New Orleans and Hurricane Katrina.
IV: Orleans East Bank (Metro) Protected Basin

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Abstract: This paper addresses damage caused by Hurricane Katrina to the main Orleans East Bank protected basin. This basin represented the heart of New Orleans, and contained the main downtown area, the historic French Quarter, the Garden District, and the sprawling Lakefront and Canal Districts. Nearly half of the loss of life during this hurricane, and a similar fraction of the overall damages, occurred in this heavily populated basin. There are a number of important geotechnical lessons, as well as geo-forensic lessons, associated with the flooding of this basin. These include the difficulties associated with the creation and operation of regional-scale flood protection systems requiring federal and local cooperation and funding over prolonged periods of time. There are also a number of engineering and policy lessons regarding (1) the accuracy and reliability of current analytical methods; (2) the shortcomings and potential dangers involved in decisions that reduced short-term capital outlays in exchange for increased risk of potential system failures; (3) the difficulties associated with integrating local issues with a flood risk reduction project; and (4) the need to design and maintain levees as systems; with each of the many individual project elements being required to mesh seamlessly. These lessons are of interest and importance for similar flood protection systems throughout numerous other regions of the United States and the world.

CE Database subject headings: Louisiana; Hurricanes; River basins; Floods.

Introduction

This paper is the fourth of a series of companion papers presenting the principal results of an investigation of the performance of the New Orleans regional flood protection systems during and after Hurricane Katrina, which struck the New Orleans region on August 29, 2005 (ILIT 2006; Seed et al., private communication, 2008). This paper addresses the performance of the regional flood protection systems surrounding the main Orleans East Bank protected basin during Hurricane Katrina.

The Orleans East Bank (downtown) basin is one contiguously protected section containing the downtown district, the French Quarter, the Garden District, and the northern Lakefront and "canal" Districts (Fig. 1). The northern edge of this protected area is fronted by Lake Pontchartrain, and the Mississippi River passes along its southern edge. The Inner Harbor Navigation Channel (IHNC) passes along the east flank of this protected section, separating the Orleans East Bank protected basin from New Orleans East (to the northeast) and from the Lower Ninth Ward and St. Bernard Parish (directly to the east). Three large drainage canals

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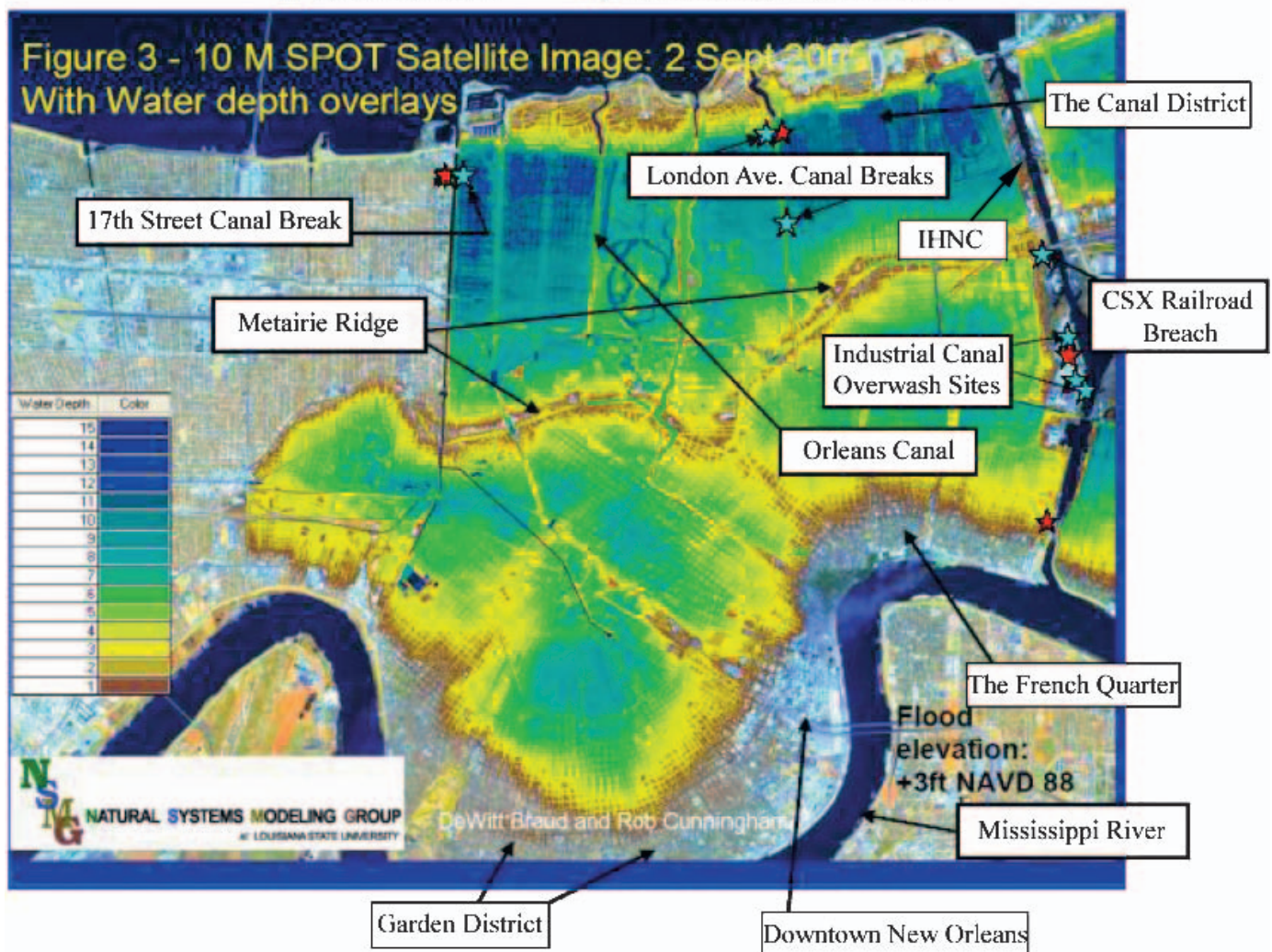
Fig. 1. (Color) Satellite view of the main (Metro) Orleans East Bank protected basin, showing depths of maximum flooding, and the locations of full levee breaches and distressed levee sections (adapted from Van Heerden 2006)

extend from the interior of Orleans East Bank to the north to Lake Pontchartrain for the purpose of conveying water pumped northwards from the basin into the lake by large dewatering pump stations within the city. These three canals, from west to east, are the 17th Street canal, the Orleans Canal, and the London Avenue Canal. The western edge of the Orleans East Bank protected basin is defined by the 17th Street canal, and also by the southwest end of the elevated ridge of Metairie Ridge. To the immediate west of the Orleans East Bank is Jefferson Parish, another heavily populated community with a population of approximately 400,000.

A majority of the flood flow into the Orleans East Bank basin came through the three large breaches that occurred on the drainage canals at the northern end of the Orleans East Bank protected area (ILIT 2006; Van Heerden et al. 2006). The remainder of the floodwaters in this basin came from (1) heavy rainfall associated with the hurricane; (2) brief overtopping of some of the levees and floodwalls on the east flank (on the west bank of the IHNC); and (3) a series of relatively minor breaches that occurred at the eastern edge of the basin, on the west bank of the IHNC. Four breaches occurred at the eastern edge of the protected basin, on

the west bank of the IHNC (Fig. 1). None of these four breaches eroded a pathway to a depth below sea level. Thus, floodwaters initially passed in through these breaches, but that inflow eventually stopped as the storm surge subsided.

One catastrophic breach occurred on the 17th Street drainage canal, and two catastrophic breaches occurred on the London Avenue drainage canal (Fig. 1). All three of these failures eroded to depths below mean sea level, and they continued to admit flow into the city from Lake Pontchartrain well after the initial storm surge had subsided, eventually equilibrating with the still slightly swollen waters of Lake Pontchartrain at an elevation of approximately +3 ft (MSL) three days later on the afternoon of Thursday, September 1. The inflow from these three large breaches on the drainage canals thus produced a majority of the floodwaters that eventually filled more than 80% of the main (Metro) Orleans East Bank protected basin; the basin within which more than half of the overall loss of life, and a similar fraction of the overall damages, occurred during this devastating hurricane and flood.

This paper presents the results of geo-forensic studies and analyses of the performance of the perimeter flood defenses

Fig. 2. View of the I-wall failure at the Port of New Orleans, from the inboard side, showing the trench scoured behind the I-wall by overtopping flows

Fig. 3. Southernmost breach along the west bank of the IHNC, showing the lightweight shell-sand fill of the remaining intact levee embankment section adjacent to the eroded breach

around this main Orleans East Bank protected basin. This includes studies of both successes and failures (especially critical incipient failures), as both serve to highlight important lessons.

West Bank of the IHNC

The eastern edge of the Orleans East Bank protected basin is defined by the IHNC, a constructed waterway serving to provide access to the main Port of New Orleans. The third phase of Hurricane Katrina's progression across the region involved the raising of the waters of Lake Borgne and then driving these storm surge elevated waters westwards through the east–west trending shared channel of the Gulf Intracoastal Waterway/Mississippi River Gulf outlet (GIWW/MRGO) waterway, which meets the north–south trending IHNC waterway at a "T" intersection at the east edge of the Orleans East Bank protected basin (Seed et al. 2008b). This resulted in a major storm surge in the IHNC, and produced numerous failures along both the east and west banks of the IHNC channel. Four full failures (breaches) occurred along the west bank of the IHNC, and two additional sections suffered major distress (Fig. 1).

The Northernmost failure and breach along this frontage was the west bank IHNC crossing of the CSX rail line; a complex "penetration" at which several sets of individual interests including a highway, a roadway operated by the Port of New Orleans, and the rail line crossed through the federal levee. This failure [discussed in detail by Seed et al. (2008b)] was primarily the result of the complex interactions between these different entities and project elements, the lack of overall coordination, and the apparent lack of a single responsible party with overall responsibility and sufficient authority as to affect a safe and reliable overall solution on behalf of all parties. It was also associated with the presence of highly erodible lightweight shell-sand fill at this breach location.

The next failure to the south was the failure of a sheetpile-supported I-wall section atop a relatively low-rise earthen levee embankment section (Fig. 2), and a full description is presented in ILIT (2006). This failure was the result of overtopping of the concrete floodwall by the elevated waters of the IHNC, which resulted in erosion of a trench behind the wall. This served to reduce the lateral bracing of the concrete I-wall and its supporting sheetpile curtain, and the lateral pressures of the elevated canal

waters then pushed the floodwall sideways, opening a water-filled gap between the sheetpile curtain and the outboard side of the earthen levee embankment. The gap allowed water pressures to act against both the I-wall and its supporting sheetpile curtain, and these lateral water pressures then pushed the I-wall (and its supporting sheetpiles) to the inboard side, rupturing the concrete I-wall and producing the breach as shown in the center of the photograph.

The breach itself represented the separation of two adjacent concrete I-wall panels, and then flow through this opening further widened the feature (but only to a width of approximately 40 ft). It is interesting to note that this overtopping failure produced only a localized failure, and that it did not scour a path to below sea level, so that although flow passed through this breach for several hours, this inflow eventually ceased as the storm surge within the IHNC channel subsequently subsided.

Two additional failures (and breaches) occurred farther to the south, along the west bank of the IHNC. Both of these breaches occurred at locations where the earthen embankment sections were comprised primarily of highly erodeable lightweight shell-sand fills. Fig. 3 shows a close-up view of the southernmost of these two breaches. In Fig. 3 the highly erodeable shell-sand fill can be seen at the rear of the photo (i.e., the striated exposed face of the end of the intact embankment at the rear of the breach in this photo). The use of this same shell-sand fill for the partially completed interim breach repair (in the foreground of the photo) can also be seen (i.e., the material upon which the person is standing). In addition, kicked up by passing foot traffic, some of the lightweight small (fine gravel-sized) mollusk shells that comprise a large fraction of this material are visible atop the asphalt pavement section.

A large number of the failures that developed during this hurricane occurred at locations where highly erodible cohesionless sands and lightweight shell sands had been used to construct portions of the levee embankments (ILIT 2006; Seed et al. 2008a,b). The paper by Briaud et al. (2008) deals specifically with the erodibility of many of the various embankment and foundation soils encountered during our investigation's studies. The use of highly erodible materials represents a major potential hazard for levee and flood protection systems, and that such use is inadvisable for systems protecting large urban populations. As part of the ongoing efforts to reevaluate and upgrade the New Orleans regional flood protection systems, a systematic effort should be made to

identify highly erodible materials within the levee system and correct this deficiency.

Drainage Canals

Dewatering the city of New Orleans is a constant concern. Because much of it is below mean sea level, rain that falls in the city must be pumped out, along with inflows from perimeter underseepage. Heavy rains routinely cause localized minor flooding, and the pumps are also heavily taxed by increased underseepage when unusually high-water flows come down the Mississippi River. The three drainage canals at the north end of the main (Metro) Orleans East Bank protected basin serve to carry water pumped from the basin out to Lake Pontchartrain. They also represent three potentially vulnerable elements of the overall perimeter defenses of this heavily populated basin.

In many regions, flood protection functions are the responsibility of a single local agency. In New Orleans, however, two sets of agencies shared portions of the overall flood protection responsibility: (1) local levee boards had primary local responsibility for the perimeter defenses (e.g., levees and floodwalls); and (2) a second set of local agencies, the local water boards, had responsibility for pumping to dewater the city. The principal routine concern of the water board has historically been handling heavy rainfall and river flows. Conversely, infrequent hurricanes are a principal concern of the levee boards. Recognizing this, the U.S. Army Corps of Engineers (USACE) had argued for more than 17 years, from 1970 to 1986, for permission to install floodgates at the north ends of these canals. These gates would be closed in the event of hurricane-induced storm surges from Lake Pontchartrain so that the storm surges would not be transmitted into these canals (U.S. Senate Committee on Homeland Security and Governmental Affairs 2006; Wooley and Shabman 2007). The installation of floodgates would have been useful in preventing hurricane-induced flooding, but the gates themselves would have been "perimeter defenses" and so would have been operated by the local levee board. The local water board was concerned that the gates would impede pumping to dewater the city during heavy storms and rainfall, especially as the gates would be closed during storm surges. A solution could have been to install the floodgates at the north ends of the canals, and to also install new sets of pumps to pump water out over (or through) the gates/levees even when the gates are closed, but that would have been expensive. However, the USACE was denied authorization to install the floodgates at the north ends of the canals, and it was decided instead to raise the heights of protection along these three canals (Wooley and Shabman 2007). After the devastation wrought by Katrina, the USACE has now been allowed to install both the floodgates at the heads of the three canals, and the extra pumps needed to dewater the city while the gates are closed.

Having been charged to raise the levels of protection along the three drainage canals, the USACE now faced a difficult task. Homes had encroached closely upon the inboard side toes of the earthen levee embankments along these canals (especially along the 17th Street and London Avenue canals), leaving little or no space for widening of the earthen levees. At some locations, the backyard property lines of the encroaching homes extended up the levee slopes to the edge of the narrow levee crest. Crest widths were also narrow, providing less than typical access for heavy vehicles for emergency operations as per typical USACE standards (USACE 2000; ILIT 2006; Wooley and Shabman 2007). Property access issues at the inboard side levee toes also hampered inspection and maintenance; this contributed to the profusion of uncontrolled growth (including large trees) at this critical inboard side levee toe region, and also failure to fully resolve reports of underseepage flows during high water events (ILIT 2006). Because of the high costs involved, it was decided not to purchase the additional land necessary to widen the levees, providing additional levee width and mass to support higher embankment crests, and the USACE ended up raising the canal levee crests by adding concrete floodwalls atop the earthen levee sections. This represented a challenging technical solution. It also represented another of many situations in which a decision had been made, by Congress and local institutional partners, to accept a higher level of risk of eventual system failure in exchange for short-term savings during initial flood protection system construction (ILIT 2006; U.S. Senate Committee on Homeland Security and Governmental Affairs 2006). As with a number of other such decisions at other locations, this would prove costly.

Recognizing the difficulties associated with the raising of these floodwalls on the crests of the existing, relatively narrow earthen levees, the local New Orleans District of the USACE performed a test of key components of this type of floodwall design in the nearby Atchafalaya River basin. That project, referred to as the E99 test section, involved the construction of a sheet pile/floodwall (I-wall) section inboard of an existing levee along the Atchafalaya River atop foundation soils that were similar to those present along portions of the 17th Street canal. A sheet pile cofferdam was constructed to allow progressively higher water levels to laterally load the E99 sheet pile/floodwall section. The test section's sheet pile wall was brought to a condition estimated to represent an incipient failure. During this field test, the sheet pile wall appeared to rotate as an essentially rigid unit. This rotation should have opened a gap between the sheet piles and the adjacent compacted earth, but the E-99 test was stopped short of a full stability failure, and there is no record of a gap being observed during testing. Because the wall and the soil in front of the wall were covered with a plastic membrane, which was in turn filled with water, it would have been difficult to observe a gap opening even if it did. Several follow-up analytical studies were performed to evaluate the E-99 test section (e.g., Jackson 1988; Oner et al. 1988; Leavall et al. 1989; Oner et al. 1997a,b), but it appears that the possibility of a water-filled gap forming on the canal side of the sheet pile/floodwall was not considered in the design analyses of these I-wall sections for the three Orleans East Bank drainage canals (USACE 1988, 1989, 1990).

When Hurricane Katrina arrived, the storm slightly inflated the level of Lake Pontchartrain, and then as the eye of the storm passed to the northeast of New Orleans, the counterclockwise swirling winds pressed the waters of Lake Pontchartrain southward, creating a storm surge against the shoreline of the Lake at the north edge of the Orleans East bank protected basin. This surge rose rapidly in the early hours of August 28, 2005, and eventually reached their peak along this shoreline at approximately 9:30 a.m. The peak storm surge at this shoreline was approximately to Elev. +11 ft (MSL) at the mouth of the 17th Street drainage canal, and to approximately +11.5 to +12 ft (MSL) at the mouth of the London Avenue drainage canal just a few miles farther to the east.

Orleans Canal

The only one of the three drainage canals that did not breach during this final phase of Hurricane Katrina was the central one, the Orleans Canal. There were three principal differences between

Fig. 4. Side view of the "gap" in the flood defenses at the south end of the Orleans canal

the Orleans canal and the other two adjacent canals (the 17th Street canal and the London Avenue canal), both of which did suffer catastrophic breaches. The first of these was the availability of a bit more land (a bit more "footprint") along most of the inboard side levee toes on both sides of the Orleans Canal, allowing more generous provision of levee embankment mass to support the floodwalls at the levee crests. A second difference was the use of SHANSEP-type soil strength interpretations for many of the design analyses (Ladd and Foott 1974), which avoided the problems associated with overestimation of shear strengths in cohesive soils beneath the levee toes based on strength testing of samples obtained mainly from beneath the centers of the levees (as occurred along much of both the 17th Street canal and the London Avenue canal). This is discussed in more detail, with regard to the 17th Street canal failure, in the preceding companion paper (Seed et al. 2008c).

A third difference was the fact that the flood protection system around the perimeter of the Orleans Canal was incomplete at the time of Katrina's arrival. A gap had been left in the system, at the south end of the canal (see Fig. 4). At the south end of the canal, on the east bank, the final 200 ft of concrete floodwall were never installed. Instead, as an interim measure, a concrete spillway section (visible in the left foreground of Fig. 4) was installed so that water flowing through this gap would not cause catastrophic erosion. This gap in the defenses provided an opening with a crest height approximately 6 ft lower than the adjacent levee/floodwall sections, so that at the peak of the hurricane-induced storm floodwaters simply flowed freely through this gap into the heart of the main (Metro) New Orleans protected basin.

The reason for the existence of this gap in the defenses occurs just to the left of the photo in Fig. 4 (just out of the photo), where an old brick building (constructed in 1903) crosses the south end of the canal as a "T," forming the southern boundary of the lined canal. That building houses several large Woods pumps, used by the Water Board to pump waters out of New Orleans into the drainage canal for transit north into Lake Pontchartrain. The ability of that old brick building to hold back high water is questionable; at canal water levels of more than about +5 ft (MSL) water seeps through the brick wall of the building and enters the structure. It is apparent that higher water levels against the front of this structure could potentially imperil the building. To complete the perimeter defenses for the Orleans Canal, either (1) the Water Board needed to construct a protective frontage wall to both pro-

(a)

(b)

Fig. 5. (a) Oblique aerial view of the breach section near the south end of the London Avenue drainage canal, on the east bank. The interim repair has been largely completed, but the arrows indicate the location of the original breach location; (b) Aerial view of initial breach closure at the London Avenue South Breach, showing the narrow breach width and the I-wall panels at each end toppling inwards toward the scoured breach opening. (IPET 2007).

tect their own building, and to complete the perimeter defenses (which are the responsibility of the local levee board); or (2) the local levee board needed to construct the protective frontage wall, to both complete their perimeter defenses and to protect the Water Board's pump building. Disputes arose between the Water Board and the local levee board as to whose responsibility it was to design and construct the necessary protective wall (and canal closure), and the situation had not yet been rectified at the time of Katrina's arrival (ILIT 2006).

London Avenue Canal

South Breach

The first of the three large breaches on the drainage canals occurred on the east bank of the London Avenue canal, approximately midway between the north and south ends of the canal (Fig. 1). This breach, referred to as the "South Breach," occurred well to the south of the other large breach that occurred on the west bank of the canal nearer to the north end. This breach was the first of the two breaches to occur, at approximately 7:00–8:00
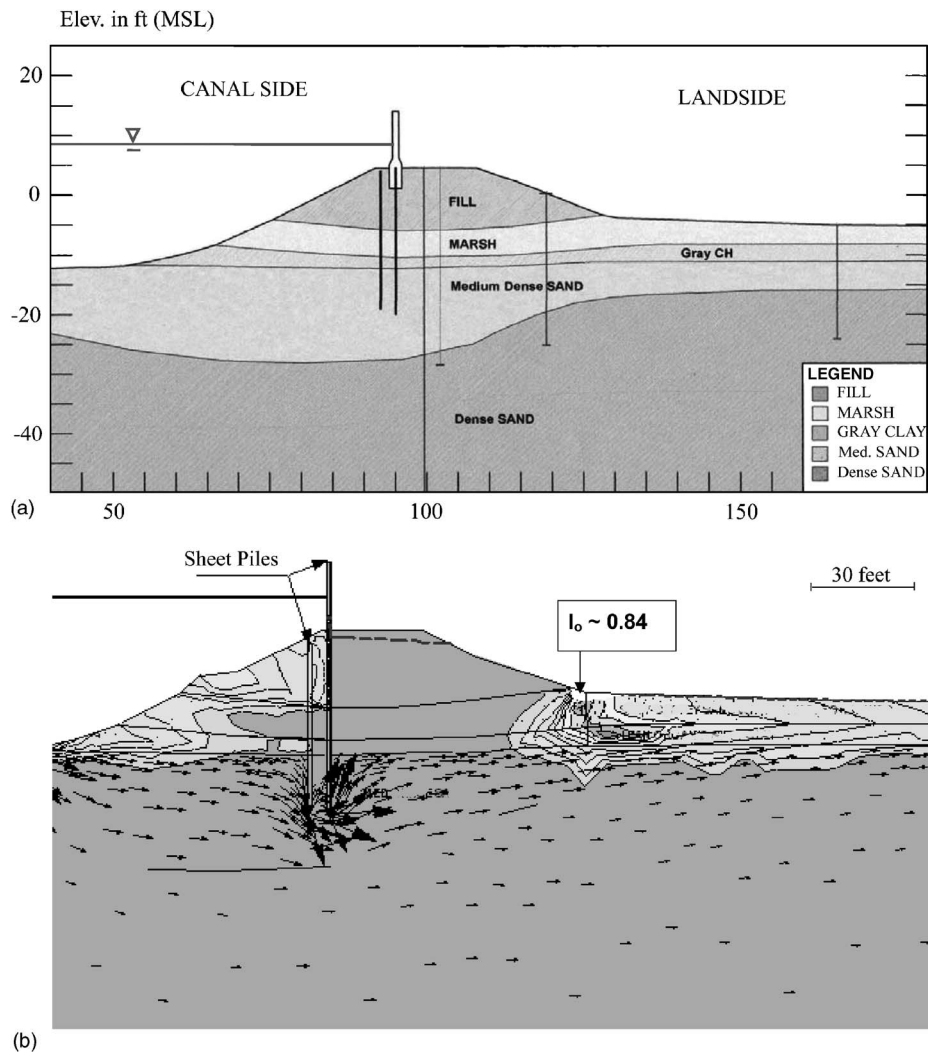
Fig. 6. (a) Cross section through the South Breach, on the east bank near of the London Avenue drainage canal, near the south end; (b) Hydraulic gradients (and flow vectors) for storm surge at +9 ft (MSL) at the South Breach on the east bank of the London Avenue drainage canal (maximum exit gradient, $i_0 \approx 0.84$)

a.m., at a canal water level of approximately +8.5 to +9.5 ft, MSL (ILIT 2006; Van Heerden et al. 2006; IPET 2007, Vol. V).

Fig. 5(a) shows an oblique aerial view of this breach, which occurred on the east bank of the canal just to the north of the Mirabeau Bridge (the bridge at the right of this photo). At the time of this photo, the interim repair embankment section was largely in place, and so it obscures the details of the original failure. Accordingly, Fig. 5(a) has been annotated by the addition of a pair of dashed lines, and arrows, indicating the location of the now obscured deep, narrow scoured opening that appears to have been the initial failure location. Fig. 5(b) shows a second view of this feature, during early stages of emergency breach repairs. In this photo, the narrow nature of the original breach can be seen. The breach was initially only three concrete I-wall panels in width (a width of less than 60 ft), and in this photo (despite the floodwaters) it can be seen that the concrete I-wall panels at both ends of the narrow breach are toppling into a deep, trench-like feature.

Fig. 6(a) shows a geotechnical cross section through the breach section. The upper portion of the earthen levee embankment was comprised of moderately compacted cohesive fill, underlain by uncompacted clay fills placed over the latter portion of

the preceding century. The embankment section was underlain by a relatively thin layer of marsh/swamp deposits, consisting of variably interbedded organic, peaty soils, and soft paludal clays. This marsh stratum was, in turn, underlain by a layer of paludal/lacustrine clay (CH) that was only one to several feet in thickness. As described and illustrated in the companion paper by Rogers et al. (2008), a "buried sand ridge" of cohesionless, sandy soils runs beneath the south end of the Orleans canal, and beneath essentially the full length of the London Avenue canal as well. This buried sand ridge represents an old depositional channel of the Mississippi River. As a result, the shallow surficial veneer of low permeability marsh and paludal clay deposits at this site is underlain at relatively shallow depth by substantially deeper deposits of significantly more permeable sands. These are relatively loose to medium dense near to the surface, and they increase in density with depth as shown in Fig. 6(a). The floodwall at this location was a conventional sheet pile-supported concrete I-wall, and a second (previous) sheet pile curtain was in place just on the outboard side of the sheet piles supporting the I-wall.

Despite the relatively low strength (and stiffness) of the upper marsh and paludal clay strata, lateral embankment stability was not problematic at this site. With the canal water level at

Table 1. Summary of Key Properties Used in Transient Seepage and Limit Equilibrium Stability Analyses

Stratum name	PLAXIS soil model	Shearing type[a]	γ_{unsat} (lb/ft^3)	γ_{sat} (lb/ft^3)	K_h (cm/s)	K_v (cm/s)	OCR	Shear strength[b]	
Embankment fill	Mohr Coulomb	Undrained	N/A	90	10^{-6}	10^{-7}	N/A	$c=800$ lb/ft^2	$\emptyset=0$
Marsh	Mohr Coulomb	Undrained	N/A	80	10^{-4}	10^{-5}	Varies[a]	$c=300-600$ lb/ft^2	$\emptyset=0$
Upper paludal clay	Mohr Coulomb	Undrained	N/A	90	10^{-6}	10^{-7}	Varies[a]	$c=200-400$ lb/ft^2	$\emptyset=0$
Upper sand/clay interface	Mohr Coulomb	Drained	N/A	100	N/A	N/A	N/A	$c=0$	$\emptyset'=31°$
Loose sand	Mohr Coulomb	Drained	N/A	100	10^{-3}	10^{-3}	N/A	$c=0$	$\emptyset'=33°$
Medium dense sand	Mohr Coulomb	Drained	N/A	110	10^{-3}	10^{-3}	N/A	$c=0$	$\emptyset'=36°$
Dense sand	Mohr Coulomb	Drained	N/A	115	10^{-3}	10^{-3}	N/A	$c=0$	$\emptyset'=40°$

[a]Overconsolidation ratio varies laterally and vertically as a function of embankment-induced overburden stress, and desiccation-induced overconsolidation of the stratum prior to the emplacement of the embankment.

[b]Shear strength within the marsh stratum and the upper paludal clays varies as a function of overburden and OCR.

Elev. +9.5 ft (MSL) (which was the maximum height of storm surge rise in the canal during Hurricane Katrina), the lowest factor of safety with regard to either lateral translational failure, or semirotational failure, of the inboard side of the earthen levee embankment (if pushed laterally by the formation of a "water-filled crack" on the outboard side of the sheetpiles) was greater than 1.5 (ILIT 2006). Lateral embankment instability is therefore not considered to have been the cause of this failure.

Instead, this failure appears to have been the result of underseepage and piping erosion. Fig. 6(b) shows flow vectors and exit gradients calculated using the finite-element program SEEP/W (Krahn 2004) to perform transient seepage analyses modeling the full time-dependent rise of canal water levels over the 24 h that preceded the failure. Table 1 shows the principal parameters used to model the various key strata in these analyses (ILIT 2006). Underseepage flow passed through the pervious foundation sands, and it also passed beneath the relatively shallowly embedded sheetpile curtain which did not extend deeply enough as to represent an effective underseepage "cut-off."

This underseepage produced high exit gradients at the inboard side levee toe. Fig. 6(b) shows calculated seepage gradients calculated for a time at about 7:15 a.m., with the canal water level at Elev. +9 ft (MSL). The calculated factor of safety against initiation of seepage erosion and piping at this inboard toe region was less than one for the relatively lightweight organic soils and clays present (i.e., FS$=i_c/i_0$, where $i_c=$critical gradient, which is the buoyant unit weight of the soil divided by the unit weight of water, and $i_0=$vertical exit gradient). In addition, the buildup of pore pressures trapped beneath the thin veneer of relatively impervious soils overlying the sands at the inboard side levee toe resulted in a potentially unstable condition with regard to hydraulic uplift or "blowout" at this same toe region. At this site, failure would have been expected to occur due to underseepage-induced piping erosion, likely exacerbated by initial uplift (or blowout) at the inboard levee toe.

Field evidence at this site is consistent with these findings. The deep, narrow erosional failure had all the characteristics of a classic underseepage and piping failure in which underseepage initiated at the inboard toe region (likely accelerated by initial uplift or blowout), and then tunneled progressively back beneath the levee crest, producing a deep, narrow trench-like feature into

which the concrete I-wall sections would topple. This was one of only two major breaches during Hurricane Katrina that appeared to be the result of underseepage-induced piping (and possible blowout). The other such feature was the North Breach at the east bank of the IHNC, at the west end of the Lower Ninth Ward. As discussed in Seed et al. (2008b), that feature was also a deep, narrow breach feature. Both of these piping-induced breaches were less than 90 ft in width, even after scouring inflows through the open breaches. The other major breaches on the three drainage canals, and at the west end of the Lower Ninth Ward, were judged by these studies to have been the result of lateral/semirotational levee embankment instability failures, and each of those stability-related breaches were more than 400 ft in width. This mode of underseepage-induced piping was also judged by the USACE's IPET investigation to represent the most likely mode of failure at this South Breach (IPET 2006).

It should also be noted that significant vegetation, including mature trees, had been allowed to grow along the inboard side toes of the levees along this canal as shown in Figs. 5 and 7.

Fig. 7. Oblique aerial view of the breached (North Breach) section on the west bank of the London Avenue Canal (on the right side of the photo, which is taken looking to the south), and the distressed section on the east bank (on the left in this photo)

Property rights, associated with property lines that often encroached the inboard side levee faces, of adjacent homes significantly complicated both maintenance and inspections along these frontages. There were even a few swimming pools in this critical region at the inboard side levee toes along the three drainage canals. Although there is no clear evidence that trees actually caused this failure, and the results from these analyses indicate that the failure would have likely occurred with or without vegetation, it is not prudent to allow significant vegetation to grow within the critical inboard side toe region.

North Breach (and Distressed Section)

The second breach that occurred along the drainage canals occurred near to the north end of the London Avenue drainage canal, on the west bank (Fig. 1). Fig. 7 is an oblique aerial view of this site, looking toward the south. The failure section is on the right of this photograph (on the west bank), just to the south of the Robert E. Lee Bridge. If one looks closely, it can also be noted in this photograph that the concrete I-wall on the opposite bank of the canal (on the east bank, on the left side in this photograph) has been laterally displaced between 1 and 3 ft, and is leaning slightly away from the canal toward the inboard (land) side.

This pair of sites posed one of the most challenging locations for geotechnical analyses and geo-forensic investigation, as the conditions on the two opposite sides of the canal were not very different; yet one side failed (and breached), whereas the other side experienced the beginnings of movements (indicative of an incipient or near-failure), but did not fail. Accordingly, this pair of sites presents a particularly fine test of the accuracy and reliability of currently available geotechnical analysis methods.

Fig. 8(a) shows conditions at the outboard side of the concrete I-wall on the west bank, at the breach location. In this photograph, it is clear that a "water-filled gap" has opened between the sheetpiles/I-wall and the outboard side of the levee embankment, peeling back the I-wall but leaving the top of the outboard side (canal side) of the earthen levee embankment in place (at the right of this photo).

Fig. 8(b) shows conditions at the outboard side of the concrete I-wall on the east bank (at the distressed section). In Fig. 8(b), it can be clearly seen that a water-filled gap has opened between the slightly laterally displaced I-wall (and its supporting sheetpile curtain) and the outboard side of the earthen levee embankment. The maximum width of this gap was measured to be 2.6 ft at the location of maximum movement. On the landside of the floodwall on the distressed east bank, several sand boils were found along the toe of the levee. In addition, several sinkholes were found on the levee crown immediately landward of the floodwall. These signs of seepage distress were situated in the area where the floodwall had moved the most.

Fig. 9 shows a pair of cross sections through the west bank breach section, illustrating conditions before and after the failure occurred. The failure at this site was a translational failure of the inboard half of the earthen levee embankment, pushed laterally by elevated canal water pressures acting against both the upper concrete I-wall, and also against the lower sheetpile curtain (from within a water-filled gap that opened on the outboard side of these sheetpiles, as shown in Fig. 8). The shear slippage within the foundation soils was apparently fomented by reductions in the strengths of these soils due to underseepage-induced pore pressures that passed beneath the inadequately deep sheetpile curtain. Removal of some of the foundation sands by underseepage piping could also have played a role, as suggested by the evidence of seepage distress found on the opposite side of the canal.

(a)

(b)

Fig. 8. (a) View of the west bank breach near the north end of the London Avenue canal, taken from the south end, showing the top of the outboard side of the earthen embankment section still in place (at the right of the photo); (b) water-filled gap at the outboard side of east bank distressed section; London Avenue Canal

Fig. 10 presents a cross section showing the similar soil conditions at the east bank distressed section. As displacements were small, both the "before" and "after" displacements are shown in Fig. 10 for clarity. Conditions at this east bank section represent an incipient failure (or distress condition), but the movements were arrested when the west bank section failed first (and drew down the canal water elevation to a safer level). The levee embankments consisted of moderately compacted clay fills, and were underlain by local marsh deposits consisting of variably interbedded peaty materials and clays. These marsh deposits were underlain by a relatively thin stratum of gray, paludal clays (CH). As was discussed in the previous section, a prominent "buried sand ridge" underlies these sites at shallow depths, so that the relatively thin veneer of low permeability marsh and clay deposits is underlain by significant thicknesses of more pervious sandy soils on both sides of the canal. Although the marsh deposits and the paludal clays are both relatively weak, these strata do not appear by themselves to have been the cause of the failure (and near-failure) that were observed. Calculated minimum factors of safety for slippage along shear surfaces passing mainly through these marsh and clay strata were significantly higher than for
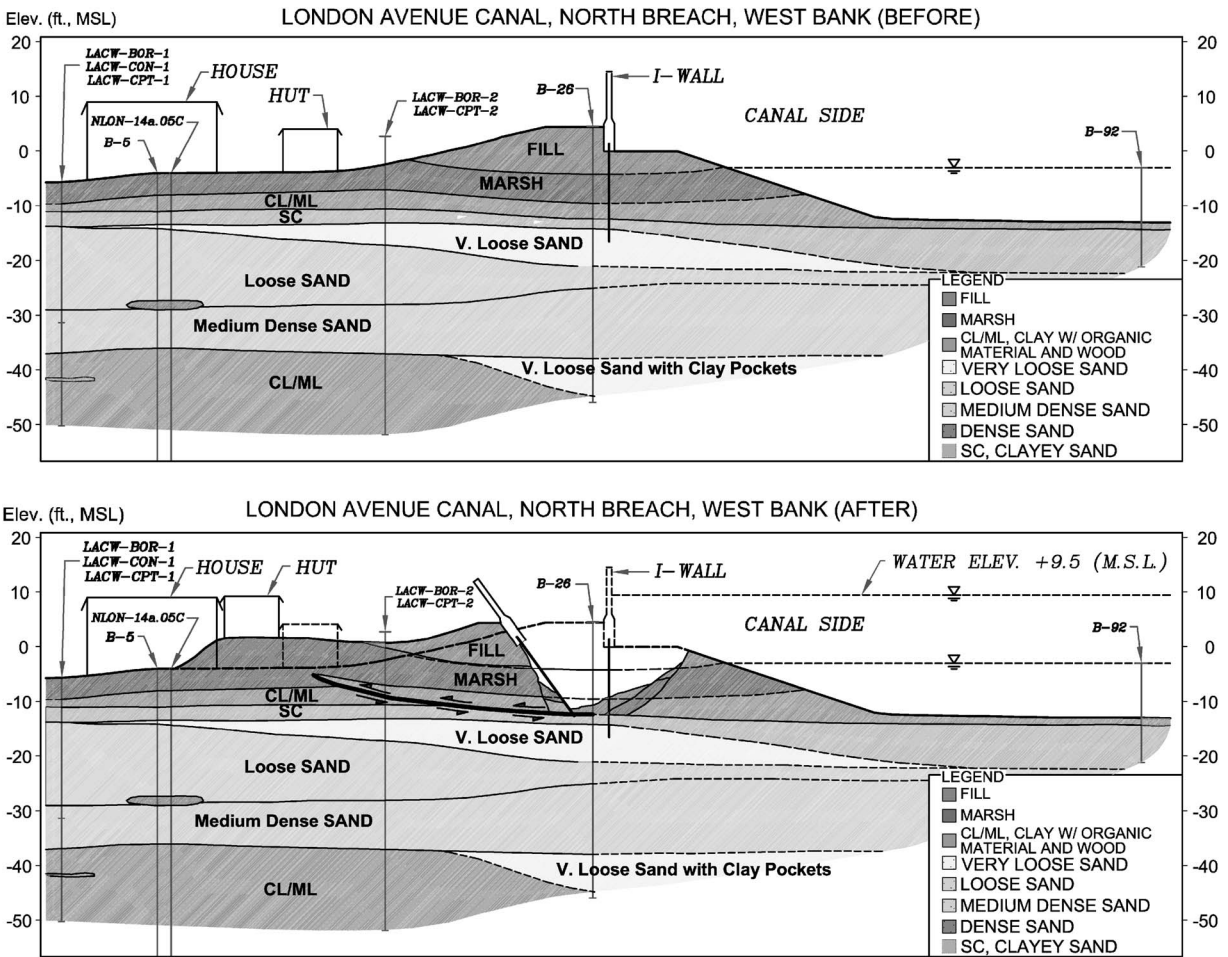
Fig. 9. Cross section of conditions before and after the failure at the North Breach on the West Bank of the London Avenue drainage canal

shear surfaces at or near to the interface between the upper sands and the base of these overlying, less pervious materials (ILIT 2006).

This transitional interface between the upper sands and the overlying clayey soils is of principal interest. This was a transitional material change, grading progressively (from bottom to top) from relatively clean, fine sands (SP) progressively through increasingly silty and clayey sands (SM and SC), before reaching the base of the stratum of soft gray paludal clays (CH). This transitional change in materials occurred over a distance of several vertical feet. The materials graded progressively less pervious at increasing elevation, and they also graded toward progressively lower effective friction angles over this same increase in elevation.
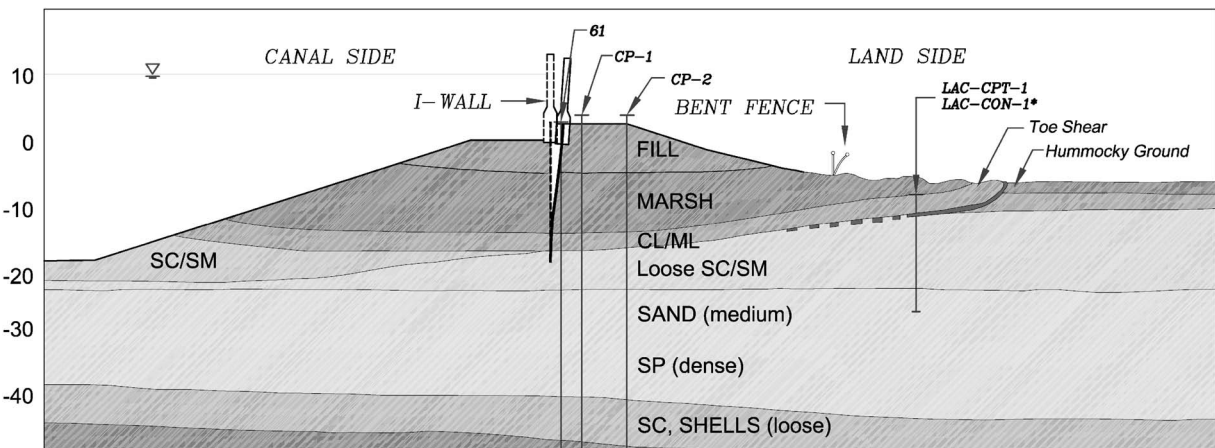
Fig. 10. Cross section at the "distressed" levee and floodwall section on the east bank of the London Avenue drainage canal, which shows partial wall deflection and minor lateral slippage at the inboard side levee embankment toe
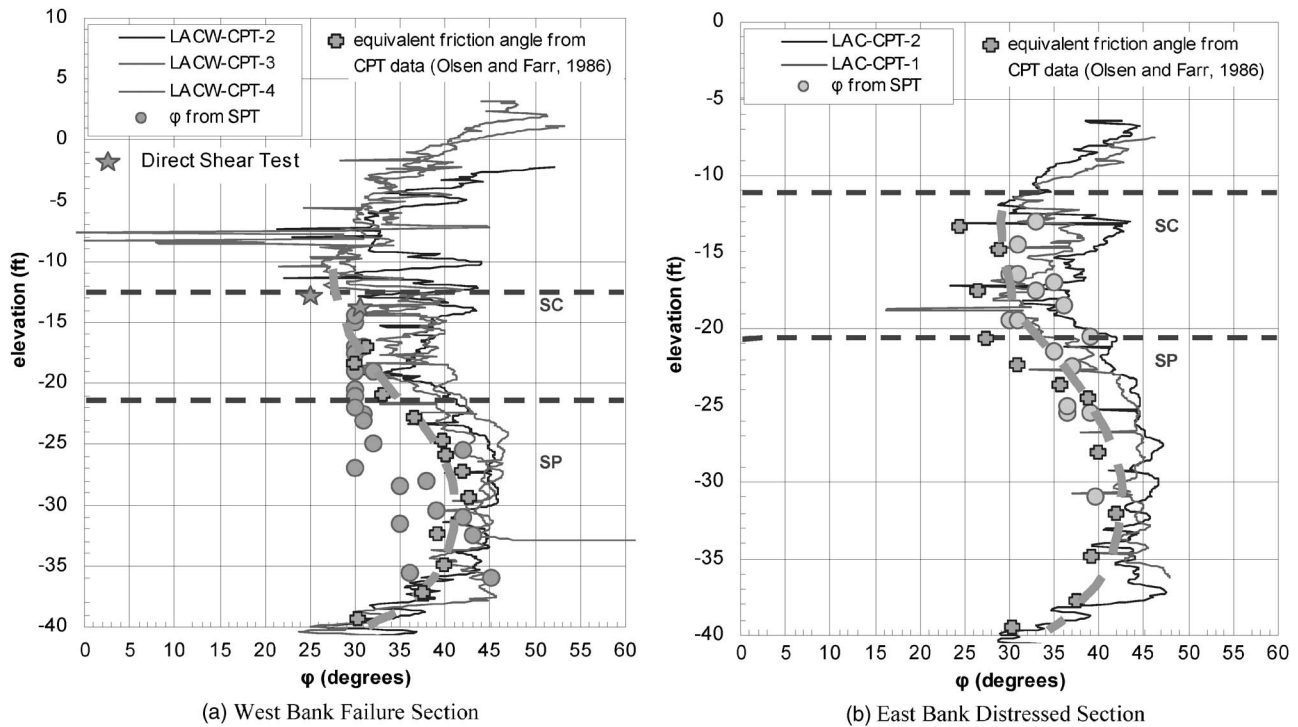
Fig. 11. Estimation of friction angle for cohesionless materials for the London Avenue North Breach section on the west bank and for the distressed section on the east bank: (a) West Bank Failure Section; (b) East Bank Distressed Section

Underseepage-induced pore water pressures passed relatively easily through the sands and beneath the inadequately deep sheet-pile curtain, and then built up beneath the inboard side (landside) "cap" of less pervious surficial deposits (the clays and marsh deposits). It was principally these high pore pressures at the base of the relatively impervious upper soil veneer, on the inboard side, that led to the reductions in soil shear strengths that eventually produced the failure (and near-failure) at these two sites.

Figs. 11(a and b) show the characterization of effective friction angles in the materials across the progressive interface (or transition) between the upper sands, the silty and clayey sands, and the overlying paludal clays at both the west bank breach site, and the east bank distressed section, respectively. Multiple approaches were used to assess the effective friction angles. The thin, solid traces in these figures are cone penetrated test (CPT)-based interpretations based on Robertson and Campanella (1983), the large solid "crosses" are CPT-based interpretations based on Olsen and Farr (1986), and the closed circles are SPT-based estimates based on Seed (2005). Finally, the large, closed stars in Fig. 11(a) represent the results of a pair of laboratory direct shear tests on samples obtained by means of a modified 2.8 in. diameter, thin-walled, fixed piston sampler that was advanced as described in Seed et al. (2008c).

It was not possible to discern the elevation and degree to which pore pressures rose within the relatively thin but critical transitional sand zone due to underseepage. Hence, various scenarios of partial pore pressure development may be postulated at different elevations across this vertical transition, and these may be paired with various effective friction angles to evaluate the shear strength within this narrow and critical transitional zone. Higher (more completely penetrating) pore pressures approaching steady state flow are appropriate at the base of this transition zone, and these would be paired with effective friction angles on the order of $\varnothing' \approx 30$–$32°$. A few inches higher in the transition

zone the effective friction angle would be somewhat lower, but this would be offset by reduced penetration of pore pressures, resulting in largely similar estimates of resultant frictional shear strength. An effective friction angle of 31° was selected, and this was coupled with assumed rapid development of essentially full steady state pore pressures as the storm surge rose. For reasonable ranges of in situ permeability of the deeper, more open and pervious sands and with reasonable ranges of specific storage for these initially saturated deposits; pore pressure development at the inboard side toe region at the top of the pervious "clean" sands was approximately 80–90% developed within 2 h of corresponding outboard side (canal side) water level increases (ILIT 2006).

Figs. 12(a and b) show the results of finite-element analyses of the west bank (breach) and east bank (distressed) section, respectively. The best-estimate properties within this critical transition zone used in both analyses are provided in Table 2. Fig. 12(a) shows an incipient translational failure begins to occur, by sliding along the transitional zone within the foundation soils, at a canal water elevation of +9 ft (MSL). Fig. 12(b) shows that the incipient failure is only just beginning to occur at a slightly higher canal water level of Elev. +10 ft (MSL). Essentially identical results were obtained using conventional coupled transient seepage/limit equilibrium analysis methods. In these analyses, transient seepage analyses were first performed using the program SEEP/W, and at each stage (each point in time, and thus each stage of progressively increasing canal water elevation), the calculated fields of pore pressure were then imported into the limit equilibrium slope stability analysis program SLOPE/W and overall stability analyses were then performed using Spencer's method (Spencer 1967).

Shear strengths modeled in these analyses, including within the critical transition zone ($\varnothing' = 31°$), are presented in Table 1. These are based jointly on data for the strata at these two sites
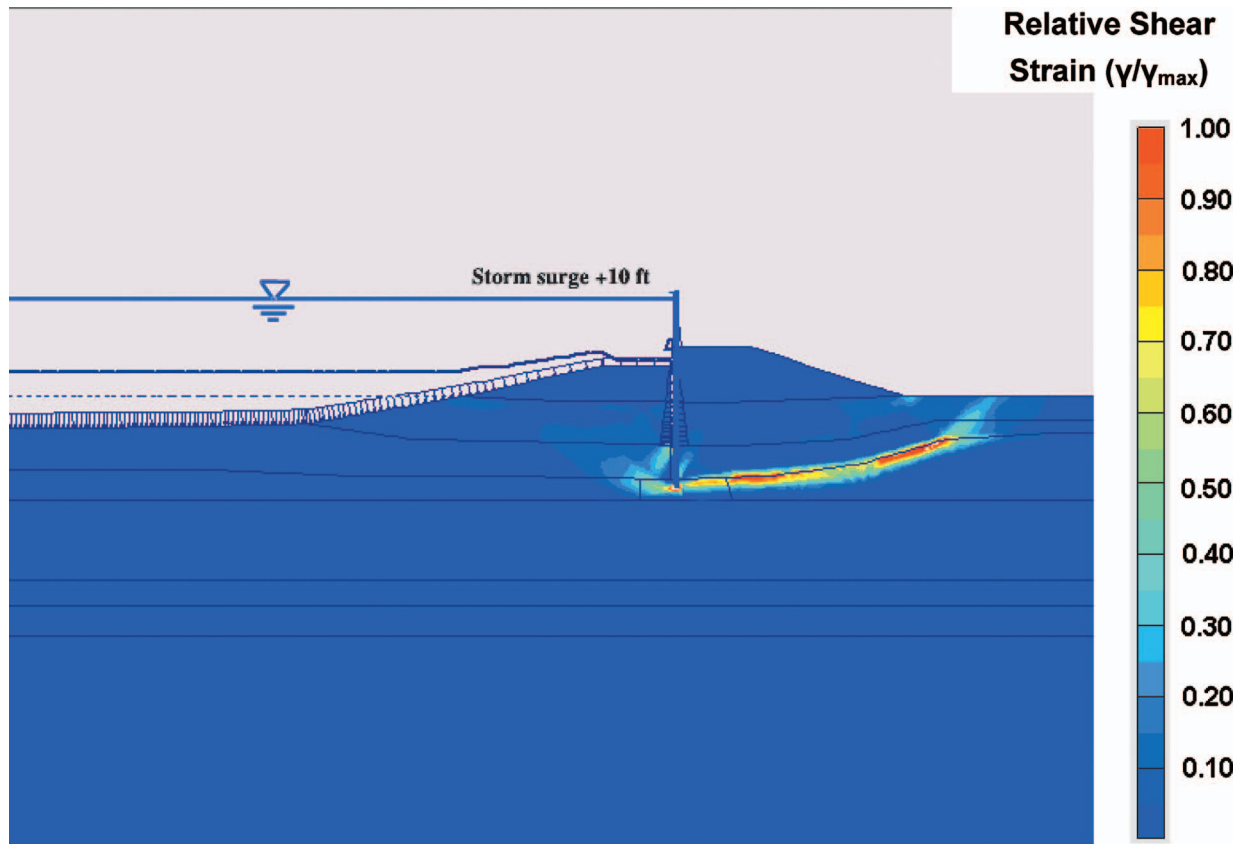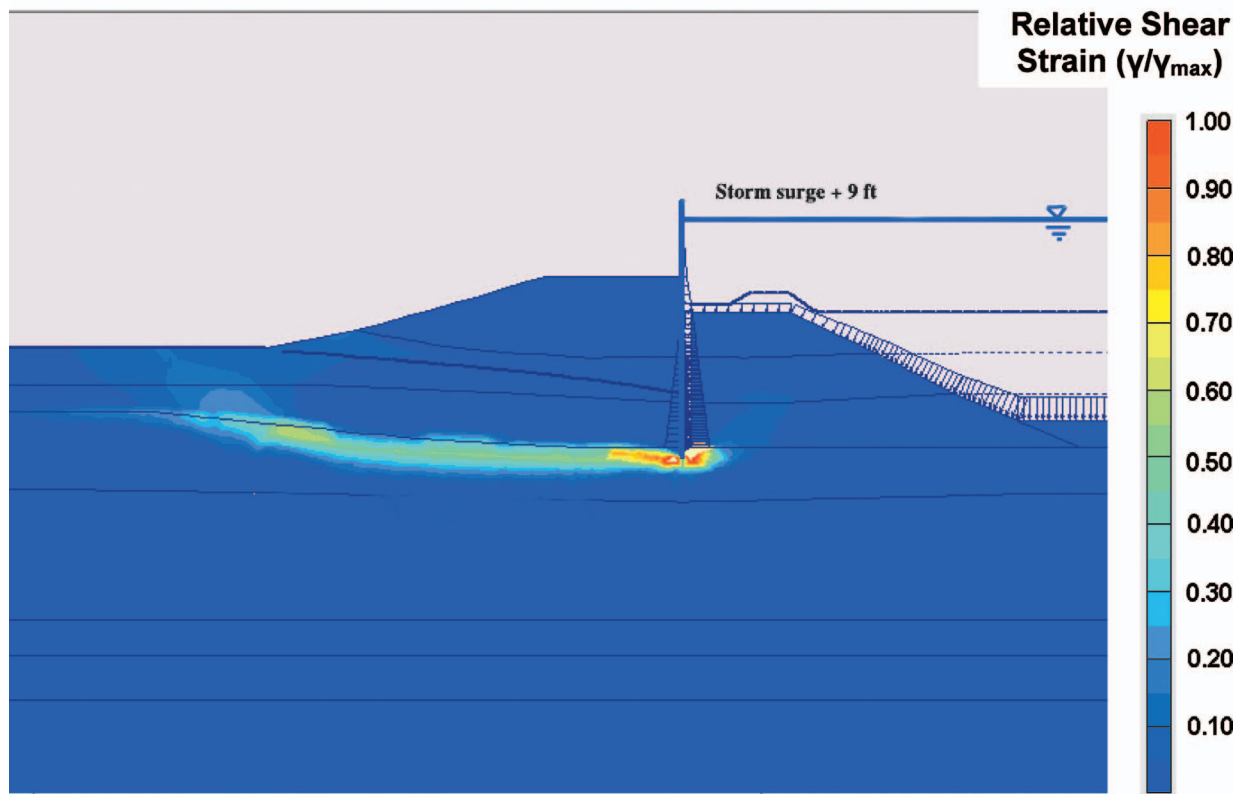
Fig. 12. (Color) (a) Normalized shear strain contours (shear strain divided by strain to failure) for a storm surge at Elev. +9 ft (MSL) at the London Avenue Canal breach site (west bank); gapping at outboard toe of floodwall is developed to full depth; (b) Normalized shear strain contours (shear strain divided by strain to failure) for a storm surge at Elev. +10 ft (MSL) at the London Avenue Canal distressed site (east bank); gapping at outboard toe of floodwall is partially developed

Table 2. Summary of Soil Model Parameters Used in PLAXIS Analyses

Stratum name	PLAXIS soil model	Shearing type[a]	γ_{unsat} (pcf)	γ_{sat} (pcf)	k_x (ft/day)	k_y (ft/day)	ν	E_{ref} (lb/ft^2)	c_{ref} (lb/ft^2)	φ' (^{0})
Embankment fill[a]	Mohr Coulomb	Undrained	N/A	90	0.0028	0.00028	0.35	1 500 000	600–800[b]	0.001
Marsh[a]	Mohr Coulomb	Undrained	N/A	80	0.3	0.03	0.35	170 000–320 000[b]	300–600[b]	0.001
Upper paludal clay[a]	Mohr Coulomb	Undrained	N/A	90	0.0028	0.00028	0.35	200 000–400 000[b]	200–400[b]	0.001
Clayey, silty sand	Mohr Coulomb	Drained	N/A	100	2.3	1.15	0.3	500 000	0.001	31
Loose sand	Mohr Coulomb	Drained	N/A	100	2.3	2.3	0.25	900 000	0.001	33
Medium dense sand	Mohr Coulomb	Drained	N/A	110	2.3	2.3	0.25	1 000 000	0.001	36
Dense sand	Mohr Coulomb	Drained	N/A	115	2.3	2.3	0.25	1 500 000	0.001	40

[a]These soils were modeled as "drained" during consolidation under the embankment fill loads, but were then modeled as "undrained" during the short-duration loading represented by the rapid storm surge rise within the canal.

[b]Values shown are "representative"; actual values vary both laterally and vertically as a function of overburden stress and overconsolidation ratio.

(the east and west bank) from pre-Katrina design investigations as well as from the post-Katrina ILIT and IPET studies (ILIT 2006; IPET 2007, Vol. V). They are developed using the same types of methods and interpretations that were described (for similar materials) in the preceding companion papers (Seed et al. 2008b, c), and complete details can be found in ILIT (2006). As discussed previously, the critical issue is the representative combination of effective friction angle and underseepage-induced pore pressure within the transitional contact between the top of the sands and the base of the overlying clay stratum.

Fig. 13 shows the resulting calculated overall factors of safety (based on Spencer's method) as a function of canal water level increase for (a) the west bank breach section; and (b) the east bank distressed section. The lightly infilled squares in Fig. 13 represent the results of analyses performed without including the presence of a water-filled gap on the outboard side of the sheetpile curtains, and the more darkly infilled squares at the left sides of these figures represent the results of analyses that model a water-filled gap at that location, which illustrates the importance of considering the formation of a water-filled gap in the analyses.

These water-filled gaps were observed by the posthurricane field geometries, on both sides of the canal, but estimates of the timing of the inception and propagation of these water-filled gaps
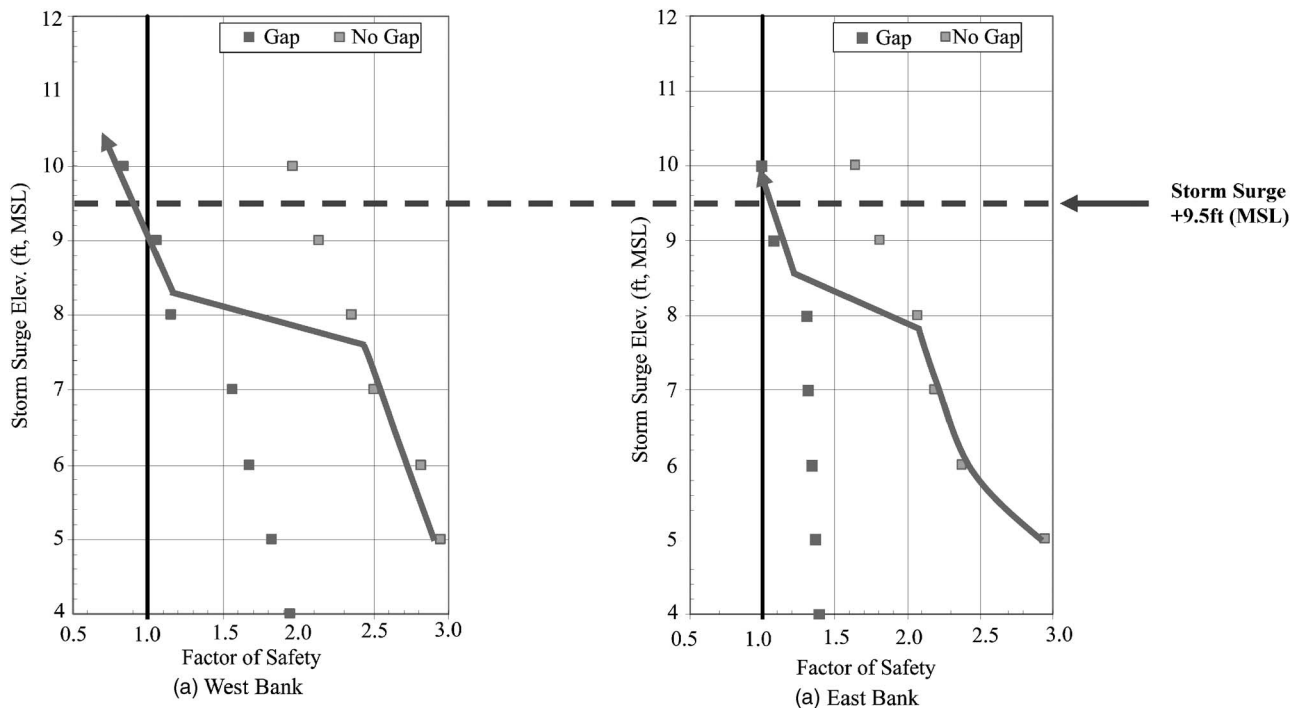

Fig. 13. Evolution of calculated factors of safety (versus rising canal water elevations) based on limit equilibrium analyses of (1) the London Avenue Canal North Breach; (2) the east bank distressed section; shows the best-estimated paths to failure
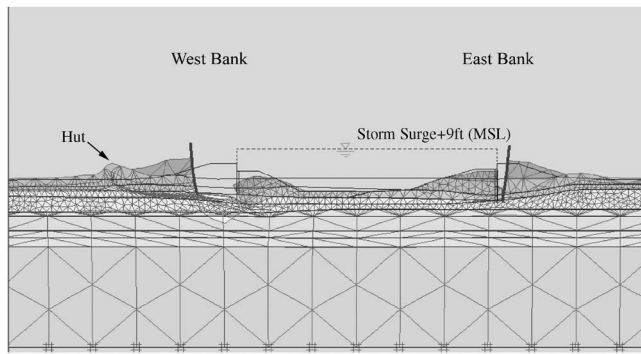
Fig. 14. Deformed mesh for a storm surge elevation +9 ft (MSL), North Breach failure section (west bank) and the east bank distressed section, London Avenue canal. Calculated displacements are scaled by a factor of 2 for clarity.

required the performance of finite-element analyses. Those analyses suggest that such gaps would begin to open at canal water elevations of approximately +7.5 to +8 ft, and that they opened rapidly thereafter, so that they were essentially fully open by the time that canal water levels reached Elev. +9 ft (MSL). Hence, the opening of these gaps is included in the interpretation of the results of the stability analyses (versus canal water elevation) shown in Fig. 13, and the solid lines (and arrows) in Fig. 13 represent the best overall interpretation of both the coupled seepage/limit equilibrium analyses and the finite-element analyses. The failure on the west bank occurred at approximately 7:15–7:30 a.m., and it thus occurred at a maximum water elevation within the canal of approximately Elev. +9 to +9.5 ft during the hurricane-induced storm surge (ILIT 2006; IPET 2007, Vol V; Van Heerden et al. 2006). Therefore, as shown in Fig. 13, the coupled seepage/stability analyses serve well to explain the difference between the performances of the east and west bank sections.

Finite-element analyses were also performed to simultaneously model the progressive rise on canal water levels on the morning of August 28, the resulting changes in underseepage-induced pore pressures, and the resulting displacements and overall stability of both the east bank and west bank sections. The program PLAXIS (Brinkgreve 2007) was used, and the parameters and models used to represent the different soil strata are presented in Table 2. Fig. 14 shows results from these PLAXIS analyses, for both sides of the canal at the same time, and for a canal water elevation of +9 ft, MSL at approximately 7:15 a.m., representing the best estimate of conditions at the point of inception of the west bank failure. Fig. 14 shows calculated displacements (exaggerated by a factor of 2 for visual clarity) at that time and water stage. As shown in Fig. 14, the west bank section has begun to experience large lateral displacements, whereas the east bank side has opened a water-filled gap but has not yet experienced large overall movements associated with global instability (instead just a bit of shearing at the inboard toe). This is in good agreement with the posthurricane field observations (see Figs. 8–11).

Based on these studies and analyses, the primary reason for the difference in observed field performance between the (failed) west bank and the (distressed) east bank is the subtle difference in subsurface stratigraphy within the upper foundation soils beneath the inboard sides of the two embankment sections. As shown in Figs. 9, 10, 12, and 14, the inclination (or slope) of the critical

transitional interface (which is the critical slippage surface) at the top of the sands is very slight at the west bank (breach) section, whereas the slope of this same transitional feature (and slippage surface) is significantly more steeply inclined at the east bank (distressed) section. Thus, the failure on the east bank side had to push the inboard half of the levee embankment "up a slightly steeper hill" as the increased (uphill) slope of this slippage surface served to slightly enhance the overall stability for this potential failure mode.

This, in turn, points up the importance of details in subsurface stratigraphy. The initial design analyses for both sections (east bank and west bank) had been performed assuming horizontally layered stratigraphy (USACE 1990). Such an approach would not have been able to recognize the importance of the subtle changes in slope of the transitional interfaces between the upper sands and the overlying, progressively finer (and clayier) soils beneath the inboard sides of the two levee embankment sections. The IPET study concurs that the most likely cause of the north breach on the west bank was slope instability, due in large part to underseepage-induced pore pressure increases and resulting reductions in strength of the foundation sands at the inboard toe. Stratigraphic details in the region at and inboard of the levee toe for the ILIT analyses of the west bank breach section differed from those of this (ILIT) study, however, as a horizontal interface was modeled at the contact between the top of the sands and the overlying lacustrine clay (beneath the marsh stratum).

As noted earlier, seepage distress in the form of boils and sinkholes was observed at the east bank (distressed) section, and similar toe boil ejecta were noted near the toe (at the base of the displaced levee slope) on the flank of the west bank (breached) section. The occurrence of such "piping erosion" does not mean that piping erosion was the cause of the west bank failure. The above-described analyses indicate that lateral water forces within a water-filled gap at the outboard sides of the sheetpile curtains, and consequent elevated pore pressures in the subsurface sands, alone could have caused the failure. Erosional boils and piping were apparently also under way, but these probably did not have time to produce a full failure before the eventual translational failure occurred. There were, apparently, two competing potential failure modes (or processes) at this site, as was found to be the case at a number of other sites investigated during these studies as well. It is thus important to assess fully all possibilities before reaching conclusions as to the probable cause(s) of failures.

Lastly, it should be noted that the design objective at these two sections was a minimum factor of safety of FS ≥ 1.3 for the transient conditions associated with a hurricane-induced canal water level of at least Elev. +12.5 ft (MSL). As shown in Fig. 13, these sections did not achieve this targeted performance. This was due in large part to: (1) the failure to include the possibility of a water-filled gap in the stability analyses for the initial design studies; and (2) the failure to appreciate fully the dangers associated with underseepage flows passing through the foundation sands and beneath the inadequately deep sheetpile curtain. The initial design studies assumed that the canal would be lined with sufficient silt to preclude the possibility of significant underseepage flow during the relative short duration of a hurricane-induced storm surge. Unfortunately, that did not recognize the possibility of (1) subsequent dredging to increase the canal capacity, which could remove the silt blanket; and (2) potential for scour due to flow along the canal in response to a storm surge on Lake Pontchartrain. Both the failure to analyze the water-filled gap, and the failure to adequately recognize underseepage risk, occurred at other failure sites during Hurricane Katrina as well (ILIT 2006).
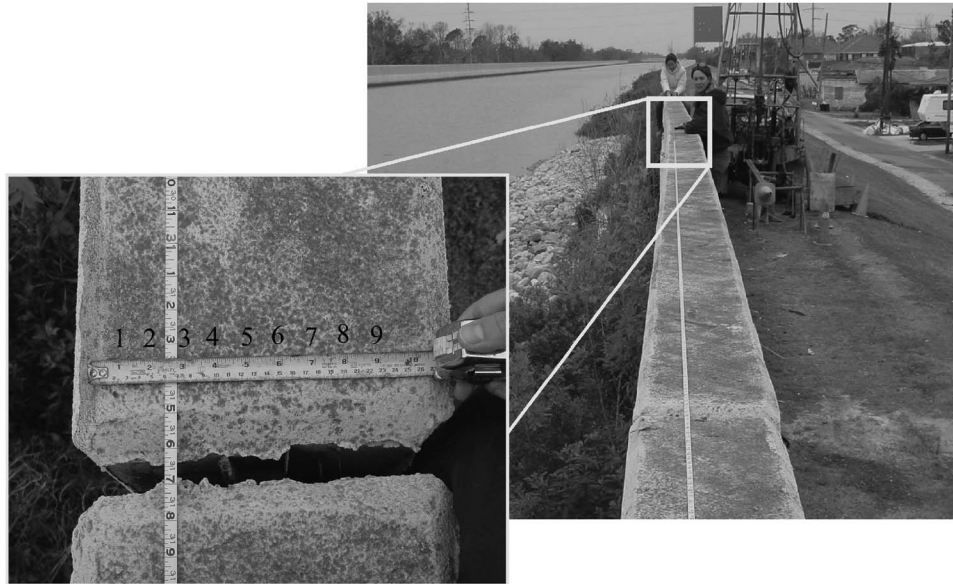
Fig. 15. Lateral displacement of 3 in. at top of distressed I-wall section on the west bank of the 17th Street drainage canal, directly across from the large failure (and breach) on the east bank

Considering these findings, a reevaluation of ongoing efforts to reassess and upgrade the New Orleans regional flood protection systems is warranted (ILIT 2006; IPET 2007).

17th Street Canal

Breach Section on the East Bank
The third major failure on the drainage canals occurred on the east bank of the 17th Street drainage canal, near the north end of the canal and it was presented in detail in the preceding companion paper (Seed et al. 2008c). This failure was a lateral translational failure, as shown in Fig. 4 of that companion paper, during which the inboard half of the earthen levee embankment slid laterally a distance of approximately 49 ft. A water-filled gap had opened on the outboard side of the sheetpile curtain supporting the concrete I-wall, effectively cutting the earthen embankment in half, and lateral water pressures within this gap drove the inboard half of the embankment sideways.

The displaced embankment section slid sideways along a shear surface confined primarily within a thin layer of weak and sensitive organic clayey silt. This layer, typically only about 1 in. in thickness, was produced by a previous hurricane approximately 700 years ago (based on carbon dating of pollens), and it was well-hidden by an overlying layer of twigs, leaves, and other organic detritus also laid down by that same previous hurricane. As a result, this stratum went undetected during the initial site investigations for the original design of this section (USACE 1989). A number of factors contributed to this failure, and these were discussed in Seed et al. (2008c).

Distressed Section on the West Bank
One of the important little-recognized events during Hurricane Katrina was the near failure of the levee and floodwall section on the west bank of the 17th Street drainage canal, directly across from the large failure on the east bank that was described earlier. This west bank section suffered lateral displacements of several inches, and our (ILIT) analyses showed that this west bank section also came perilously close to failure. The near failure of this

distressed section would have had catastrophic consequences if it had progressed to a full failure (and breach). The 17th Street drainage canal defines the western edge of the main (Metro) Orleans East Bank protected basin, separating this heavily populated basin from the adjacent basin to the west. The basin to the west is Jefferson Parish; a second heavily populated basin that is not part of the City of New Orleans, but which also contains the homes and businesses of several hundred thousand people. If the west bank of the 17th Street canal had failed instead of the east bank, the resulting inundation of this heavily populated region would have nearly doubled the overall number of homes and businesses flooded by the overall event, and would have massively increased the overall damages, and likely the loss of life, produced by this hurricane.

Fig. 15 shows the top of the concrete I-wall at this distressed section on the west bank, where 3 in. of lateral deflection of the I-wall occurred at this location. The potential seriousness of that deflection can only be investigated by analyses. Accordingly, detailed analyses (using both conventional limit equilibrium and finite-element analyses) were performed at this section, and the full results are available in ILIT (2006).

The analyses closely parallel those described in the preceding companion paper for the major failure (and breach) section across the canal on the east bank (Seed et al. 2008c), as the foundation soils conditions and stratigraphy are similar beneath the east bank and west bank sections. One critical difference, however, was the depth at which the critical stratum of weak, sensitive organic silty clay occurred beneath the west bank distressed section. Fig. 16(a) shows a cross section through this distressed section. The critical (thin, weak, sensitive) organic silty clay stratum (at this west bank site) is located higher up from the base of the sheetpile curtain than it had on the (breached) east bank section. This was the critical difference that permitted the east bank section to fail first, whereas the west bank section remained barely stable.

Because it was not close enough to the base of the sheetpiles to form the basal sliding surface for the most critical mechanism, this thin (weak and sensitive) hurricane stratum was not the "critical" stratum at the distressed west bank site. Instead, the most
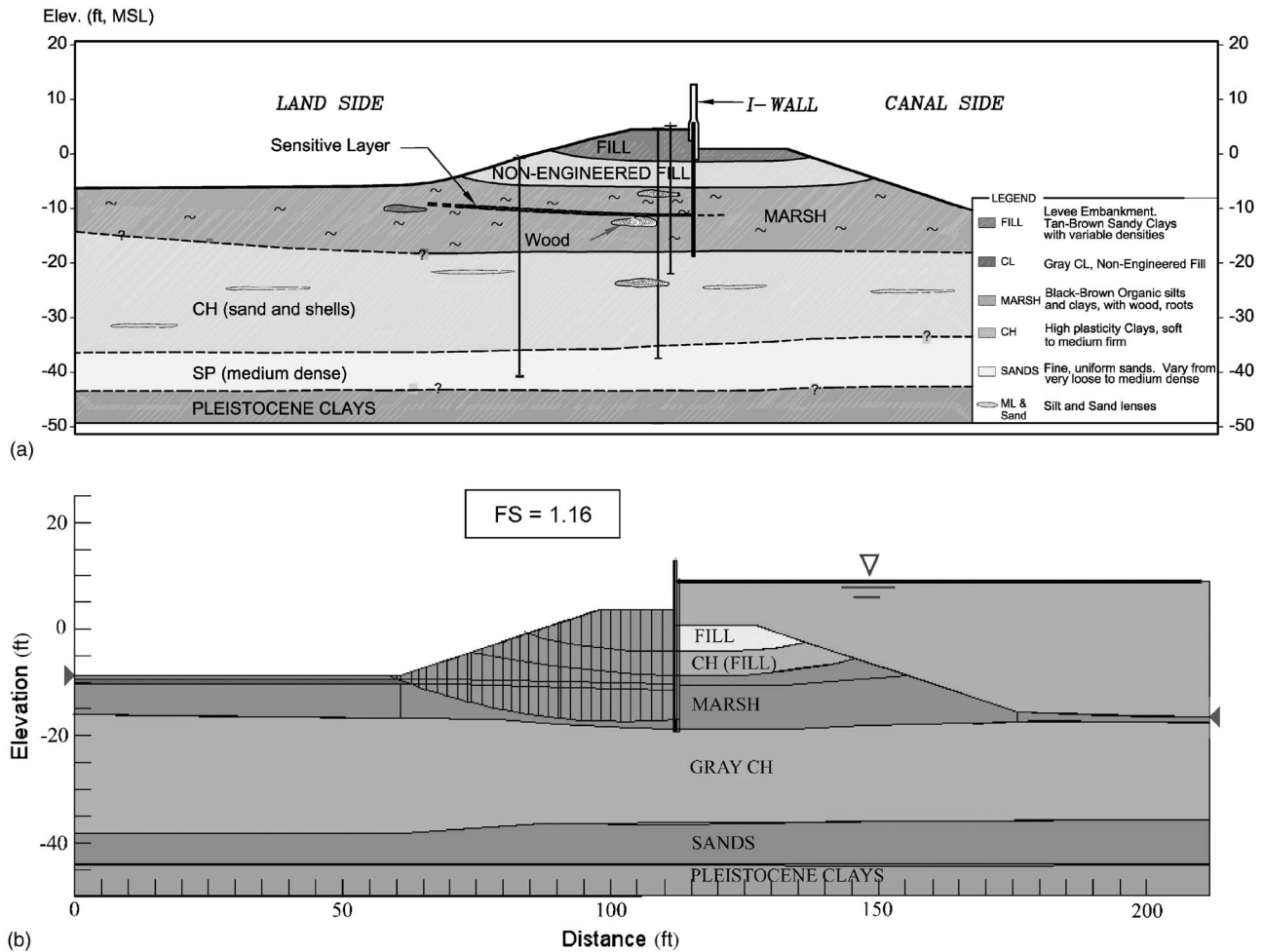
Fig. 16. (a) Cross section at distressed levee embankment and I-wall section on the west bank of the 17th Street drainage canal; (b) stability analysis of the west side of the 17th Street canal with water at Elev. +9 ft (MSL) for case of failure along the base of the marsh layer (with water-filled gap)

critical potential failure mechanism was found (both by finite-element analyses, and by conventional limit equilibrium analyses) to be a deeper, semirotational/translational failure through either the weak organic marsh/swamp deposits, or through the soft gray clays (CH) underlying these marsh deposits. Fig. 16(b) shows the most critical failure mechanism at this site, based on limit equilibrium analyses (Spencer's method) and indicates that the calculated factor of safety (with a water-filled gap on the outboard side of the sheetpile curtain) is approximately FS=1.16 with a canal water level of Elev. +9 ft (MSL). Strengths of the critical embankment and soil strata in this analysis are exactly the same as for the same strata (on the west bank) as presented previously in Tables 1 and 2 of Seed et al. (2008c).

Fig. 17 shows the evolution of factor of safety for this distressed section as water levels rose within the canal, again based on the same soil strengths as were used for the east bank (breach) section. The maximum water level within the north end of this canal rose to approximately +8 to +8.5 ft (ILIT 2006). Based on finite-element analyses, it was determined that a water-filled gap would have begun to open on the outboard side of the sheetpiles at elevations of between +8 and +9.5 ft (MSL), depending upon the ranges of stiffness and strength modeled in those analyses. This is reflected in Fig. 17, where the right-hand side open squares represent the results of limit equilibrium analyses Spencer's method) performed without a water-filled gap, and the left-

hand side closed squares represent similar limit equilibrium analyses with a water-filled gap. As shown Fig. 17, the best analytical estimates are that such a gap would have begun to open at a water level of approximately +8 to +9.5 ft, in which case the overall factor of safety would have then been rapidly reduced to a low level; not much greater than 1.0 (ILIT 2006).

It is interesting to note that the storm surge at the edge of Lake Pontchartrain, at the north end of the canal, rose to an elevation of approximately +9.5 ft (MSL) at its peak, but that debris from lakeshore facilities (e.g., docks and buildings) had partially plugged the openings beneath the bridge at the north end of the canal, partially impeding inflow so that the maximum surge rise within the canal appears to have been on the order of +8.5 ft, as discussed in Seed et al. (2008c). The failure of the levee section on the opposite (east) bank also began approximately 2 h before the peak surge in the lake, and this appears to have also served to reduce the surge levels within the canal. If the full storm surge had risen to +9.5 ft within the canal, the analyses presented in Fig. 17 suggest that distressed west bank section may have failed and produced a breach.

This pair of case histories (the failed east bank breach section, and the distressed west bank section) thus provides an additional excellent test of the accuracy and reliability of current geotechnical analysis approaches. As shown in the preceding companion paper, both limit equilibrium analyses and finite-element analyses
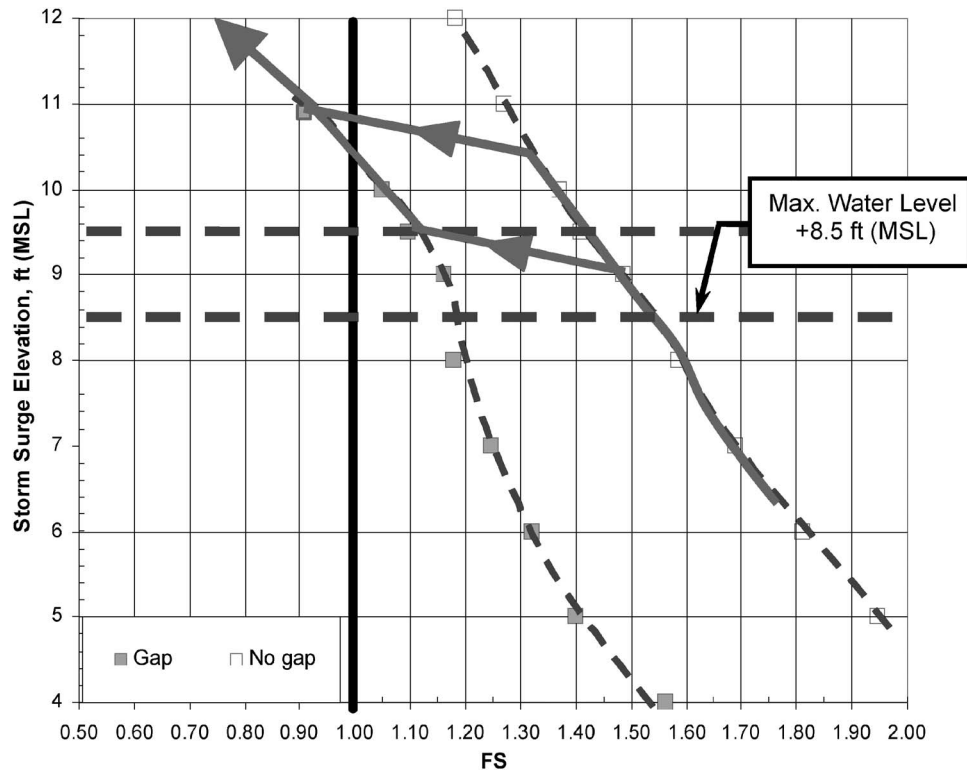
Fig. 17. Evolution of calculated factor of safety versus canal water level for the distressed section on the west bank of the 17th Street drainage canal

are well able to explain the failure (and breach) on the east bank of the 17th Street drainage canal (ILIT 2006). Equally important, limit equilibrium analysis methods and finite-element analyses (performed similarly, and using essentially the same strength interpretations) are both also able to explain the marginal survival of the distressed section on the west bank of the 17th Street canal. Thus, it appears that currently available limit equilibrium analyses, and available finite-element analysis tools, are again both able to accurately explain and reproduce the observed failure on the east bank, and the incipient (or near-failure) of the distressed section on the west bank.

Finally, it should also be noted that the design at this west bank distressed section was targeted to produce a minimum factor of safety of at least FS ≥ 1.3 for a design canal water elevation of at least +12.5 ft (MSL). As shown in Fig. 17, the design failed to achieve this by a considerable margin.

Conclusions

Nearly half of the loss of life produced by Hurricane Katrina, and a similar fraction of the economic damages, occurred within the main (Metro) Orleans East bank protected basin (U.S. Senate Committee on Homeland Security and Governmental Affairs 2006). The failures, and near failures, that occurred along the defensive perimeter of this protected basin present a number of important lessons.

Several relatively minor failures occurred at the east edge of this protected basin, along the west bank of the IHNC channel. These failed to scour paths down to a depth below sea level. Although they briefly admitted inflow of floodwaters during the peak of the storm surge, inflow through these features soon ceased as the storm surge subsequently subsided. One of these

failures occurred at a relatively complex "multiple penetration"; a location at which a highway, a Port road, and a rail line all passed through the federal levee perimeter. The lack of an overall integration of the different flood protection elements was likely a key cause of the failure at this location. A similar failure at the same location occurred during Hurricane Betsy in 1965.

Two other failures along this eastern frontage were the result of the use of unsuitable, highly erodeable levee embankment fill materials (lightweight shell-sand fills), and without taking adequate steps to protect these unsuitable materials (e.g., internal cutoff walls, erosion protection armoring). The final failure along this frontage was the result of overtopping of a concrete I-wall, resultant erosion behind the wall, and the localized lateral stability failure of this section (and resultant separation between adjacent concrete I-wall panels) due in large part to the reduced lateral support for the floodwall and its supporting sheetpiles.

The main sources of the floodwaters that filled nearly 85% of this heavily populated basin, however, were due to the three major breaches that occurred along the drainage canals at the north end of the protected basin. All three of these failures rapidly scoured paths below sea level, and as a result, inflow through these three breaches continued for 3 days, eventually equilibrating on the afternoon of Thursday, September 1.

The USACE had long sought authority and funding to install floodgates at the north ends of these three canals (at the edge of Lake Pontchartrain) to prevent hurricane-induced storm surges from entering these canals. If that had been done, it is unlikely that these failures would have occurred, and approximately half of the overall damages and loss of life due to Hurricane Katrina would have been prevented.

A second policy failure associated with conflicting local objectives occurred at the south end of the Orleans canal. At this location, the final 200 ft of concrete floodwall were never constructed

due to differences of opinion regarding how to protect an aging interior pump station. As the storm surge rose within this canal, there was a lengthy gap where the top of the levee was approximately 6–6.5 ft lower than the tops of the floodwalls along the other 10 mi of levees and floodwalls lining this canal. As a result, during the peak of the storm surge, floodwaters simply flowed freely into the heart of the Orleans East Bank at this location.

All three of the major failures along the drainage canals involved the opening of a water-filled gap on the outboard side of the concrete floodwalls (and their supporting sheetpile curtains). These gaps, which opened between the sheetpiles and the outboard halves of the earthen levee embankments, effectively cut the embankments in half, and they also permitted lateral water forces to be applied over essentially the full depths of the sheetpile curtains. These water-filled gaps had not been considered in the analyses upon which the designs of these sections had been based.

Two of the three major failures along the drainage canals were due primarily to underseepage passing beneath sheetpile curtains that had been designed primarily to support the concrete I-walls, rather than to prevent underseepage. The south breach on the east bank of the London Avenue canal was the result of underseepage and piping. The north breach on the west bank of the London Avenue canal was a lateral translational failure of the inboard half of the earthen levee embankment, pushed sideways by the water pressures from the water-filled gap acting laterally against the sheetpile curtain. The foundation soils were reduced in strength due to underseepage-induced pore pressure increases, and a lateral translational foundation failure resulted. At this location, another failure mode in the form of underseepage piping and erosion also appeared to be developing, but was probably not far enough along as to be the primary cause of the failure.

An overarching lesson of major importance was the level of accuracy and reliability with which engineering analyses were able to reproduce, and explain, the observed field performances of both failed sections and sections that nearly failed. At all of the major breach sites studied in the Orleans East Bank, geotechnical analyses were found well able to provide insights that explain both the occurrence and nonoccurrence of failures. This included both conventional limit equilibrium analysis methods, as well as more advanced finite-element analyses. Both methods produced results that were in good agreement with each other, and both methods were able to consistently reproduce and explain observed field performance. That included several challenging sites at which conditions on opposite banks of the same canal were similar, so that only subtle differences spelled the difference between marginal failure and marginal success (survival of one of the two nearly identical sections without breaching). This repeatedly successful performance of current analysis methods is encouraging, and it highlights the importance of sound analysis that is supported through comprehensive field and laboratory testing coupled with an appreciation of the importance of local geology.

Finally, the low factor of safety (FS=1.3) used in the original designs for the transient loading conditions represented by hurricane storm surge allowed little room for error, oversights, or undetected geological details. As some level of uncertainty will always remain, the use of a higher factor of safety for rare, transient loadings that could produce catastrophic failure of systems protecting large populations should be considered.

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