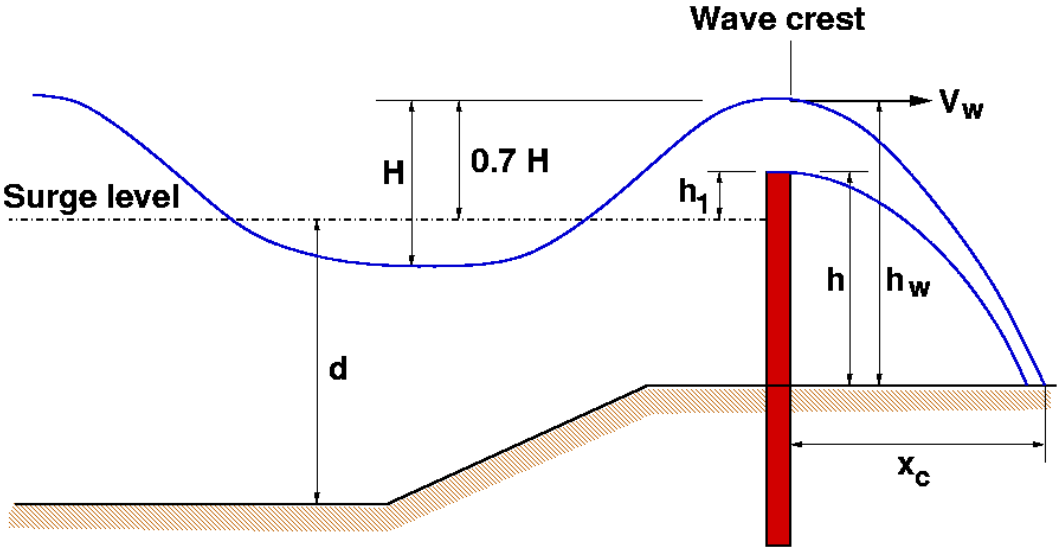

Figure 4.14. Definition sketch of wave overtopping floodwall

The hydrodynamics of this phenomenon is quite complex because a substantial portion of the incident wave is reflected by the floodwall, and the reflected wave will interact nonlinearly with the incident wave. Therefore, a few simplifying assumptions are necessary for the approximation given here.

Assume the incident waves are reasonably approximated as shallow water waves. Furthermore, assume the incident wave crest height reaches the floodwall without being modified by the reflected wave. In other words, there is no nonlinear interaction between the incident and reflected wave. Waves in deeper water are symmetrical about the still water level (swl) with the vertical distance between the wave crest and swl is the same as the vertical distance between the wave trough and swl. However, in shallow water the wave crests become more peaked and the troughs become flatter, and the vertical distance between the wave crest and the swl becomes proportionally larger. For this simple development, assume the distance of the wave crest above the swl is 70% of the wave height, *H*, as shown in Figure 4.14.

As the wave crest passes over the floodwall, the orbital velocity of water particles at the free surface will be nearly the same as the wave celerity. Using the expression for wave celerity given by third-order theory for nonlinear, shallow water waves, the horizontal velocity V_w is given by

$$V_w = C = \sqrt{g(d+H)} \tag{4.24}$$

where *g* is gravity, *d* is water depth, and *H* is incident wave height. Note that wave celerity is independent of wave period in shallow water, and instead depends only on water depth and wave height.

The distance from the wall to where the plunging wave crest impacts the ground level is found using the formulas for an object in free fall having an initial horizontal velocity of V_w and falling a vertical distance h_w. The total vertical fall distance is given as

$$h_w = h + 0.7\,H + h_1 \tag{4.25}$$

where h is the vertical distance between the top of the flood wall and the ground level, and h_1 is the distance between the top of the wall and the surge level. If the surge level is lower than the floodwall, h_1 is negative. When the surge overtops the floodwall, h_1 is positive.

The vertical fall distance is a function of fall time and gravitational acceleration, i.e.,

$$h_w = \frac{1}{2} g\, t^2 \tag{4.26}$$

Thus, the fall time for a water particle at the wave crest free surface to fall to the ground level is given by

$$t_f = \sqrt{\frac{2\,h_w}{g}} \tag{4.27}$$

The horizontal distance traversed by the water particle during this free-fall time is simply

$$x_C = V_w\, t_f \tag{4.28}$$

Substituting Eqn. 4.24 for V_w and Eqn. 4.27 for t_f into Eqn. 4.28 yields

$$x_C = \sqrt{2\left(d + H\right)\left(h + 0.7\,H + h_1\right)} \tag{4.29}$$

Figure 4.15 shows the variation in impact distance from the floodwall as a function of surge elevation relative to floodwall elevation for different floodwall heights above the ground level. These curves were calculated using Eqn. 4.29 with a wave height of $H = 4$ ft, and a water depth of $d = 16$ ft. Different curves should be generated for other values of H and d.

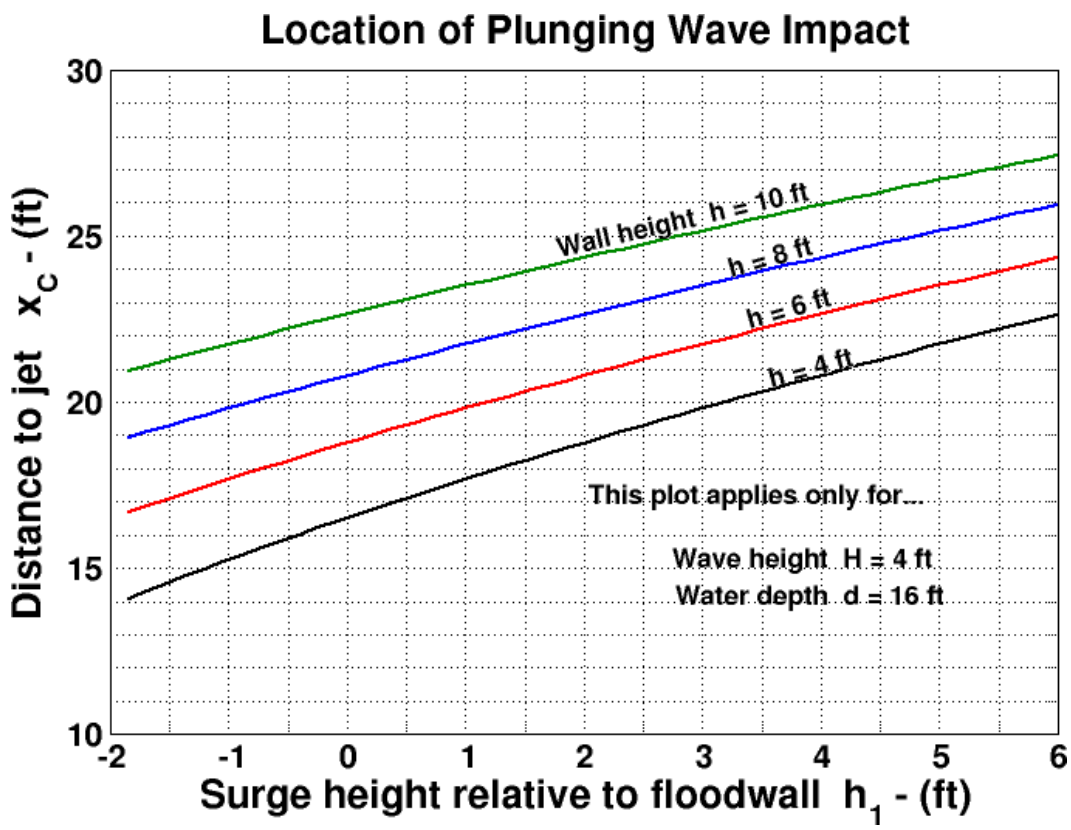

Figure 4.15. Horizontal distance between the floodwall and approximate impact point of plunging wave crest

The horizontal distance between the floodwall and the plunging wave impact point is appreciably farther than corresponding distances for surge overtopping without waves as estimated from Figure 4.9. This difference is due to the forward speed of the wave crest, which is greater than the fluid velocity of the overtopping surge. If the elevation of the surge level is substantially below the floodwall top elevation, only the highest waves will overtop the wall, and the quantity of overtopped water will be relatively small. As the surge level rises, more of the wave crests will topple over the wall, and the likelihood of scour damage increases.

Depending on the cross section of the earthen levee supporting the floodwall, the horizontal projection of the overtopping jet may over-shoot the crown of the earthen levee and impinge on the protected side slope. It this case it is a simple matter to continue the parabolic trajectory used in this analysis to estimate the point of impact on the rear slope. The easiest procedure is trial and error solution of Eqn. 4.29 until values of x_C and h correspond to the surface of the levee protected side slope.

Wave and Surge Overtopping

Where both waves and storm surge overtop the floodwall the hydrodynamics are complex, and the simple methods provided here are less valid. More research is needed to establish accurate hydrodynamic design criteria. Steady overflow associated with the storm surge elevation above the top of the floodwall is combined with the unsteady

waves propagating on top of the surge. This results in a pulsating unsteady flow over the wall with larger discharge when the wave crest passes over the wall, and decrease discharge when the wave trough is at the wall. This pulsating action affects the location of the free-falling water jet in time with the jet landing farther from the floodwall with greater flow volume when the wave crest overtops. Consequently, scour protection for the case of wave and surge overtopping must be more robust then needed for surge overtopping alone, and the protection must extend a greater distance from the protected side of the floodwall.

A first approximation of the maximum jet impact horizontal distance from the wall can be estimated using Eqn. 4.29 with h_1 specified as the distance between the surge elevation and the top of the floodwall (positive value). The actual impact distance may be slightly farther because the overtopping flow could add to the initial horizontal velocity (V_w) of the wave. The maximum impact force of the falling jet will be greater than that estimated for surge overtopping alone (see Figure 4.13).

Performance Criteria

Below are listed the key performance criteria pertinent to protection for vertical floodwalls and sheetpile walls. Many of the criteria given below are nearly identical to those given in earlier chapters. At this stage much of the performance criteria are in the form of questions related to various aspects of armor and protection performance. Some question responses may yield specific answers based on test results and/or previous experience, whereas answers to other questions may result in assigning a value such as poor, fair, good, excellent, or unknown. Performance criteria will continue to evolve as additional information is gathered.

Survivability Considerations

Survivability of floodwall toe protection on the protected side can be divided into two categories. The first category is survivability of the protection over the relatively short duration of a major hurricane event when the floodwall is overtopped and large quantities of free-falling water impact the ground with substantial force. Wave and water overtopping will cause maximum destructive loading on the protective system, and thus, constitute the critical design condition.

Evaluation of potential armoring or protection alternatives for the overtopping category should determine which of the following scenarios best describes how the system will respond to a major overtopping event where the storm surge level exceeds the top of the floodwall for as much as three hours, and the flow parameters are within the ranges estimated in the previous section.

a) The protection system is expected to survive intact with only minor damage that does not endanger the floodwall's integrity and does not result in a significant loss of foundation material that provides lateral support. Repairs may be needed, but the repairs are not urgent and can be scheduled as resources allow.

b) The protection system suffers damage; but the damage is progressive in time, and more importantly, the loss of foundation material does not ultimately result in loss of lateral support and floodwall displacement or collapse. In other words, the floodwall has sustained some damage to the scour protection and considerable loss of foundation material, but the wall remains intact through the duration of the event. Immediate repairs must be undertaken as soon as feasible.

c) The protection system holds for a while, but then fails in a catastrophic manner with nearly complete loss of protective functionality. Foundation soil will erode as if unprotected, and the floodwall is at risk as lateral supporting soil is removed. The floodwall must be repaired, and a nearly complete reinstallation of the protection is required.

The second survivability category pertains to long-term deterioration of the scour prevention system or some of its components, even in the case where the floodwall is not exposed to overtopping flow for many years. Factors that may be important include material degradation, adaptation to differential settlement of the earthen levee supporting the floodwall, gradual stress loading on components during settlement, burrowing by small mammals, and tolerance to unintended plant growth within the protective system.

Long-term survivability assumes necessary monitoring and maintenance is performed as recommended (see below).

Geotechnical Considerations

Armoring is the only practical solution for preventing scour caused by water and waves overtopping a vertical floodwall. Soil strengthening techniques and some products designed to help soil embankments resist lateral flow will most likely not withstand the direct nearly-vertical impact of the overtopping water jet. The following are the main geotechnical considerations related to armoring the levee crest on the protected side of vertical floodwalls and sheetpile walls.

a) Bearing capacity. The soil must have adequate bearing capacity to support the overlaying scour protection without significant differential settlement.

b) Soil retention. The scour protection system must be designed to prevent foundation soil from leeching out between voids in the protective layers. Excessive loss of soil could result in localized collapse of the scour protection that might rapidly spread. If a geotechnical filter fabric is placed under the protection system, it must relieve any built-up pore pressure.

c) Erosion at protection termination and tie-in locations. Ideally, the scour protection will continue some distance farther away from the wall and eventually either terminate or tie into slope protection. Where no tie in to slope protection exists, the soil abutting the protection must have sufficient strength to resist the erosive effect of the overtopping water runoff.

Construction/Installation Considerations

The following list provides the more important considerations related to installation of scour protection systems on the protected side of existing undamaged and repaired vertical floodwalls. The items are not listed in any particular order of importance.

a) Design modification. Does the scour protection method require modifying the floodwall design to accommodate the armoring system? For example, is the added weight of the protection system such that underlying soils will compact resulting in loss of levee height through settlement.

b) Site access. Some portions of the existing levee and floodwall system may have limited access for heavy equipment, or for transporting materials to the work site. What site access and maneuverability are required to install a particular protection system?

c) Equipment requirements. Are there any special equipment requirements to install a particular system that might be considered out of the ordinary? If so, how might this impact construction schedule and cost per installed area?

d) Installation skills. Are there any particular or unusual skills required to install a particular system successfully? If so, what are these skills, how can these skills be obtained by the work force, and what construction monitoring and oversight are needed to assure competent installation?

e) Installation tolerances. Does successful system installation depend on precise placement of system components? If so, what are the tolerances, what methods are used to attain accurate placement, what onsite oversight and inspection are required, and what are the consequences if tolerances are not met?

f) Rate of installation. How much scour protection can be installed in an average work week, and what are the parameters associated with this rate (personnel, equipment, etc.)?

g) Protection termination and tie-in locations. The peripheral boundaries where the protection system terminates or joins with some other form of protection are often where initial damage occurs. Scour protection must extend away from the floodwall a sufficient distance to cover the region where direct water jet impact is expected. However, overtopping water will flow laterally after impact, most likely flowing down the earthen levee slope on the protected side. This flow will have high velocities, and some of the same slope protection concepts discussed in Chapter 3 apply here. Relatively light-weight scour protection systems should be affixed to the side of the floodwall to prevent possible dislocation by uplift forces. How does the particular protection system deal with transition points? What is the recommended extent of protection coverage to assure no problems will arise at the transition between protection and no protection? Is it possible and advisable to reinforce the boundaries with a more robust form of armoring (e.g., at the toe where head cutting is likely to initiate)?

h) Immersion effects. Are there any adverse consequences arising from immersion of the scour protection? If local topography is such that overtopping water can pond immediately behind the floodwall, the immersed weight of the scour protection will be considerably less than the dry weight (less than half for concrete). The impinging jet will have reduced impact force, but the capability of the protection to resist the force by self-weight is significantly reduced.

i) Construction staging. What is the construction sequence for a particular scour protection system, and does this have any effect on installation. For example, can some system components be prefabricated offsite, and then transported to the construction site by the most economical means?

j) Safety. What are the safety concerns and issues associated with a particular protection system? Will special precautions or training be needed, and what is the plan to assure all safety measures will be strictly implemented and enforced?

k) Removable and reinstallation. It is anticipated that some levee crest elevations and associated floodwalls may need to be increased by addition of earthen material to compensate for settlement or to increase the level of protection. In such an event, can a particular scour protection system be removed and reinstalled? The reinstalled protection must provide the same level of protection afforded by the original installation, and most of the cost will be associated with re-handling the armoring and not purchase of large quantities of new armoring materials

Maintenance and Repair Considerations

Long-term maintenance of the floodwall foundation scour protection is paramount for assuring continual integrity of the southeast Louisiana levee system. Without proper inspection, maintenance, and repair of deteriorated or damaged sections of floodwall scour protection, risk of damage from hurricanes weaker than the design storm increases. Below are evaluation considerations related to maintenance and repair of armoring systems.

a) Maintenance requirements. What are the specific maintenance requirements for a particular scour protection system? Is special equipment or specific skills required for ongoing maintenance?

b) Timing for maintenance. Is maintenance for a particular system performed at regular intervals, or only when needed as determined by inspection?

c) Inspection. How often is inspection recommended for a particular protection system? What is the recommended inspection technique? Which aspects of the system should be inspected? How much of the protection can be inspected in a day? Are any special tests or testing apparatus required to conduct inspections?

d) Signs of deterioration. What are the signs that a protection system is deteriorating, and can these signs be readily detected during inspection? What are the indicators that maintenance needs to be performed?

e) Maintenance costs. What costs are associated with maintenance beyond personnel time? For example, does usual maintenance require a significant mobilization of equipment?

f) Damage repair procedures. After episodes resulting in significant or wide-spread damage, what are the repair procedures? How is the repaired section tied into the adjacent undamaged protection? Can small sections of isolated damage be repaired by a small crew using readily obtained equipment?

g) Robustness of repair. Will repaired sections of damaged protection retain the full strength and resistance to damage as the original installation, or will the repair section represent a weakened area that may require additional strengthening?

h) Safety during inspection, maintenance, and repair. Are there any safety concerns or safety procedures specific to a particular protection system? Are there any additional risks working near a damaged portion of the protection beyond those that could be reasonably identified or anticipated?

Environmental Considerations

The following is a list of considerations related to environmental consequences that might apply to some scour protection alternatives.

a) Environmentally sensitive areas. Are there any aspects of the scour protection system that might make it difficult to deploy on floodwalls located in environmentally sensitive areas?

b) Toxic materials. Does the protection system contain any toxic materials or chemicals that might be released into the environment either during installation or over time due to deterioration? Are there any special treatments or handling considerations to assure no toxics are released?

c) Endangering animals or plant species. Are there any aspects of the protection system that might be considered detrimental or dangerous to local plant and animal species?

Overview of Concrete and Sheetpile Floodwall Protection

The forceful, near vertical, impact of falling water due to surge and wave overtopping at vertical floodwalls imposes loads on the protection system that are vastly different than loads exerted by water flowing parallel to the protection surface. As a consequence, armoring systems fully capable of protecting backside slopes of levees and earthen levee transitions may not be appropriate for protecting the levee crown soil on the protected side of an overtopped floodwall. For example, individual stones will be dislodged in riprap protection, turf reinforcement mats might not withstand forces applied perpendicular to the mat, and soil or small stones used as geocell fill will be flushed out by the water.

This section briefly overviews four protection alternatives that have sufficient strength, rigidity, and robustness to withstand high impact loads from overtopping water jets without loss of functionality. All the options have the disadvantage of adding significant weight to the levee foundation, and this could be problematic where soil is weak. The following are considered to be viable alternatives for armoring floodwalls on the protected side:

a) Poured-in-place reinforced and non-reinforced concrete

b) Grouted stone riprap

c) Rock-filled mattresses

d) Articulated concrete mats

Below are brief generic descriptions of these protection systems.

Poured-in-place reinforced and non-reinforced concrete. Levee soil can be protected by an impermeable, continuous, reinforced concrete slap containing light reinforcement mesh. Alternately, the concrete slab can be made thicker without reinforcing. The slab is formed, and concrete is poured in place to cover the area from the base of the floodwall protected side out a distance beyond the expected splash-down point of the overtopping jet. The slab can be tied into the floodwall using a variety of techniques. This provides a rigid horizontal surface that can absorb the impact of falling water and divert the overtopping jet toward the backside slope of the earthen levee. Advantages include high strength and durability, readily available materials, and flexibility to vary project dimensions as needed. Where appropriate, the concrete apron can be designed as a roadway for vehicular traffic. The main disadvantage of reinforced and non-reinfoced concrete is its relative intolerance to differential settlement. Where future plans call for addition of levee height, concrete aprons cannot be easily removed and re-used.

Grouted stone riprap. This protection method consists of conventional riprap armoring placed on top of a bedding layer and then filled with a concrete grout mixture. The purpose of the grout is to solidify the riprap protection into a solid, continuous, impermeable structure and to prevent loss of individual stones when impacted by the falling water jet. Because the grout mixture has minimal strength in tension, grouted stone riprap will have little tolerance for differential settlement of the underlying levee

crown. Once the bond between adjacent stones is broken, riprap stones can be dislodged by the overtopping flow. Advantages of grouted stone riprap are ease of installation, capability to protect varying terrain, and ease of removal for future increases in levee height. However, the removed riprap is not readily re-usable because much of the grout will remain intact. The main disadvantage of grouted stone riprap is the uncertainty associated with the long-term integrity of the grout/stone bonds if there is any ground settlement.

Rock-filled mattresses. Rock-filled mattresses are containers fabricated of geogrid material and filled with small rocks varying in size from 2 inches up to about 5 inches. Mattresses are placed directly on top of a geotextile filter cloth or conventional gravel filter layer. Rock-filled mattresses are flexible, and they can adapt to terrain changes easily. They are also tolerant of differential settlement, and they will continue to be fully functional if the ground settles beneath them. Overtopping water landing on the mattress fills the voids between stones and helps reduce the flow energy. Soil could be placed over the mats to support vegetative growth. For application at the base of floodwalls, special attention is needed to assure mattresses are placed with minimal gaps between adjacent units. Gaps between mattresses are weak points that could allow soil to escape if the geotextile is punctured. Advantages of rock-filled mattresses include lower cost for smaller stone, rapid installation, off-site fabrication, and the capability to remove the protection and re-use the mattresses if the levee needs to be raised. Disadvantages of rock-filled mattresses include the need for heavy equipment to lift and place the mats, potential gaps between adjacent mats and next to the floodwall, and long-term durability of the geogrid material when subjected to UV radiation. Whereas the mats could support vehicular traffic, there is a risk of damaging the geogrid material or the lacing that holds the mats together.

Articulated concrete mats. Articulated concrete mats consist of concrete block units linked together with cables made of metal or other high-strength material. Blocks can be solid or open, with gaps between adjacent blocks. Articulated concrete mats are fabricated off-site and rapidly installed using heavy lifting cranes. The concrete blocks have sufficient strength to resist the battering of overtopping jets of water, but the gaps between the blocks could allow underlying soil to erode. Therefore, these mats will be most effective if placed over a stone or gravel bedding layer sized to prevent movement of the gravel through the gaps in the mat. Articulated concrete mats are flexible and very tolerant of differential settlement. The mats are easily removed and re-used without any loss of effectiveness, and they have no problem supporting low-speed vehicular traffic. Advantages of articulated concrete mats include off-site fabrication, rapid placement, capability to cover irregular terrain, tolerance to differential settlement, and long service life. Disadvantages include the need for heavy-lift cranes during installation and providing adequately-sized gravel underlayers to prevent loss of material through gaps.

Alternative: Poured-in-Place Reinforced and Non-Reinforced Concrete

1. Manufacturer.

   No specific manufacturer.

2. Product Description.

   Poured-in-place concrete provides effective armoring of the levee crown soil on the protected side of a vertical floodwall. The concrete apron is formed, and concrete is poured in place to cover the area from the base of the floodwall protected side out a distance beyond the expected splash-down point of the overtopping jet. Concrete offers great flexibility for protecting odd-shaped areas, gaps between the floodwall and existing structures as shown in Figure 4.16, and around corners in the floodwall protection. Reinforced concrete slabs can be thinner because the reinforcing mesh resists tension loads. The slab can be tied into the floodwall using a variety of techniques. Details of reinforced and non-reinforced concrete aprons are shown on Figures 4.16 and 4.17, respectively. These specific plans are being implemented by Task Force Guardian.

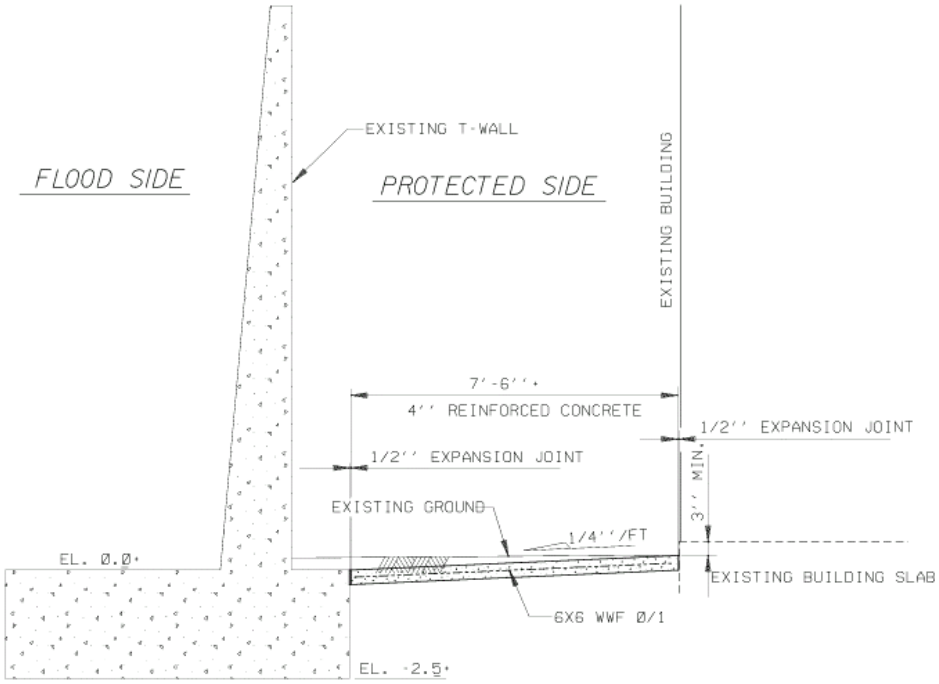

Figure 4.16. Detail of 4-inch-thick reinforced concrete apron (from URS drawing for IHNC West side)

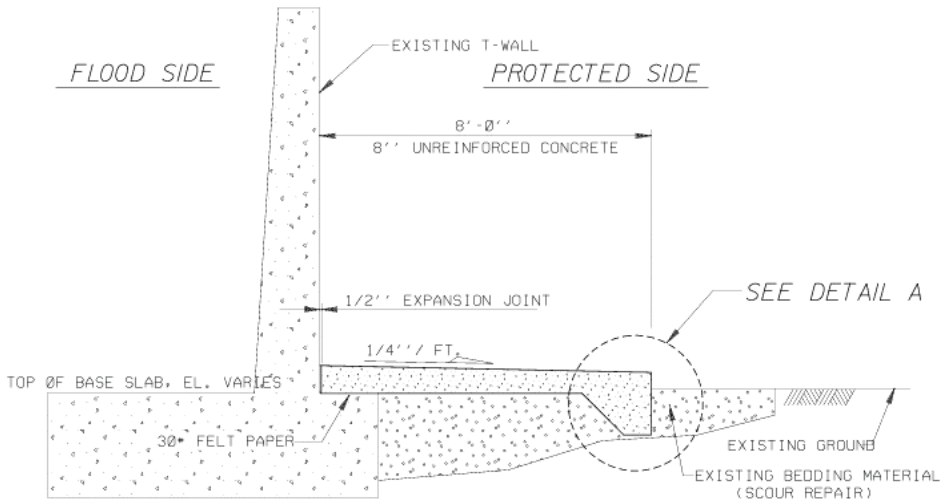

Figure 4.17. Detail of 8-inch-thick unreinforced concrete apron (from URS drawing for IHNC West side)

3. Product Functionality.

   Levee soil is protected by an impermeable, continuous, concrete slap with or without light reinforcement mesh. This provides a rigid, nearly horizontal surface that can absorb the impact of falling water and divert the overtopping jet toward the backside slope of the earthen levee.

4. Stated Applications.

   Applications of formed and poured-in-place concrete to control flow and prevent scour are wide spread and very successful. Implementations illustrated in Figures 4.16 and 4.17 are most appropriate, and these designs should be fully successful under design load conditions.

5. Potential Failure Modes and Mechanisms.

   The loads to which the concrete slab might be subjected are not well defined, and this makes design of the slab difficult. If the slab remains on firm footing with no loss of underlying material, loads generated by the falling jet of water should be transferred to the foundation. However, if the ground beneath the slab settles, there may be locations where the slab spans a void and must function like a beam. The slab will crack if the reinforcement mesh is not near the bottom, and this could lead to partial breakup of the slab. Alternately, if a portion of the slab is cantilevered by loss of supporting material at the outer edge, the reinforcement mesh is then needed near the top surface of the slab.

6. Application Limitations.

   There are few limitations on poured-in-place concrete slabs. Near full strength is attained in about one month, and strength continues to increase slowly for some time.

7. Documented Applications.

   Numerous.

8. Costs.

   Cost is a function of project location, site accessibility, coverage, slab thickness, and reinforcement. Preparation costs will vary. The experience of Task Force Guardian should provide an idea of installed costs.

9. Technical Evaluation Relative to Performance Criteria.

   a) Survivability Criteria. Concrete has excellent survivability characteristics. Properly designed slabs should withstand the dynamic forces, and the relatively short duration of overtopping events precludes erosion of the concrete surface. Properly prepared concrete is durable, and it weathers well.

   b) Geotechnical Criteria. Concrete provides an impermeable barrier, so any loss of underlying soil will be at the slab boundaries or perhaps through the activities of burrowing animals. The underlying soil must provide adequate bearing capacity for the slab (and any anticipated vehicular traffic) without differential settlement.

   c) Construction/Installation Criteria. Cracks will form during the concrete curing process, so steel mesh must have sufficient coverage so corrosion does not occur. Steel corrosion will cause spalling and a reduction in slab width. Usual practices must be followed as with any poured concrete slab, e.g., water should not be added to increase the concrete flow characteristics during placement, etc. Expansion/contraction joints are necessary, and it may be advisable to tie the slab into the existing floodwall.

   d) Maintenance and Repair Criteria. Concrete requires little maintenance. If inspection indicates an area of deteriorating concrete due to corrosion of reinforcement or spalling due to poor quality materials, those sections should be cut out and replaced with new concrete.

   e) Environmental Criteria. Concrete slabs do not cause any environmental problems. Site access may disrupt the local ecology temporarily.

   f) Design Requirements. Conventional concrete slab design for typical dead and live loads is well understood and dictated by building codes. Slab resistance to the impact loading of falling water caused by wave and surge overtopping is not as well understood. An initial estimate of the total force in the water jet (per unit length along the floodwall) is provided by Figure 4.13 for the case of surge overtopping. The associated bearing pressure can be estimated using Figure 4.10 to find the jet thickness at impact. Apply the resulting pressure as a live load. It might be prudent to include a factor of safety given the uncertainty of wave overtopping loads.

10. Summary of Poured-in-Place Concrete Alternative.

a) Advantages. Advantages of poured-in-place reinforced and non-reinforced concrete include high strength and durability, readily available materials, and flexibility to vary project dimensions as needed. Where appropriate, the concrete apron can be designed as a roadway for vehicular traffic. Where site access is limited, concrete can be placed using a crane bucket or by pumping short distances.

b) Disadvantages. The main disadvantage of reinforced and non-reinfoced concrete is its relative intolerance to differential settlement. Buckled sections of the paved area are more apt to allow leaking of underlying soil. Where future plans call for addition of levee height, concrete aprons cannot be easily removed and re-used.

c) Risk and uncertainties. The suggested method for estimating the live loads due to overtopping water are approximate, and wave overtopping has not been included. The estimated load is considered a live load, but the impact force created by initial splash-down of the jet is not included in the force estimate.

Alternative: Grouted Stone Riprap

1. Manufacturer.

   No specific manufacturer.

2. Product Description.

   This protection method begins with conventional riprap armoring placed on top of a bedding layer and geotextile filter fabric. The voids in the riprap are then filled with a concrete grout mixture. The final protection is a solid, impermeable protection layer. Figure 4.18 below illustrates typical project dimensions for rehabilitation of scour holes caused by floodwall overtopping during Hurricane Katrina.

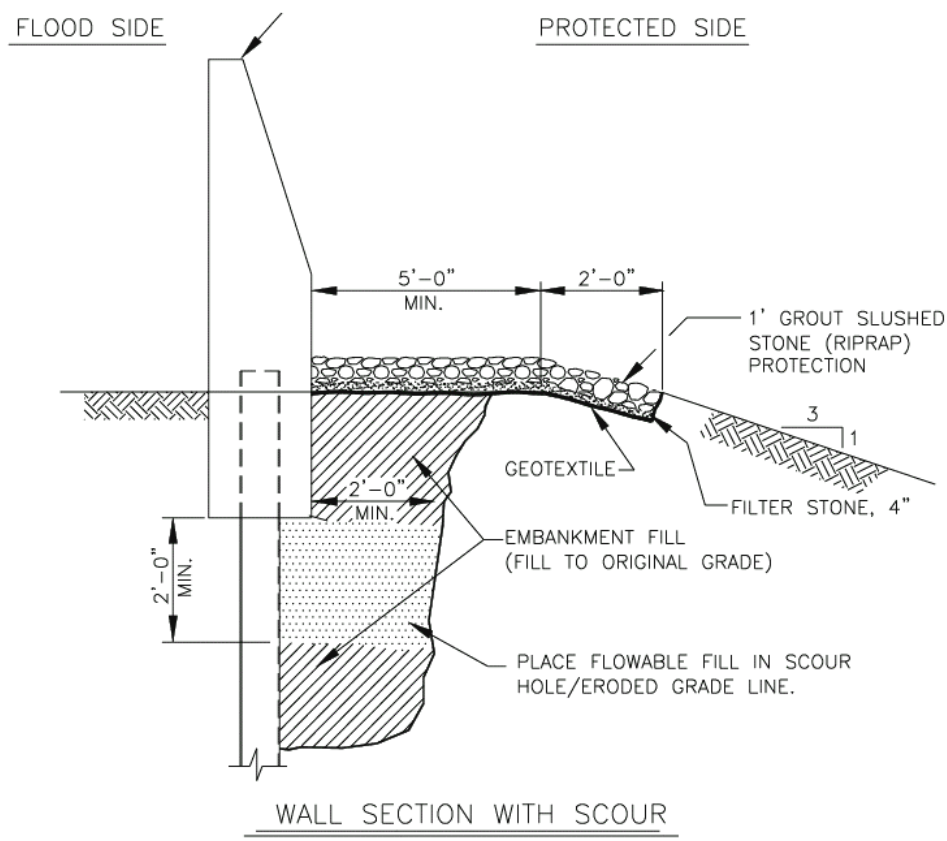

Figure 4.18. Detail of grouted stone riprap floodwall apron (from URS drawing for IHNC East side)

3. Product Functionality.

   The purpose of the grout is to solidify the riprap protection into a solid, continuous, impermeable structure, and to prevent loss of individual stones when impacted by the falling water jet. Whereas the grouted riprap might support vehicular traffic, the risk

of damage is too great, and vehicles should be banned from driving on the protection. The underlying soil is shielded from the forces of falling water, and the only loss of soil might occur at the project boundaries if steps are not taken to prevent erosion.

4. Stated Applications.

   Grouted riprap has been used successfully at numerous locations as protection against water flowing parallel to the armoring. It is not known whether or not grouted riprap has been used where high quantities of overtopping water are expected to impact with forces normal to the slope.

5. Potential Failure Modes and Mechanisms.

   Because the grout mixture has minimal strength in tension, grouted stone riprap will have little tolerance for differential settlement of the underlying levee crown soil. Once the bond between adjacent stones is broken, riprap stones can be dislodged by the overtopping flow, and this could start an unraveling of the protection. Poor quality grout will be ineffective and easily broken by the force of water impact. Deterioration of grouted riprap is expected to occur more rapidly than for concrete slabs. Grouted riprap will not expand and contract with temperature change as much as concrete, but expansion and contraction might cause the grout to crack and break.

6. Application Limitations.

   Grouted riprap should not be used where foundation conditions cannot support the weight of the protection or where different soil types might cause differential settlement of the monolithic protection. It would be advisable to have expansion/contraction joints between the riprap and the floodwall, and expansion/contraction joints perpendicular to the floodwall at given spacing.

7. Documented Applications.

   The report authors are not aware of documented cases of grouted riprap used where the protection must resist high volumes of falling water, but that does not mean such applications do not exist. Grouted riprap has been successful in numerous other applications where water flows parallel to the protection.

8. Costs.

   Costs for grouted riprap are unknown, but the experience of Task Force Guardian's implementation of similar protection in the reconstruction of damaged levees and floodwalls in New Orleans should provide sufficient cost guidance.

9. Technical Evaluation Relative to Performance Criteria.

   a) Survivability Criteria. Survivability of grouted riprap to protect foundation soils against surge and wave overtopping is not proven. The main weakness is the inability of the grout to withstand tensile stresses, and the possibility of individual stones breaking free and becoming dislodged. The long-term

durability of grouted riprap will be a function of foundation stability and quality of the cement grout.

b) Geotechnical Criteria. The foundation soil must be strong and well compacted to prevent differential settlement. Steps must be taken at the protection boundaries to prevent erosion of supporting soil. This is critical where the riprap ends on the protected side of the earthen levee. Water flowing down the slope will erode the soil as it passes over the terminus of the grouted riprap.

c) Construction/Installation Criteria. Dumped riprap must be checked for good distribution of riprap material sizes. Avoid hotspots where there is a congregation of smaller stones. Grout must be of high quality and only fluid enough to assure that all the voids in the riprap are filled.

d) Maintenance and Repair Criteria. Sections of grouted riprap can be repaired by replacement of the damaged section. However, this patched area will not be well tied into the neighboring intact section, and this might cause a weakness in the protection.

e) Environmental Criteria. Grouted riprap does not cause any environmental problems.

f) Design Requirements. Guidance on the design and construction of grouted riprap revetments is given in the Corps of Engineers' Technical Letter, "Design and Construction of Grouted Riprap" (Corps of Engineers, 1992).

10. Summary of Riprap Alternative.

a) Advantages. Advantages of grouted stone riprap are ease of installation, capability to protect varying terrain, and easy removal for future increases in levee height. However, the removed riprap is not readily re-usable because much of the grout will remain intact. Grouting provides increased stability for riprap that would be dislodged by the overtopping flow.

b) Disadvantages. The main disadvantage of grouted stone riprap is the uncertainty associated with the long-term integrity of the grout/stone bonds if there is any ground settlement. Also, cracks will form around larger stones, and this could lead to gradual deterioration of the grout bonding.

c) Risk and uncertainties. The main uncertainty of grouted riprap is its resistance to large impact forces associated with overtopping jets of water. There is little evidence of grouted riprap being used for this particular application.

Alternative: Rock-Filled Mattresses

1. Manufacturer.

   Marine Mattress
   Tensar Earth Technologies, Inc.
   5883 Glenridge Drive
   Suite 200
   Atlanta, GA 30328-5363
   (888) 828-5126 Toll Free
   (404) 250-1290 International
   (404) 250-0461 Fax
   www.tensarcorp.com

2. Product Description.

   Rock-filled mattresses, often referred to as marine mattresses, are containers fabricated of geogrid material and filled with small rocks varying in size from 2 inches up to about 5 inches. Mattresses are placed directly on top of a geotextile filter cloth or conventional gravel filter layer. Rock-filled mattresses can be fabricated and filled off-site and transported by truck or barge to the job site. Mattresses dimensions are typically 5-ft wide and up to 35 ft long. Depending on the application, mattress thickness can be as little as 4 inches or as large as 2 ft. For application as floodwall overtopping protection mattress thickness should probably be at least 6 inches thick.

3. Product Functionality.

   Rock-filled mattresses are flexible, and they can adapt to terrain changes easily. They are also tolerant of differential settlement, and they will continue to be fully functional if the ground settles beneath them. Overtopping water landing on the mattress fills the voids between stones and helps reduce the flow energy. Soil could be placed over the mattresses to support vegetative growth. The surface of a rock-filled mattress is not intended for vehicular traffic, and the surface may become a slipping hazard if placed on a slope.

4. Stated Applications.

   Rock-filled mattresses have been used as revetments, scour protection, foundation mats, and for protection at culverts and bridge abutments. The writers are not aware of any applications where rock-filled mattresses were intended to resist the forces of water impacting normal to the mattress.

5. Potential Failure Modes and Mechanisms.

   Rock-filled mattresses fail if the supporting container is breached either by failure of the geogrid material or by failure of the lacing and connectors used to construct the cage. The geosynthetic materials used to construct the mattresses are treated against UV radiation, but the long-term (tens of years) durability of the material is unknown.

Mattress protection can also fail if the mattress is lifted by the hydrodynamic forces and displaced laterally as a unit. This might occur if the mattress is too thin relative to the lifting force. Erosion might occur at the mattress boundaries, but the flexible nature of the mattress allows it to slump into any scour hole and continue to provide a reasonably high degree of functionality.

6. Application Limitations.

Rock-filled mattresses add a considerable weight to the levee crown, and they should not be used where foundation soils cannot bear the additional weight. Heavy equipment is required for installation, so site access is a critical issue.

7. Documented Applications.

Numerous field applications including USACE applications as breakwater and revetment foundation support, contaminated sediment cap, and streambank protection. See U.S. Army Engineers Technical Note ERDC/CHL CHETN-III-72, "Uses of Marine Mattresses in Coastal Engineering" (available at http://cirp.wes.army.mil/cirp/cetns/chetn-iii-72.pdf). There are no documented applications where mattresses were expected to resist the impact forces of falling water. However, marine mattress have been reported to be stable as revetments in waves as high as 8 ft. This condition could have generated breaking wave impacts similar to the impact of surge overtopping a floodwall.

8. Costs.

Initial cost estimates can be derived from the table below that was reproduced from the above-cited Technical Note. Installed costs for rock-filled mattresses depend on such factors as application, proximity and cost of rock-fill material, site accessibility, placement method (land-based or from barge), availability of equipment, and project size.

Table 1 Installed Mattress Cost per Square Foot

Application	Mattress Placement	Mattress Thickness	Cost per square foot
Breakwater construction	In water	12 in.	$15
Riverbank revetment	On land	12 in.	$10
Revetment foundation	In water	6 in.	$13

9. Technical Evaluation Relative to Performance Criteria.

a) Survivability Criteria. Rock-filled mattresses should be capable of withstanding the forces of surge and waves overtopping a vertical floodwall; however, this aspect has never been tested to the knowledge of the report authors. The one

weakness might be where adjacent mattresses abut if any gaps are allowed. Water hitting any gaps could rupture the underlying geotextile filter fabric and allow soil to erode. Long-term durability depends on the effectiveness of the geogrid and lacing material UV resistance. Mattresses covered with a layer of vegetated soil should have excellent service life.

b) Geotechnical Criteria. Rock-filled mattresses are heavy, and the levee soil must be able to support the weight of the armoring system. However, the system will probably weigh less than comparable grouted riprap solutions. The flexible nature of the mattress allows them to adapt to differential settlement or local losses of underlying soil. Mattress deployment requires minimal compacting of soil, and soil surface preparation requirements are minimal beyond grooming of the soil in preparation for covering with filter cloth.

c) Construction/Installation Criteria. For application at the base of floodwalls, special attention is needed to assure mattresses are placed with minimal gaps between adjacent units. Gaps between mattresses are weak points that could allow soil to escape if the geotextile is punctured. Mattresses are placed by heavy cranes, and adequate site access is needed. Placement from barges is also an option.

d) Maintenance and Repair Criteria. Ruptures to the mattress containers can be repaired in-situ using a patching technique. Extensive mattress damage is repaired by removing the entire mattress and replacing with a new unit.

e) Environmental Criteria. There are no environmental impacts associated with rock-filled mattresses.

f) Design Requirements. There is ample guidance related to mattress fabrication for best service life, but no design guidance exists suggesting appropriate mattresses thicknesses to resist a given overtopping water force load.

10. Summary of Rock-Filled Mattress Alternative.

a) Advantages. Advantages of rock-filled mattresses include lower cost for smaller stone, rapid installation, off-site fabrication, and the capability to remove the protection and re-use the mattresses if the levee needs to be raised.

b) Disadvantages. Disadvantages of rock-filled mattresses include the need for heavy equipment to lift and place the mats, potential gaps between adjacent mats and next to the floodwall, and long-term durability of the geogrid material when subjected to UV radiation.

c) Risk and uncertainties. Behavior of rock-filled mattresses when subjected to the forces of overtopping water is largely unknown. Whereas the mats could support vehicular traffic, there is a risk of damaging the geogrid material or the lacing that holds the mats together if vehicular traffic is allowed.

Alternative: Articulated Concrete Mats

1. Manufacturer.

   Several commercial manufacturers. For example…

   ARMORTEC
   Mid-South Regional Manager
   301 Pascoe Boulevard
   Bowling Green, KY 42104
   Phone: 270-843-4659
   Mobile: 270-535-3539
   Fax: 270-783-8959
   E-Mail: dbkees@armortec.com

   Submar, Inc.
   805 Dunn Street
   Houma, LA 70360
   Email: submar@submar.com
   Phone: 985-868-0001
   Fax: 985-851-0108
   Toll free: 800-978-2627

   The Mat Sinking unit of the Corps of Engineers produces articulated concrete mats annually for bank protection on the Mississippi River.

2. Product Description.

   Articulated concrete mats consist of concrete block units linked together with cables made of metal or other high-strength material. Mattress thickness varies between manufacturer and intended application with the thickness range between about 5 to 12 inches. Articulated concrete mats are fabricated off-site and rapidly installed using heavy lifting cranes. Mattresses are laid over a filter layer, typically a geotextile fabric, and adjacent mattresses are interlocked or cabled together to form continuous coverage.

3. Product Functionality.

   The cabling between blocks serves two purposes: (1) the cabling holds the blocks together so they can be lifted as a unit for placement, and (2) the cabling provides additional mattress stability and prevents loss of individual blocks. The concrete blocks have sufficient strength to resist the battering of overtopping jets of water, but the gaps between the blocks could allow underlying soil to erode. Therefore, these mats will be most effective if placed over a stone or gravel bedding layer sized to prevent movement of the gravel through the gaps in the mat. The mats are strong, durable, and they have no problem supporting low-speed vehicular traffic.

4. Stated Applications.

Articulated concrete mats have been used in a wide variety of applications related to protecting soils from flowing water. They are even appropriate as protection against small waves. It is not readily apparent if concrete mats have been used specifically to resist the forces of overtopping water impact normal to the mat. For use as foundation armoring near the protected-side base of vertical floodwalls, perhaps the most appropriate mat would be similar to those constructed by the Corps' mat-sinking unit. These mats have larger rectangular concrete blocks with fewer gaps. The mats are not as flexible as some of the commercial mats, but this particular application is mostly flat, narrow areas without terrain variation (in contrast to the need for articulation at levee transitions).

5. Potential Failure Modes and Mechanisms.

Concrete mats should have sufficient self-weight to prevent lifting and lateral shifting. Anchoring is an option for the mats. The main concern is loss of underlying soil through gaps, even if covered with a geotextile that could be breached by the falling water impact. For this reason it is advisable to use mats with larger concrete area and smaller gap area. Mats should be placed over a gravel filter layer with stone sizes greater than the gap width. Cable breakage could result in block displacement and erosion of soil in a localized area, but the damage is not likely to spread without wholesale cable breakage.

6. Application Limitations.

Foundation soils must be able to support the additional weight of the mats. Coverage pattern (long dimension parallel or perpendicular to the wall) will be dictated by the particular mat geometry.

7. Documented Applications.

There are numerous successful applications of articulated concrete mattresses used to protect against flow parallel to the mat, including the experience of the Corps of Engineers' Mat Sinking Unit. Experience related to water forces applied normal to the mats is limited to breaking of small waves. Very heavy mats may have been used to prevent scour at dam spillways.

8. Costs.

Typical costs were unavailable at the time of this writing.

9. Technical Evaluation Relative to Performance Criteria.

a) Survivability Criteria. Articulated concrete mats are expected to have good survivability characteristics during short-term overtopping events. Even if some of the underlying soil is lost during an extreme event, the mattress protection retains most of its functionality. The mats are very durable over the long term with corrosion of the cabling being the only concern.

b) Geotechnical Criteria. The underlying soil must be able to support the mattress weight without undue differential settlement, and the geotextile filter fabric must provide continuous coverage to retain the soil while relieving built-up pore pressure. The smallest stones in the bedding layer must be larger than the gaps between the concrete blocks.

c) Construction/Installation Criteria. Mattresses are fabricated off-site and delivered by flatbed trucks (or barges) to the site. The mattresses require heavy equipment for installation. When placing the mattresses special attention should be given to minimizing gaps between adjacent mats so bedding stone is not lost.

d) Maintenance and Repair Criteria. Generally, articulated concrete mats require no maintenance. If differential settlement becomes problematic, individual mats can be lifted out, and fill soil can be added and compacted before replacing the mat. If mattress cabling corrodes, the entire mattress can be replaced.

e) Environmental Criteria. Installation of articulated concrete mattresses does not cause any adverse environmental consequences. Mattresses do have aesthetic appeal versus riprap protection.

f) Design Requirements. Individual manufacturers provide design information and installation guidelines. The most important parameter is appropriate mattress thickness because this influences the installed cost of the protection. Unfortunately, no guidance exists at present to make this determination.

10. Summary of Articulated Concrete Mat Alternative.

a) Advantages. Advantages of articulated concrete mats include off-site fabrication, rapid placement, capability to cover irregular terrain, tolerance to differential settlement, and long service life. The mats are easily removed and re-used without any loss of effectiveness.

b) Disadvantages. Disadvantages of articulated concrete mats include the need for heavy-lift cranes during installation and providing adequately-sized gravel underlayers to prevent loss of material through gaps.

c) Risk and uncertainties. As with all the alternatives for protecting the base of floodwalls, the greatest unknown is how the system responds to high impacts of overtopping surge and waves.

References

Corps of Engineers. 1992. "Design and Construction of Grouted Riprap," Engineering Technical Letter No. 1110-2-334, U.S. Army, Corps of Engineers, Washington, D.C.

Chow, V. T. 1959. *Open-Channel Hydraulics*, McGraw-Hill Book Co., New York.

Henderson, F. M. 1966. *Open Channel Flow*, MacMillian Publishing Co., Inc., New York.

Hoffmans, G. J., and Verheij, H. J. 1997. *Scour Manual*, A.A. Balkema, Rotterdam.

Milne-Thompson, L. M. 1960. *Theoretical Hydrodynamics*, MacMillian Publishing, Inc., New York.

Morris, H. M. 1963. *Applied Hydraulics in Engineering*, The Ronald Press Co., New York.

Rouse, H. 1961. *Fluid Mechanics for Hydraulic Engineers*, Dover Publications, Inc., New York.

Seed, R. B., Nicholson, P. G., Dalrymple, R. A., Battjes, J. A., Bea, R. G., Boutwell, G. P., Bray, J. D., Collins, B. D., Harder, L. F., Headland, J. R., Inamine, M. S., Kayen, R. E., Kuhr, R. A., Pestana, J. M., Silva-Tulla, F., Storesund, R., Tanaka, S., Wartman, J., Wolff, T. F., Wooten, R. L., and Zimmie, T. F. 2005. "Preliminary Report on the Performance of the New Orleans Levee Systems in Hurricane Katrina on August 29, 2005," Report No. UCB/CITRIS-05/01, American Society of Civil Engineers, National Science Foundation, and Center for Information Technology Research in the Service of Society.