Analysis of the Stability of I-Walls with Gaps between the I-Wall and the Levee Fill

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Abstract: Following Hurricane Katrina an extensive investigation of the performance of floodwalls in the New Orleans area was undertaken by the U. S. Army Corps of Engineers and others. This investigation included detailed study of failures of cantilevered sheet pile "I-walls" during the hurricane. An important lesson from this investigation was that gaps can form on the canal side of I-walls as the water rises in the canal and causes the I-wall to deflect. Once formed, these gaps filled with water, resulting in significantly higher loads on the walls. Gap formation was a key factor in several I-wall failures, and modeling such gaps correctly is clearly an important aspect of analyzing I-wall stability. This paper describes simple procedures for estimating the depths of gaps behind I-walls, for calculating the loads to which they are subjected, and for including them in stability analyses. The effects of gaps on the stability of the 17th Canal and the London Avenue Canal I-walls are discussed.

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Introduction

Soon after Hurricane Katrina, the Department of Defense formed the Interagency Performance Evaluation Taskforce (IPET) to study the behavior of the flood control system during the hurricane. Task Group 7, the Floodwall and Levee Performance Analysis Team, was charged with analyzing the performance and stability of flood control structures. Over 50 different structures were analyzed as part of the IPET investigation.

The IPET investigation showed that an important aspect of flood wall performance was related to formation of a gap between the sheet pile "I-walls" and the levee embankment and foundation soils. As the water rose on the flood side of the wall, the sheet pile deflected. In some locations a gap formed between the I-wall and the embankment, and this gap filled with water. The water in the gap applies a hydrostatic pressure to the wall, adversely affecting stability.

This paper describes the considerations involved in predicting the depth of a water-filled gap behind an I-wall. The procedures for modeling gaps that extend to the bottom of the I-wall, and gaps that extend only part way down the wall, are presented. The important effects of a gap on wall stability are illustrated by failures of I-walls at the 17th Street Canal and the London Avenue Canal in New Orleans.

Evidence of Gap Formation

Observations made during field reconnaissance after Hurricane Katrina indicated that gaps formed at several I-wall sections due to the canal water loads. A photograph of a gap behind the Michoud Canal I-wall is shown in Fig. 1. Although a gap formed behind the Michoud Canal wall, the wall did not fail. As shown in the figure, it is common for the exposed portions of the sheet pile to be encased in reinforced concrete.

Because of their severe effect on I-wall stability, study of gap formation became a central element of the IPET investigation (IPET 2007). The IPET investigation included physical modeling (centrifuge tests), soil-structure interaction analyses, and limit equilibrium analyses. These studies showed that:

1. Formation of a gap always reduces the factor of safety against I-wall instability.
2. Although the factor of safety decreases when a gap forms, it does not always decrease to a value of 1.0 or less.
3. It is possible to evaluate the likelihood of gap formation through soil-structure interaction analyses and centrifuge model tests. Gaps are more likely to form when the foundation soils on the protected side of the wall are soft and compressible, and less likely to form when those soils are stiff.
4. Because evaluation of soil stiffness is difficult, and because soil stiffness is highly variable, it is not possible to predict with a high degree of reliability whether or not a gap will form at a particular location behind an I-wall. The IPET investigation showed that gaps formed at some locations, but gaps did not form in adjacent seemingly identical locations.
5. Given the impossibility of predicting with confidence whether or not a gap will form, it should always be assumed that a gap will form when an I-wall is loaded by water above the top of the levee in which it is embedded.

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Fig. 1. Photograph of gap on canal side of Michoud Canal I-wall in New Orleans after Hurricane Katrina

Corps of Engineers E-99 Field Test

Prior to Hurricane Katrina the Corps of Engineers performed a field test in the eastern Atchafalaya basin that involved constructing a 200 ft long section of PZ-27 steel sheet pile I-wall and loading it with water (Jackson 1988; Leavell et al. 1989; Oner et al. 1997a,b). The sheet pile was not encased in reinforced concrete at the top as were the I-walls in New Orleans. The purpose of the "E-99 field test" was to compare measured I-wall performance with that calculated using conventional limit equilibrium analyses and soil-structure interaction analyses.

The foundation soils at the test site were soft, highly plastic clays, comparable to the weakest of the lacustrine clays found in the New Orleans area. The instrumentation employed to monitor wall performance included inclinometers, strain gauges, and piezometers.

The test was terminated after about 60 days when a rubber seal between the test wall and one of the side walls broke, and the water spilled out from behind the wall within a few hours. The wall did not fail, either by structural overstress or by foundation instability, although the test report surmised that failure "may have been imminent" at the highest water level (Jackson 1988). No gap between the wall and the soil on the flood side of the wall was observed (the back of the wall and the ground behind the wall was covered with a sheet of plastic), and there is no indication in the test report that a gap might have developed.

After the E-99 test, two papers (Oner et al. 1997a,b) and an additional report (Leavell et al. 1989) were written describing soil-structure interaction analyses of the test. The report by Leavell et al. (1989) and the paper by Oner et al. (1997b) described analytical methods that were capable of modeling a gap between the wall and the soil on the flood side of the wall, should one develop. However, the results of the analyses did not indicate formation of a gap. Thus, neither the field observations made during the test nor subsequent analytical results indicated formation of a gap behind the wall.

The presence of a gap was not assumed in the design of the I-walls in New Orleans. Although media reports suggested that the E-99 test showed that gaps form behind I-walls, this was not the case. Gaps behind I-walls were first observed during the post-Katrina field investigation.

Gaps in Uniform Cohesive Soils

In order to assess rationally the impact of a gap behind a sheet pile floodwall in limit equilibrium slope stability analyses, the potential depth of the gap must be estimated. The depth of the gap, in turn, is related to the ability of the soil on the flood side of the sheet pile to support itself and sustain the gap, and this is determined by the strength of the soil.

When an I-wall is loaded by water and deflects away from the soil on the flood side, the lateral stress in the soil on that side decreases. The lowest possible lateral stress in the soil is the minimum active earth pressure. If the minimum active earth pressure is lower than the water pressure, a gap can form and remain open. If the minimum active earth pressure is higher than the water pressure, a gap cannot remain open. The analysis presented in this paper assumes that the wall will displace or rotate sufficiently such that active earth pressures will be developed.

To illustrate the conditions where a gap forms, consider the homogeneous levee and foundation shown in Fig. 2. The soils in the levee and foundation are assumed to be saturated, and to have the same unit weight and strength. The loads are considered to be

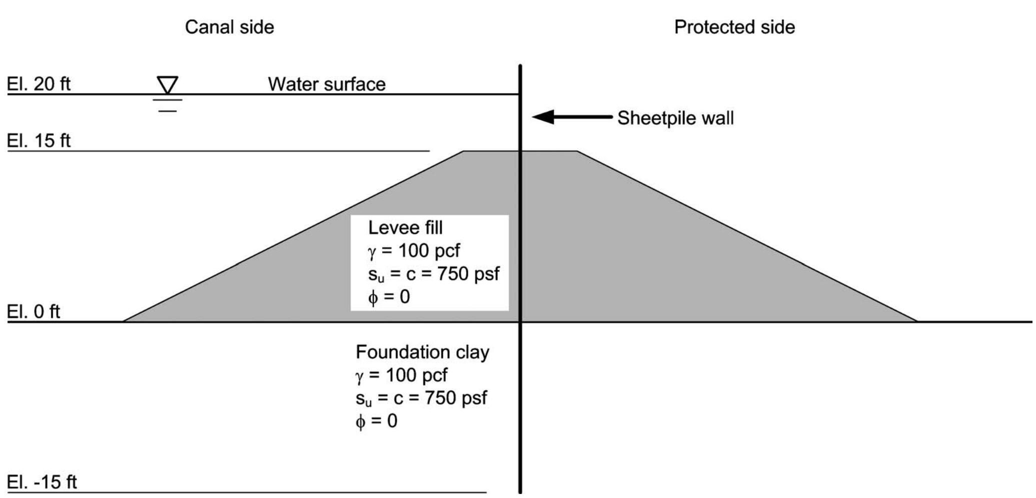

Fig. 2. Example 1—homogeneous levee and foundation

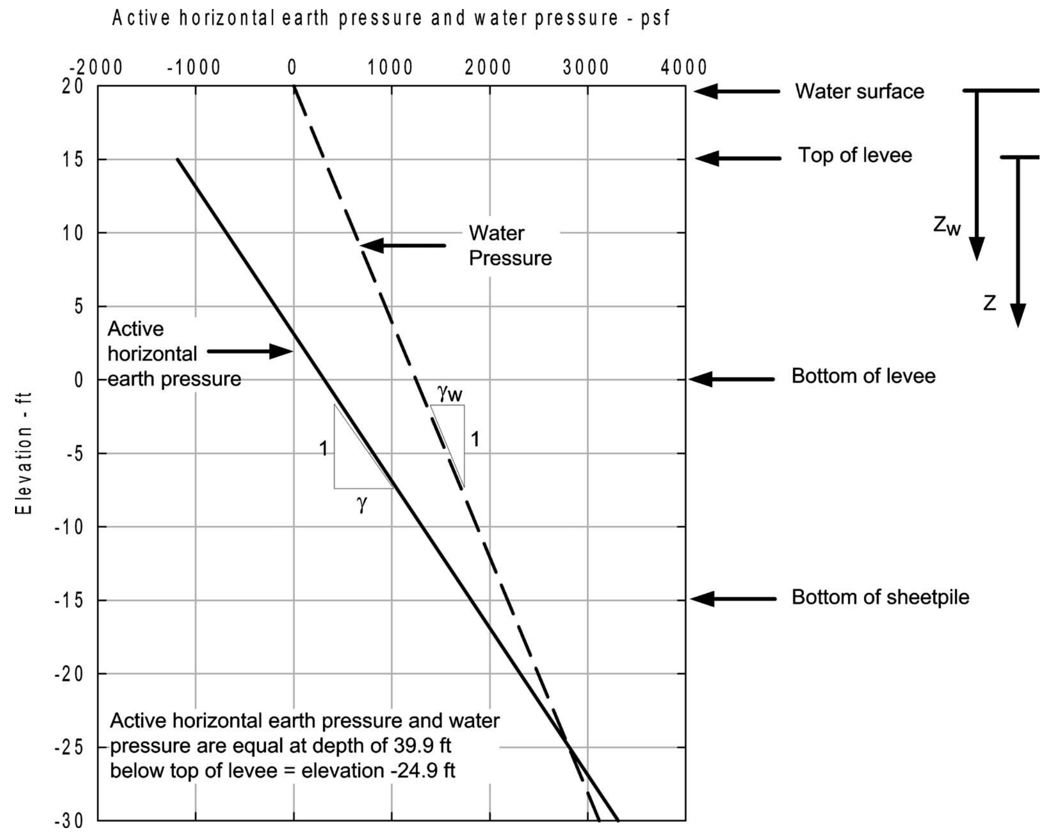

Fig. 3. Pressure distributions for Example 1

imposed quickly so that no drainage occurs. Thus the soil strength is represented by cohesion ($s_u=c$) and the friction angle (ϕ_u) is zero. The total horizontal active Rankine earth pressure and water pressure for this condition are shown in Fig. 3. It can be seen that the hydrostatic pressures exceed the total lateral earth pressures to a depth of 39.9 ft (elevation −24.9 ft). As shown in Fig. 3, this is about 10 ft below the bottom of the sheet pile.

The depth where the earth pressure and water pressure are equal can be calculated as follows:

active earth pressure$=\sigma_{ha}=\sigma_{v0}-2c=\gamma_w(z_w-z)+\gamma z-2c$

water pressure$=\gamma_w z_w$

where these pressures are equal $(\gamma-\gamma_w)z-2c=0$

The depth, z_0, where the earth pressure and water pressure are equal is

$z_0=2c/\gamma-\gamma_w=2c/\gamma_b$

$z_0=(2)(750\text{ psf})/37.6\text{ pcf}=39.9\text{ ft}\approx 40\text{ ft}$ for the conditions shown in Fig. 2

where γ=unit weight of the soil; γ_b=buoyant unit weight of the soil; γ_w=unit weight of water; z=depth below the ground surface; and z_w=depth below the water surface, as shown in Fig. 3.

Although these calculations show that the water pressure exceeds the active horizontal earth pressure for some distance below the bottom of the wall, it would not be expected that the gap would extend below the bottom of the wall since horizontal movement of the wall is required to create the gap. Movement of the wall could reduce the earth pressure acting on it, but it would not be expected that the earth pressures in the soil below the wall would be reduced similarly. As a result, it seems logical that a gap would form and remain open to the bottom of the wall if, as shown in Fig. 3, the calculated value of z_0 is below the bottom of the wall. Because the hydrostatic pressures are greater than the active earth pressures over the full depth of the gap, formation of a gap increases the load on the wall, and makes the wall less stable.

Gaps in Layered Cohesive Soils

When the soil is homogeneous as in the previous example, the depth of the gap can be calculated directly, as shown above. When the soil profile contains soil layers of differing unit weights and shear strengths, the possible depth of a gap can be determined as shown in Figs. 4 and 5.

The active horizontal earth pressure, σ_{ha}, at any depth is calculated using the expression

$$\sigma_{ha}=\gamma_w h_w+\sum(\gamma_i h_i)-2c_i$$

where h_i=height of the ith layer of soil *above* the depth of interest; c_i=undrained shear strength *at* the depth of interest; and h_w=height of water above the ground surface (top of levee).

By plotting this earth pressure along with the hydrostatic water pressure, the depth of the gap can be determined as the depth where the two are equal. For the conditions shown in Fig. 4, the active horizontal earth pressure and the hydrostatic water pressure are equal at elevation −15.6 ft, as shown in Fig. 5. Thus, for Example 2 a gap also would extend to the tip of the sheet pile at a depth of 30 ft (elevation −15 ft).

Modeling Full-Depth Gap in Slope Stability Calculations

When the gap extends fully to the bottom of the wall, the condition can be modeled by removing the soil on the canal side of the

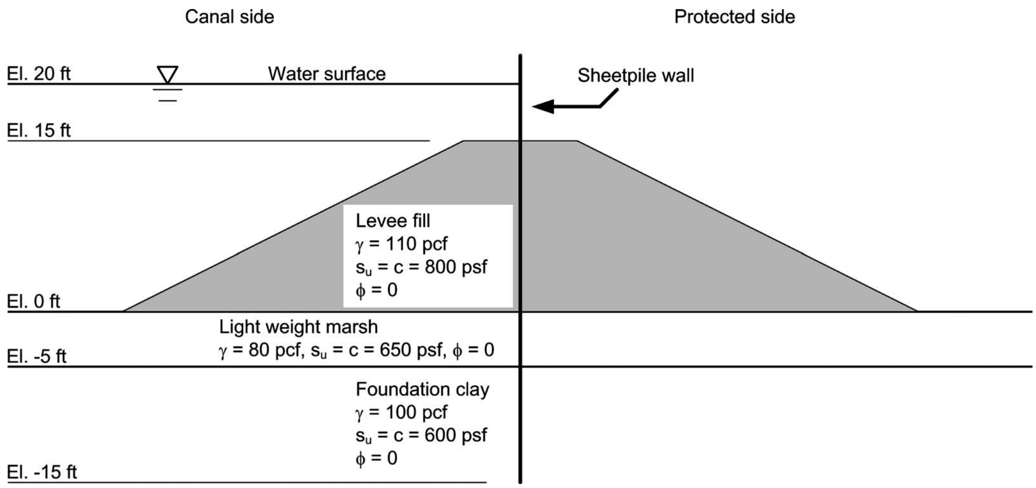

Fig. 4. Example 2—levee on stratified foundation

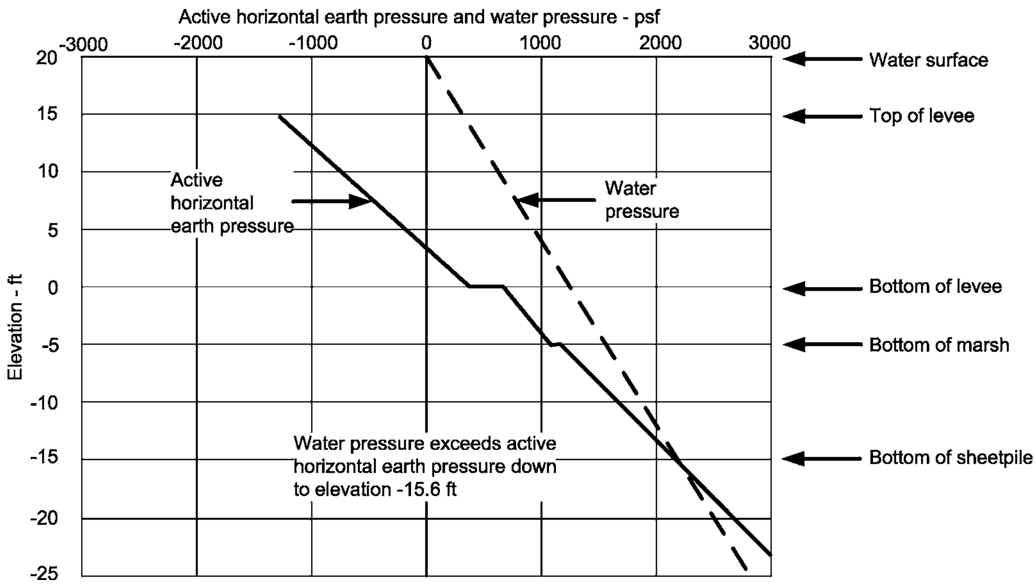

Fig. 5. Pressure distributions for Example 2

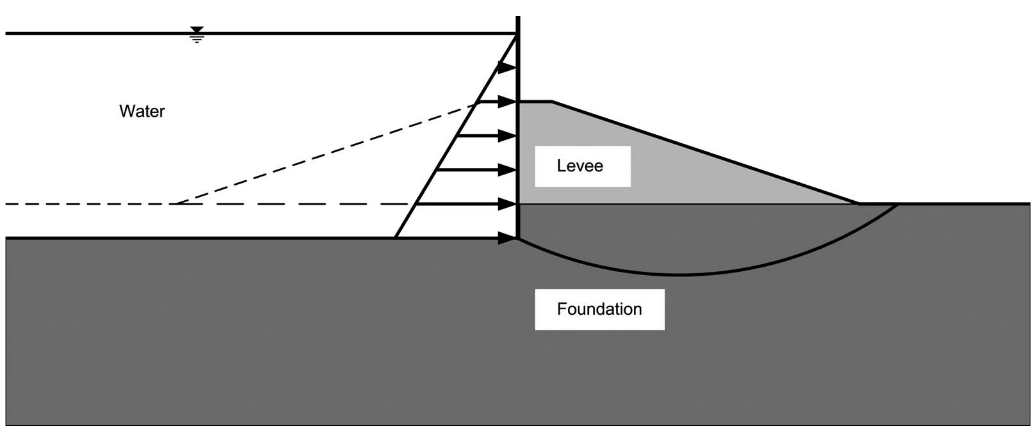

Fig. 6. Modeling of water-filled gap by removing soil on loaded side of gap and applying hydrostatic water pressure to wall

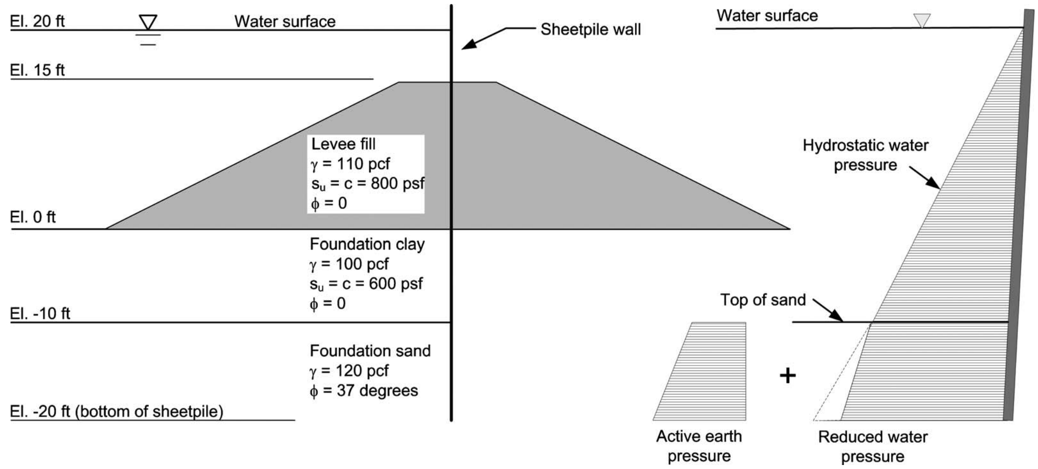

Fig. 7. Example 3—partially penetrating gap

wall, and representing the water load as a distributed pressure, as shown in Fig. 6. Alternatively, the water load can be represented as a concentrated force (the resultant of the triangular pressure distribution in Fig. 6) applied at the lower third point of the pressure diagram.

Some slope stability computer programs allow specification of a vertical "tension" crack, filled with water to some level. To correctly model conditions with a gap, the software should automatically exclude the soil beyond the gap. An appropriate force due to the hydrostatic water pressures in the gap should be calculated by the computer program and applied to the vertical boundary. The most critical failure surface would normally intersect the tip of the sheet pile wall.

In the course of the analyses that were performed by the writers for the New Orleans flood control structures, it was found that three widely used computer programs did not properly model a water-filled crack when the water level in the crack was above the ground surface. Consequently, it is strongly recommended that analyses of this type be performed in more than one way, or with more than one computer program, and that spread sheet calculations be used to check results, in order to ensure that computer programs are modeling the water-filled gap correctly.

Modeling Partially Penetrating Gap in Slope Stability Calculations

In some instances the calculated depth of a gap will not extend to the bottom of the wall. This condition can occur where cohesive soil behind the wall has insufficient strength to keep the gap open. It always occurs when the wall is driven into cohesionless soil, as shown in Fig. 7.

Below the top of the sand layer in Fig. 7, the total horizontal pressure is equal to the water pressure plus the effective horizontal earth pressure, and therefore cannot be less than the water pressure. The water pressure and total horizontal earth pressure for this case are shown in Fig. 8.

When a gap forms down to the top of a sand layer, water can

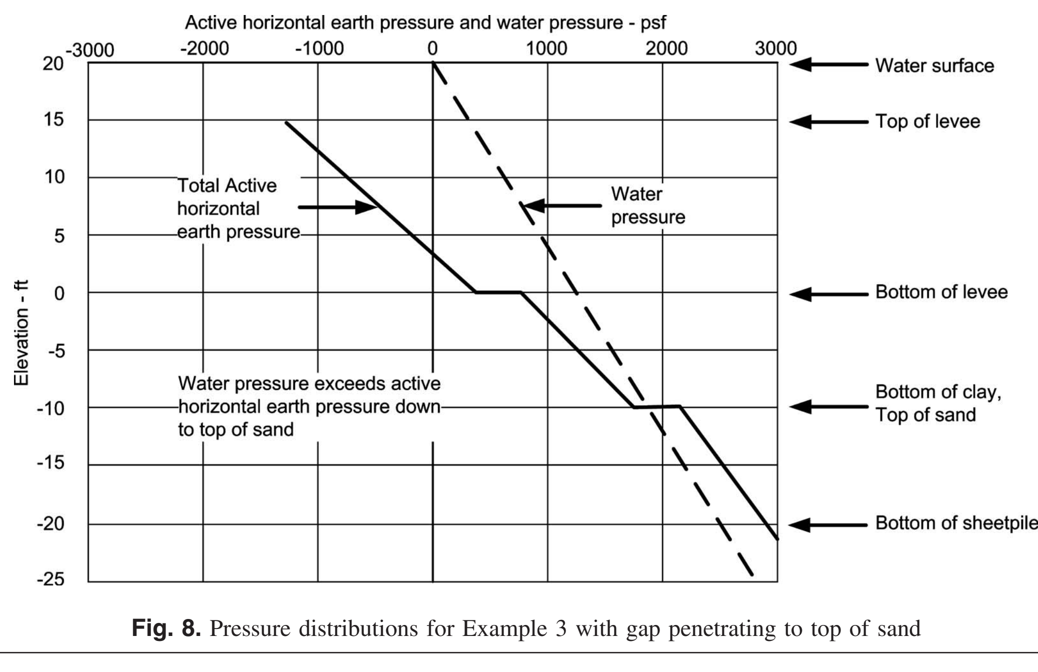

Fig. 8. Pressure distributions for Example 3 with gap penetrating to top of sand

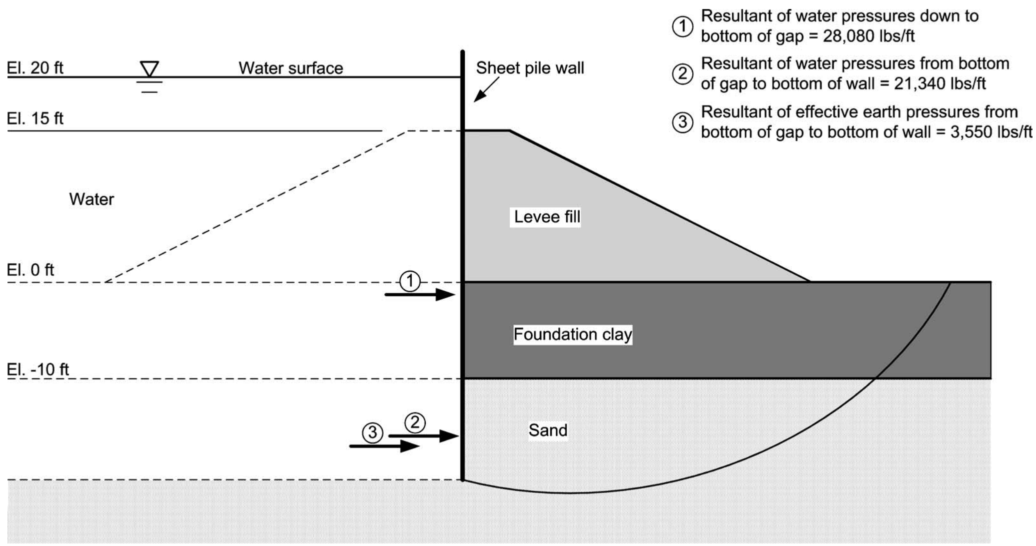

Fig. 9. Modeling of water-filled gap by removing soil on loaded side of gap and applying concentrated forces to represent water pressures and effective earth pressures

flow down through the gap and into the sand. In this condition the water pressures within the sand will not be hydrostatic unless the head in the sand happens to be the same as the head in the gap. In the case shown in Fig. 7, it has been assumed that the head in the sand decreases below the bottom of the gap. Hence, water flows into the sand, and the water pressures below the top of the sand are smaller than hydrostatic, as shown in Fig. 7. In calculating the pressures shown in Fig. 8, it was assumed that the water pressure at the bottom of the sheet pile is 100 psf less than hydrostatic pressure at that depth. In other words, 1.6 ft of head loss occurs from the top of the sand layer to the tip of the sheet pile. For the analyses of such conditions in New Orleans, water pressures within the sand were determined by performing steady-state seepage analyses using the finite-element method.

When a gap extends only part way down the wall, the condition can be modeled by removing all of the soil on the canal side of the wall, and representing the loads on the wall as follows:

1. Down to the bottom of the gap, the water load can be represented as a distributed pressure, or as a concentrated force equal to the resultant of the pressure diagram, as for a fully penetrating gap. This is labeled as Load 1 in Fig. 9; and
2. Below the bottom of the gap, the load on the wall can be represented as a distributed pressure, or as a concentrated force equal to the resultant of the pressure diagram. In the case where the wall is driven into sand, and seepage occurs, the load on the wall within the sand can be modeled in two parts, the water pressure (Load 2 in Fig. 9), and the effective earth pressure (Load 3 in Fig. 9).

Analyses of New Orleans I-Walls

To illustrate the effect of a gap on computed factors of safety, analyses are presented for two of the New Orleans I-walls that failed as a result of Hurricane Katrina—the 17th Street Canal I-wall and the London Avenue Canal I-wall. Duncan et al. (2008) describe both of these failures.

17th Street Canal I-Wall failure

A cross section of the 17th Street Canal at station 10+00, in the center of the section that failed, is shown in Fig. 10(a). The topography at this location was determined by a pre-Katrina light detection and ranging (LIDAR) survey conducted in May, 2001. The elevations of the crest and tip of the I-wall are based on "as-built" drawings made after construction of the I-wall, and augmented by forensic studies of the failed sections. The stratigraphy is based on pre-Katrina and post-Katrina borings, and cone penetration tests.

The elevations shown in Fig. 10(a) are based on the North American vertical datum of 1988 (NAVD88). A significant effort was made to sort out the different data used in engineering projects in the New Orleans area. Details of this effort can be found in Volume II of the IPET report (IPET 2007).

The shear strength parameters and unit weights used in the slope stability analyses are listed in Table 1. Different shear strengths were used for the marsh layer beneath the levee and at the toe of the levee. The shear strength was assumed to vary linearly from the crest to the toe, and to be constant beyond the toe toward the protected side.

The undrained shear strength of the lacustrine clay was calculated based on an undrained strength ratio $s_u/\sigma_v'=0.24$, and calculated effective stresses with the canal water elevation at 0.0 ft NAVD88. Additional details regarding shear strength assessment for the 17th Street Canal analysis can be found in Appendix 3 of Volume V of the IPET report (IPET 2007) and Duncan et al. (2008).

At the 17th Street Canal I-wall section, the sheet pile extended through the levee fill and marsh deposits into a lacustrine clay deposit. Calculations of the active Rankine earth pressure indicate that hydrostatic pressures would exceed the horizontal active total earth pressure to a depth below the bottom of the sheet pile. Accordingly, for analyses with a gap behind the wall, the gap was assumed to extend from the top of the levee to the bottom of the wall. This condition was modeled by removing the soil on the canal side of the wall, and applying hydrostatic pressure to the

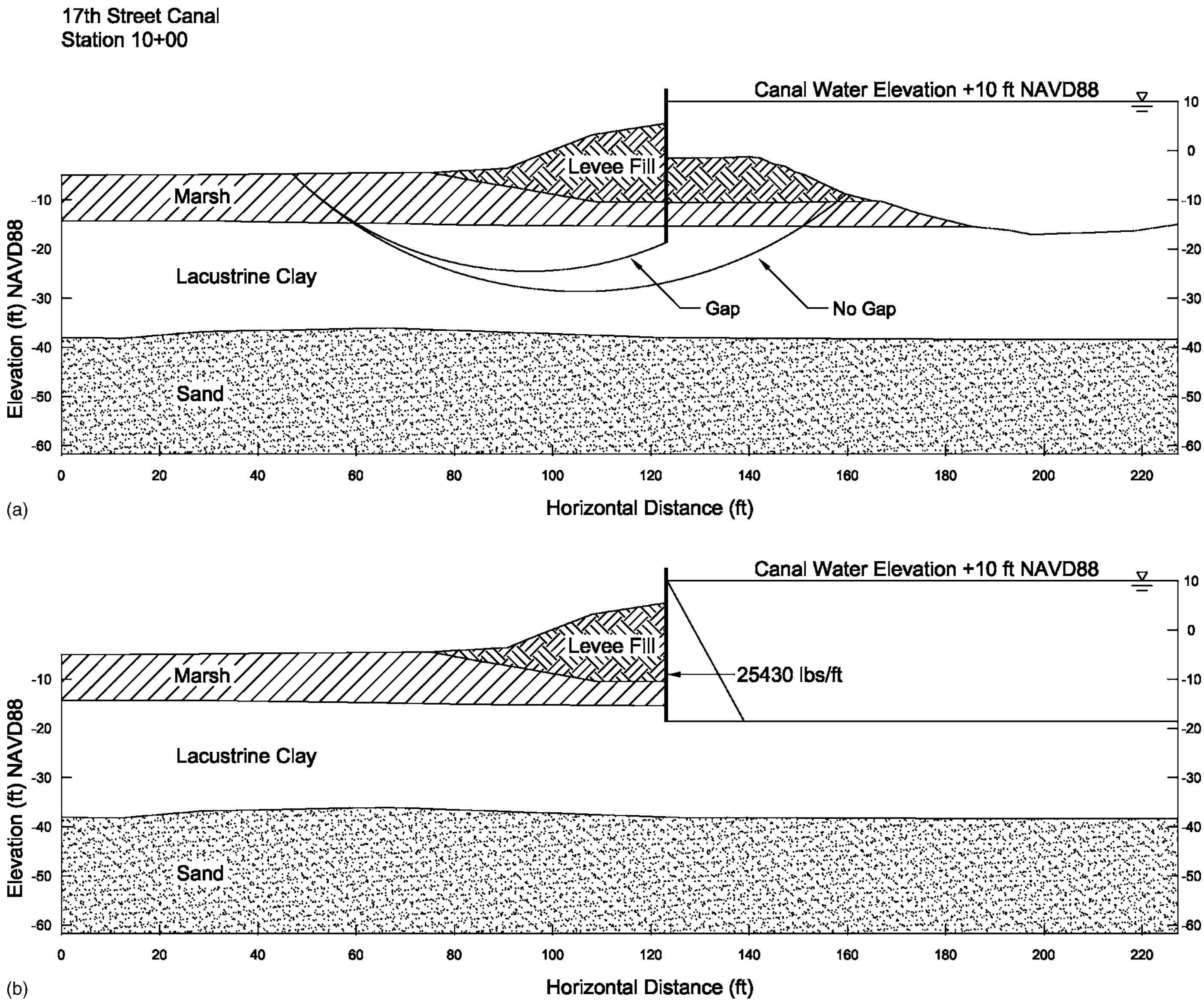

Fig. 10. (a) Cross section through 17th Street Canal I-wall in New Orleans showing critical circles for gap and no gap cases; (b) hydrostatic force required to model gap to bottom of sheet pile

Table 1. Shear Strengths and Unit Weights Used for Analysis of Station 10+00 for 17th Street Canal Failure

Material	Shear strength	Unit weight γ (pcf)
Levee fill	$s_u=900$ psf, $\phi_u=0$	109
Marsh (under levee crest)	$s_u=400$ psf, $\phi_u=0$	80
Marsh (beneath and beyond levee toe)	$s_u=300$ psf, $\phi_u=0$	80
Varied linearly between levee crest and toe		
Lacustrine clay	s_u calculated based on $s_u/\sigma'_v=0.24$ $\phi_u=0$	109
Relic beach sand	$c'=0$, $\phi'=35°$	120

wall from elevation +10.0 ft NAVD88 to the bottom of the wall. Fig. 10(b) shows the cross section with the canal-side soil removed, and a force (representing the hydrostatic water pressure) of 25,430 lb/ft applied at elevation −9 ft NAVD88 has been added.

Slope stability computations were performed with Spencer's method using UTEXAS4 (Wright 1999) and SLIDE (Rocscience 2005). The same factors of safety were calculated using both programs. Without a gap behind the wall, the calculated factor of safety was 1.28. With a gap, the calculated factor of safety was 0.98. Fig. 10(a) shows the critical circles for the analyses performed with and without the gap. These results show that the gap has a very significant effect on the computed factor of safety, and that formation of a gap played a critical role in the failure.

Additional analyses were performed using noncircular slip surfaces with UTEXAS4. The critical failure surfaces for the noncircular analyses were very similar in shape and position to the

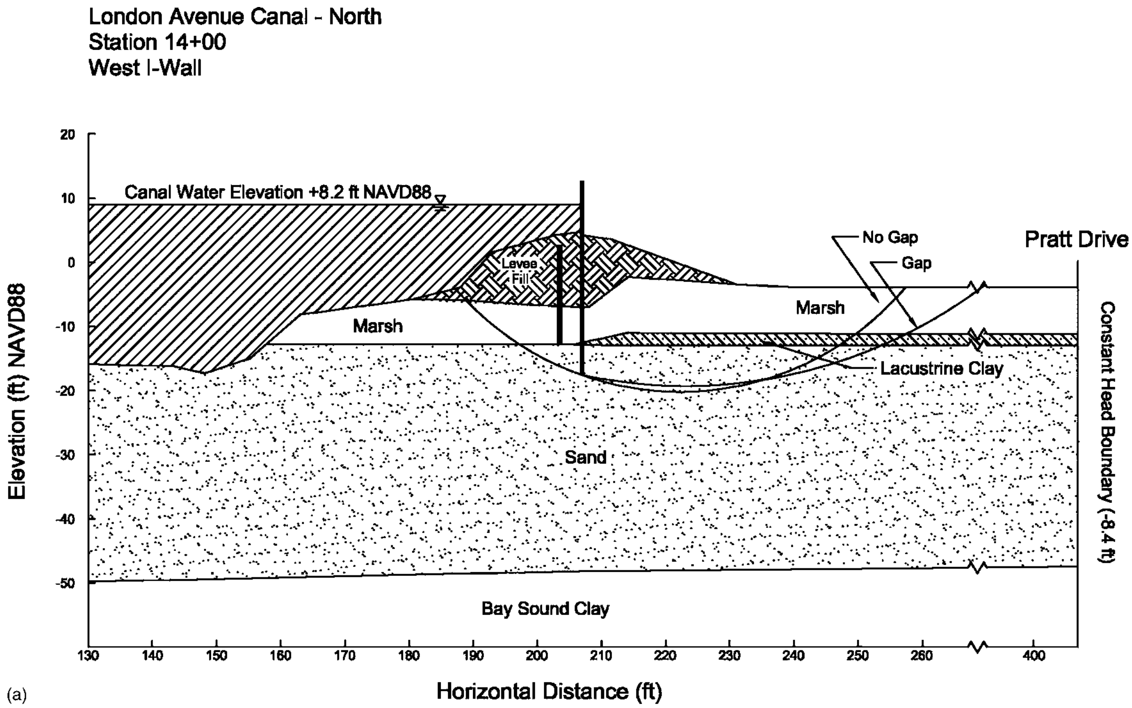

Fig. 11. (a) Cross section through London Avenue Canal I-wall in New Orleans showing critical circles for gap and no gap cases; (b) water pressure and earth pressure forces required to model gap to top of sand layer

critical circles. Generally, the factors of safety for the noncircular surfaces were about 6% lower than those determined using circular failure surfaces.

London Avenue Canal-West Bank Failure

A cross section through the reach of the London Avenue Canal I-wall where the northern breach occurred is shown in Fig. 11(a). It can be seen that there was a thin layer of lacustrine clay beneath the levee on the landward side of the wall. On the canal side of the wall, the levee was underlain by marsh. The clay and marsh layers were underlain by a thick sand stratum, which was in turn underlain by a layer of stiff clay at great depth. The shear strength parameters and unit weights used in the stability analyses are listed in Table 2. Additional details regarding soil conditions at the London Avenue Canal can be found in Appendix 8 of Volume V of the IPET report (IPET 2007).

Calculation of water pressures and horizontal active pressures showed that a gap would form to the top of the sand layer with water in the canal at elevation +8.2 ft NAVD88 as shown in Fig. 11(a). Hydrostatic water pressures were applied to the portion of the wall above the sand. Water pressures within the sand were computed from steady-state seepage analyses, and forces representing these water pressures and the effective earth pressures were applied to the portion of the wall below the top of the sand. Fig. 11(b) shows the forces that must be applied to model the gap for the conditions shown. A force of 14,830 lb/ft is applied at

Table 2. Shear Strength and Unit Weights Used for Analysis of North London Avenue Canal Failure (West Bank)

Material	Shear strength	Unit weight γ (pcf)
Levee fill	s_u=900 psf, ϕ_u=0	109
Marsh	s_u=300 psf, ϕ_u=0	80
Relic beach sand	c'=0, ϕ'=32°	115

Note: The same strength was used throughout the marsh layer.

elevation −6.3 ft NAVD88 to represent the water pressures acting on the wall above the top of the sand layer. A second water force of 4,710 lb/ft is applied at elevation −16.1 ft NAVD88 to represent the water pressure acting on the portion of the sheet pile embedded in the sand. In addition, an active effective earth pressure force of 1,245 lb/ft is applied at elevation −16.1 ft NAVD88.

Slope stability computations were performed with Spencer's method using UTEXAS4 and SLIDE, as for the 17th Street Canal I-wall. The factors of safety calculated using UTEXAS4 and SLIDE were the same. Without a gap behind the wall, the calculated factor of safety was 1.52. With a gap, the calculated factor of safety was 0.99. The critical failure circles for these cases are shown on Fig. 11(a). As was the case for the 17th Street Canal I-wall, it can be seen that the gap has a very significant effect on the computed factor of safety, and that formation of a gap played a critical role in the failure. At London Avenue the gap also had an important effect on the seepage conditions, by providing a path for flow directly into the sand beneath the levee.

Discussion of Partially Penetrating Gaps

It might be argued that a gap that extends only part way to the slip surface is entirely internal to the slide mass and should not be represented as a loaded external boundary using the procedure illustrated in Fig. 9. However, if the gap is omitted and the slip surface extends behind the wall, and includes the soil in back of the wall, the computed factor of safety for the London Avenue Canal I-wall increases to 1.52, and does not indicate an unstable condition, even though failure actually occurred.

Having studied these cases in detail, the writers have concluded that it is appropriate to model a partial gap as shown in Fig. 9, and that this is necessary in order to represent correctly a failure mechanism that is realistic and more critical than a continuous slip surface extending behind the wall.

Summary and Conclusions

The formation of a gap between a sheet pile I-wall and the floodside embankment due to water loading was an important factor in the performance of these flood control structures during Hurricane Katrina. The depth of the gap that can be sustained can be calculated based on the total horizontal active earth pressure and the hydrostatic pressures that act on the sheet pile. For cases where the gap does not extend to the bottom of the wall, the wall below the bottom of the gap is loaded by total active earth pressures where the soil below the bottom of the gap is cohesive, and by effective active earth pressures and water pressures where the soil below the bottom of the gap is cohesionless. Correct modeling of the effects of a gap on stability analyses should use a critical slip surface that terminates at the tip of the wall, as illustrated in Figs. 9–11.

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