Technical Report VI

2D and 3D Seepage Analyses:

Lower 9th Ward Breaches

Report to

Katrina Canal Breaches Consolidated Litigation

[Civil Action Number: 05-4182 "K" (2)]

United States District Court

Eastern District of Louisiana

Pertains to MR-GO, Robinson

[No. 06-2268]

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North Breach

Forensic engineering studies indicate the North Breach on the east bank of the IHNC-Lower 9th Ward initiated before the wall was overtopped (about 5:00 am, CDT), with the breach fully developing between 6:00 and 7:00 am on August 29, 2005.

Based on analyses of available aerial and ground photography, an excavation was present on the waterside of the IHNC levee, located approximately 60 ft away from the centerline of the levee – floodwall. Based on available photographic evidence, the hole was determined to be approximately 23 ft deep and 70 ft wide. The soil profile at this site was characterized by a short levee underlain by a thick marsh deposit, which seems to be continuous below the entire IHNC area. This swamp layer is located atop a high plasticity clay layer bottomed below elevation -50 ft. The sheet pile top was located at +12.0 ft, and extended down to elevation -10.5 ft. It is evident that the sheet pile penetration was not sufficient to completely cut off the Marsh layer. Figure 1 shows the interpreted cross section at this location.

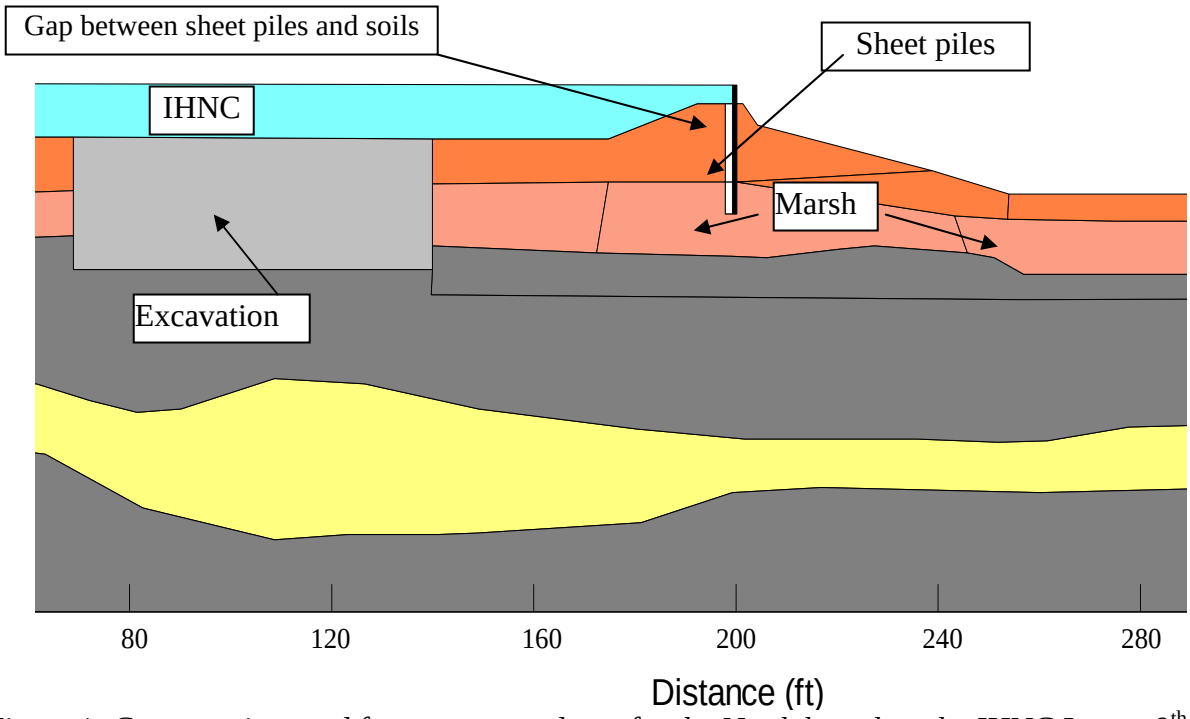


Figure 1**:** Cross section used for seepage analyses for the North breach at the IHNC Lower 9th Ward

2D Seepage Analyses

Transient seepage analyses were performed for the rising water level in the IHNC according to the measured surge elevation hydrographs in this area (Figure 2). The surge hydrograph was input into the seepage models which generated analytical results beginning at a water elevation +2 ft. Table 1 summarizes the hydraulic properties of the soil layers, including conductivity at saturation and anisotropy ratio (ratio of vertical to horizontal permeability, ky/kx). Since no field hydraulic conductivity tests were performed on these soils, best estimates of hydraulic conductivities based on data from available studies on comparable materials were used. The last row in Table 1 gives the permeability value used for gap formed between the levee and sheet-pile.

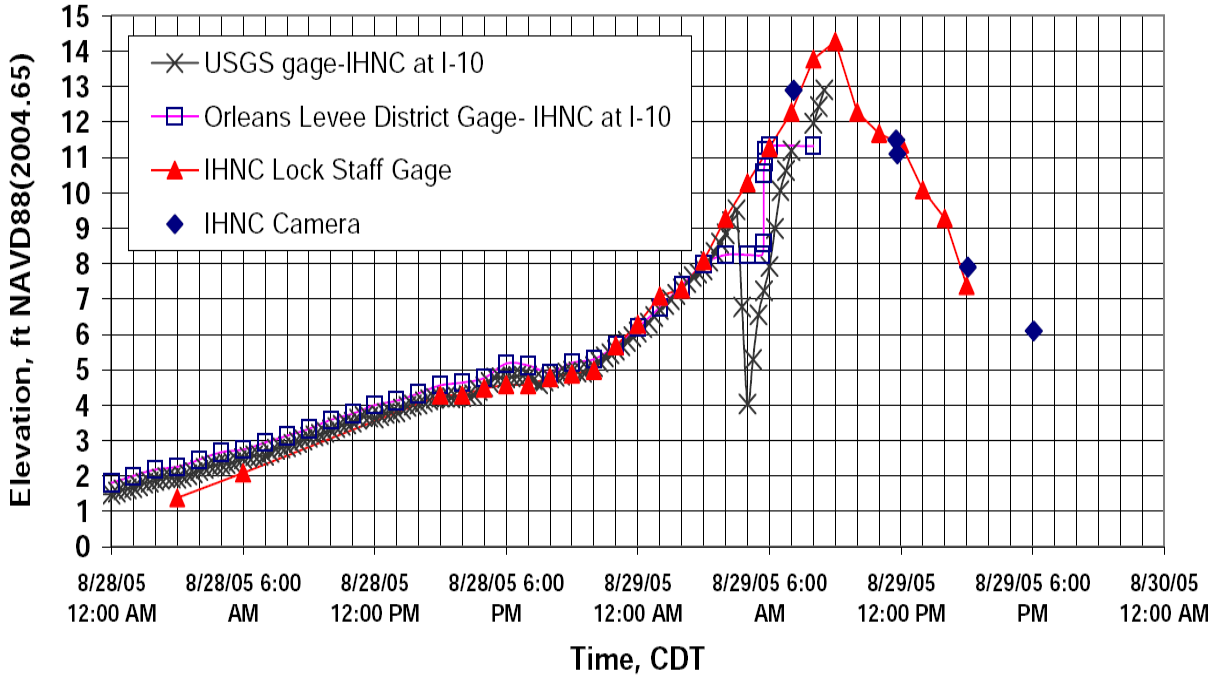


Figure 2: Hydrographs showing measured (and photographed) water levels at gage stations along the IHNC (USACE 2007).

Table 1: Material properties used in IHNC breach seepage analyses

Material	**Horizontal Permeability (kx) cm/s**	**Anisotropy Ratio (ky/kx)**
Clays	10^{-6}	0.3
Sands	10^{-2}	0.25
Marsh (upper bound)	10^{-3}	0.10
Waterside Excavation Fills	10^{-1}	1.0
Gap	100	1.0

Based on results from transient seepage analyses, Figure 3 shows the evolution of pore water pressure near the toe of the levee. After 6:00 am on August 29, 2005, there is a sudden increase in pore pressures, related to the point in time when the gap forms and water starts hydraulically loading the soils beneath the levee from the tip of the pile.

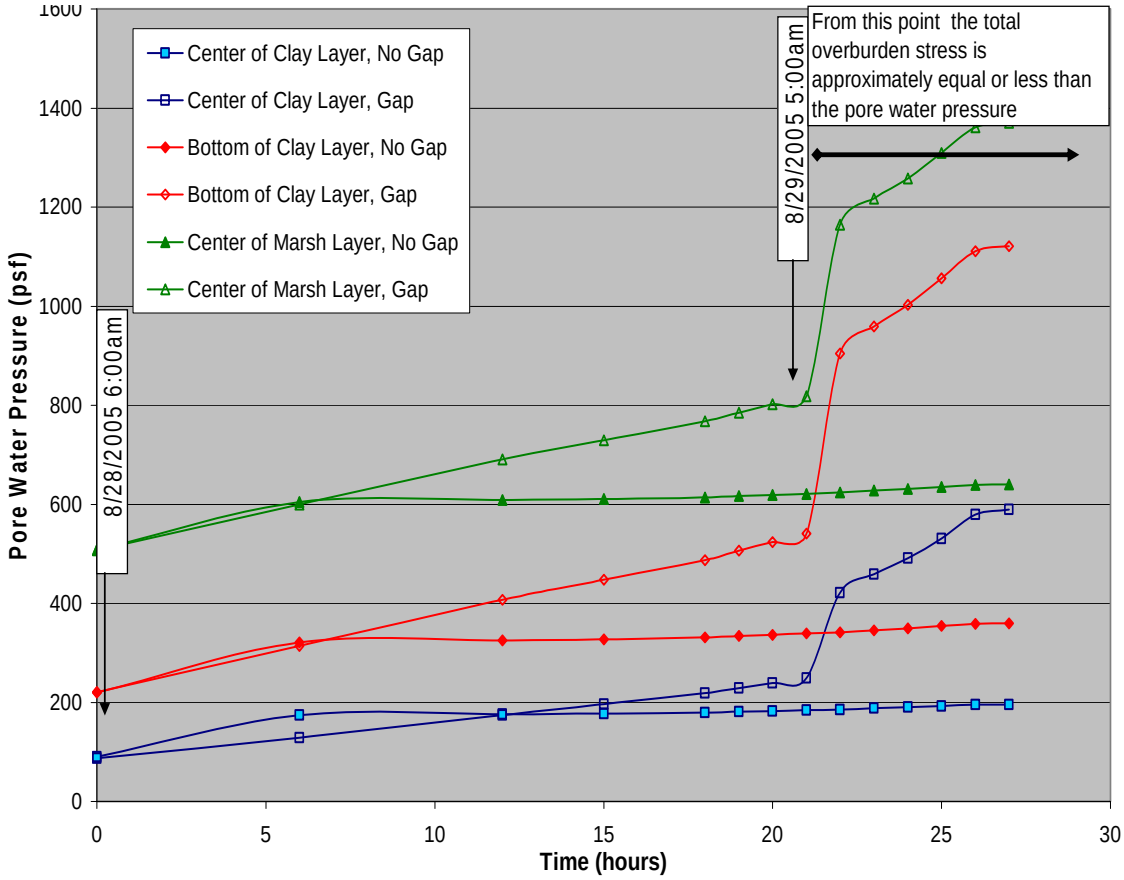


Figure 3: Transient seepage analyses results. Pore pressure versus time near the toe of the levee.

Figure 4 shows the computed hydraulic gradients. An arrow points to the approximate time of 5:00 am on August 29, 2005. These high flow gradients would lead to softening of sensitive soils near the toe of the levee.

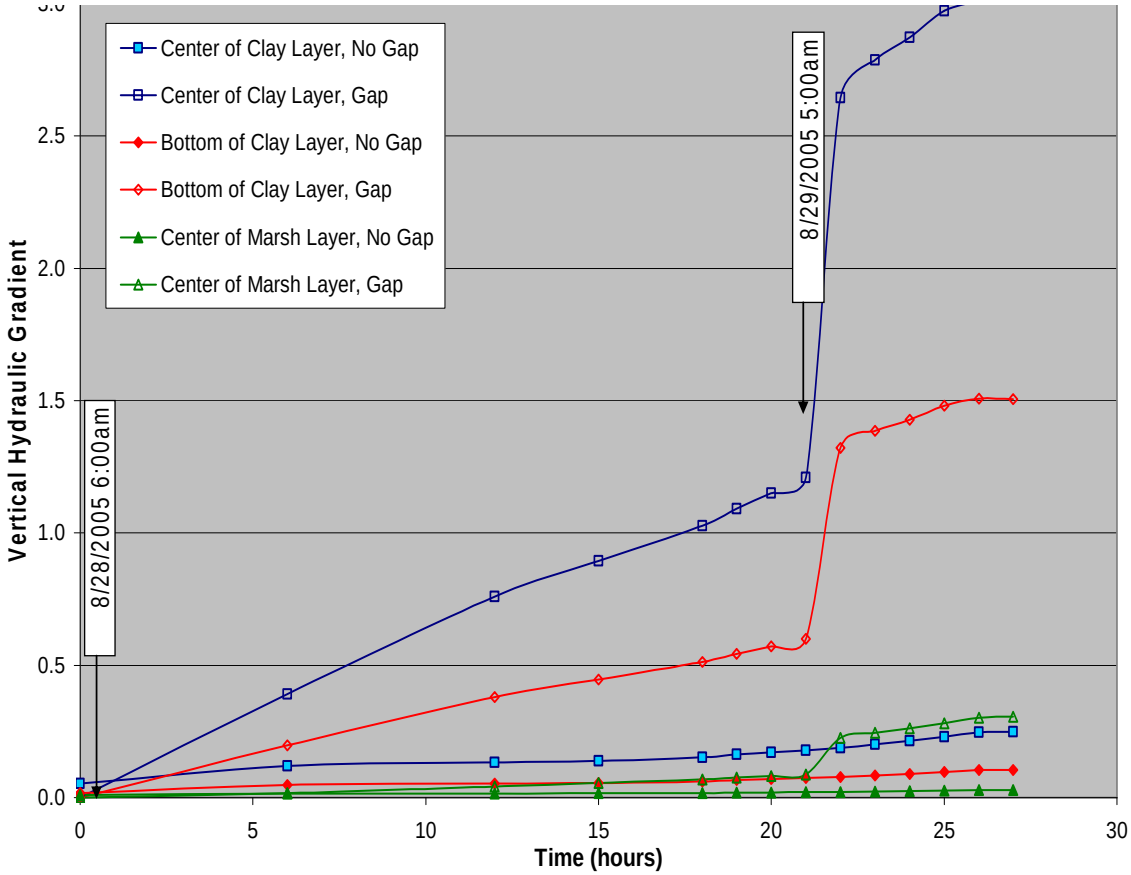


Figure 4: Hydraulic gradients near the toe of the levee at the North Breach. Marsh kx = 10^{-3} cm/s

Figure 5 shows a plot of total head contours showing that the majority of head loss takes place through the shallow clay on the landside of the North Breach; evidenced by the head contours near the toe. This translates to the high gradients shown in Figure 6, where hydraulic gradients i ≥ 1.0 are computed between the landside toe and the drainage canal structure.

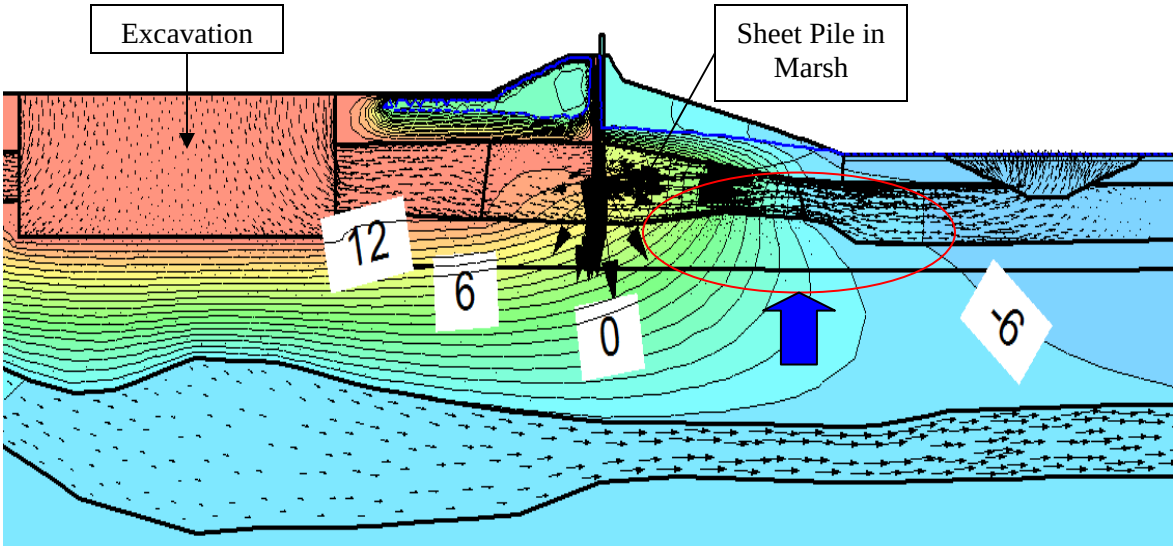


Figure 5:Transient model result, Total Heads in feet. Time: between 7:00 and 8:00 am on August 29, 2005.

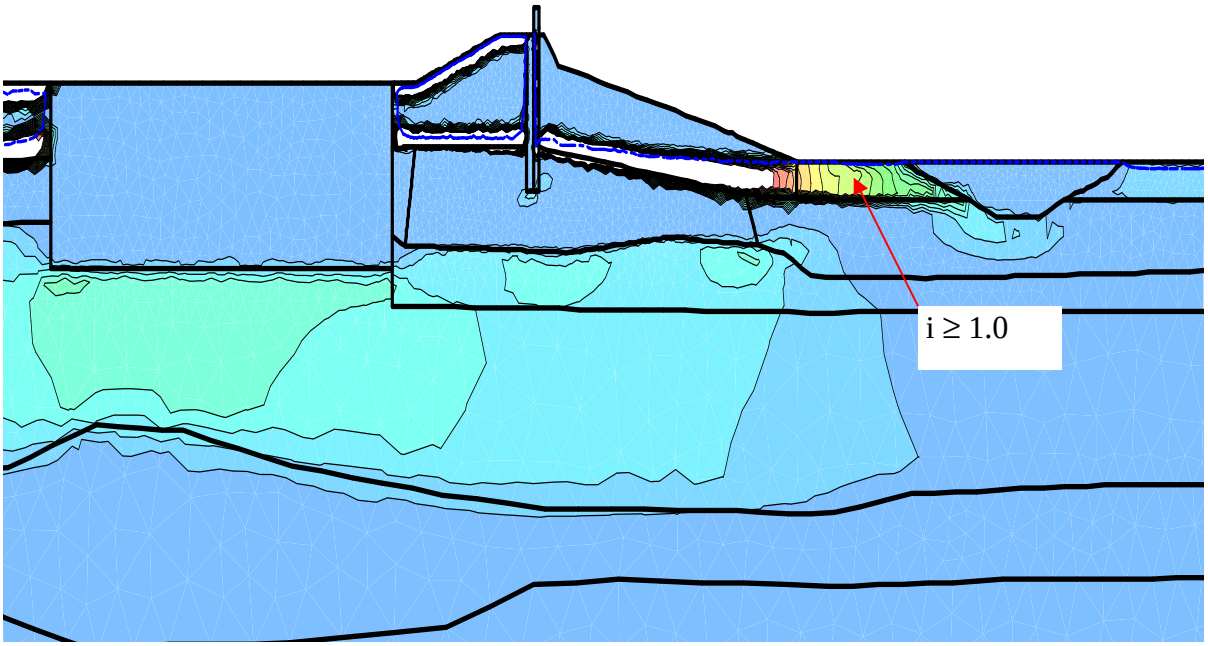


Figure 6: Transient 2D seepage hydraulic gradients (contour interval = 0.1). Time: between 7:00 and 8:00 am on August 29, 2005.

3D Seepage Analyses

The cross section used for the 2D model was used as a starting point for the 3D computations. The excavation was again modeled with vertical walls, approximately 60 ft away from the centerline of the levee (Figure 7). The permeable layer used for the excavation fill is shown by the gray area in Figure 7.

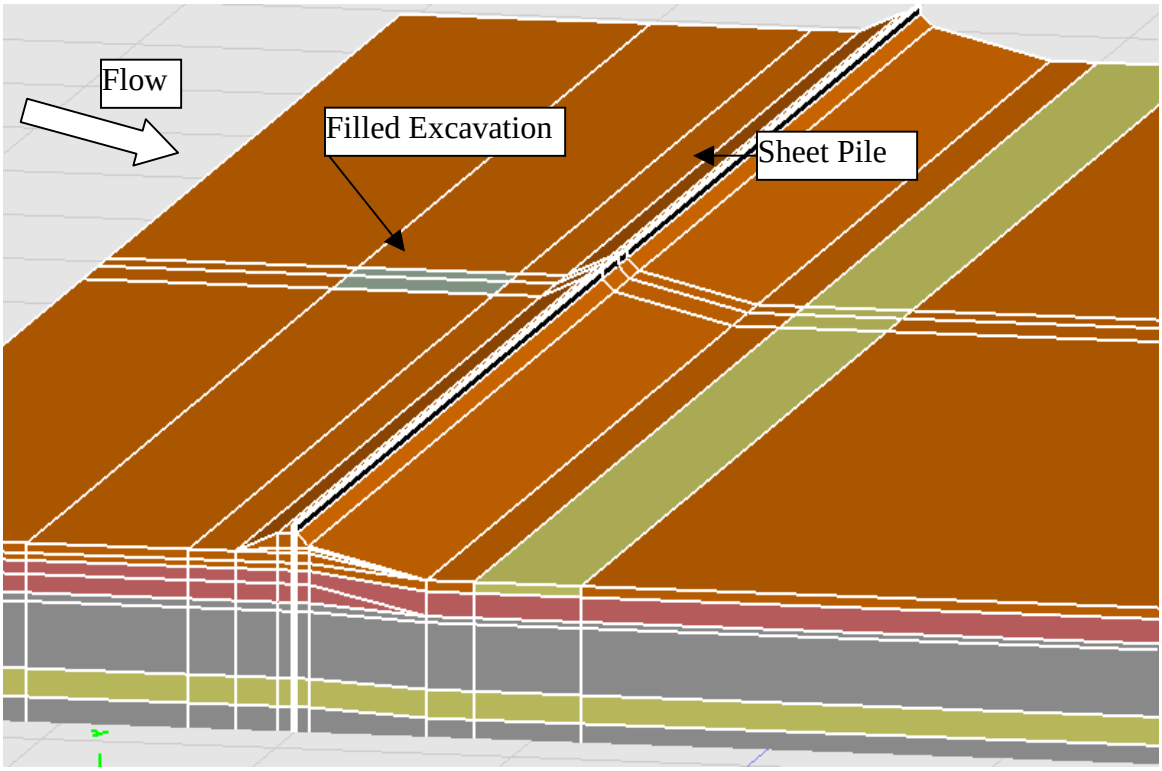


Figure 7: 3D seepage model for IHNC-Lower Ninth Ward North breach

Figure 8 shows the total head contours computed using the 3D model. The plot shows a similar distribution of heads as the 2D model, with minor losses in the soil layers upstream of the sheet pile located near the vicinity of the excavation. Figure 9 shows the computed hydraulic gradients at the surface concentrating several tens of feet landward of the toe. Figure 10 shows hydraulic gradient values developed through the storm, at the center of the clay layer beneath the landside

levee toe for two points along sections or ‘slices’ of the 3D model; one through the filled excavation, and another one immediately outside the excavated area. There is a 30% increase in the gradients near the landside toe in the cross section through the filled excavation, compared to the section outside this area. The blue series corresponds to the upper plot line of Figure 4 (center of clay layer with gap), with a hydraulic gradient value below the landside toe just under i = 2.0 at 6:00 am on August 29, 2005, and the black line captures the 3D effect of the excavation, by increasing the gradients on the area beneath the landside toe.

Computed flow quantities on the lines normal to the levee centerline show that the flow seeping through the waterside excavation towards the landside toe is approximately 20% higher than the flow 100 ft outside of the excavation.

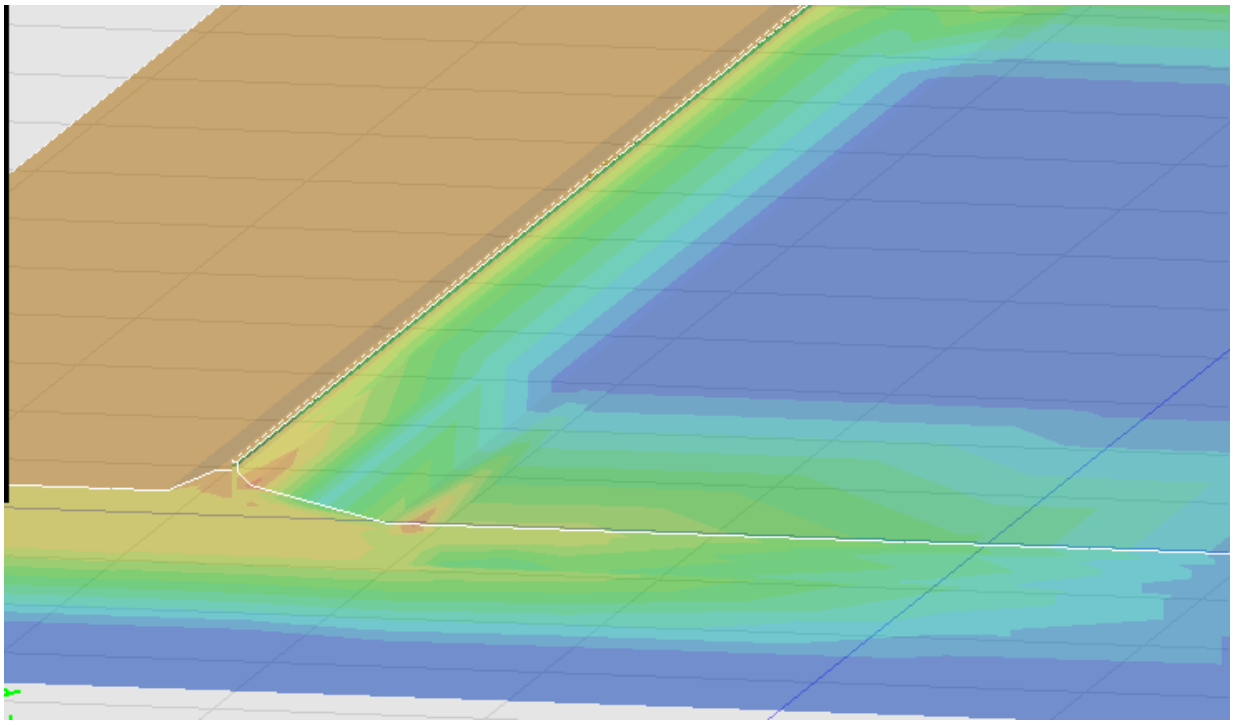


Figure 8: 3D total head contours. Time between 7:00 and 8:00 am on August 29, 2005.

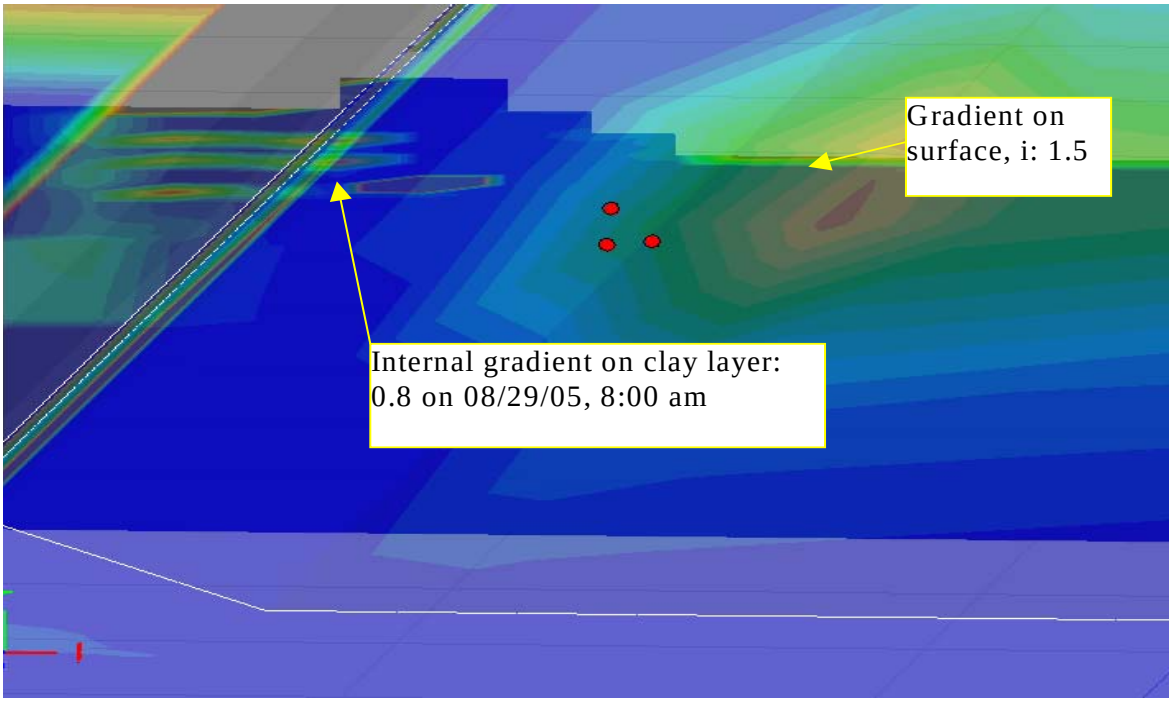


Figure 9: 3D hydraulic gradients. Time: between 7:00 and 8:00 am on August 29, 2005.

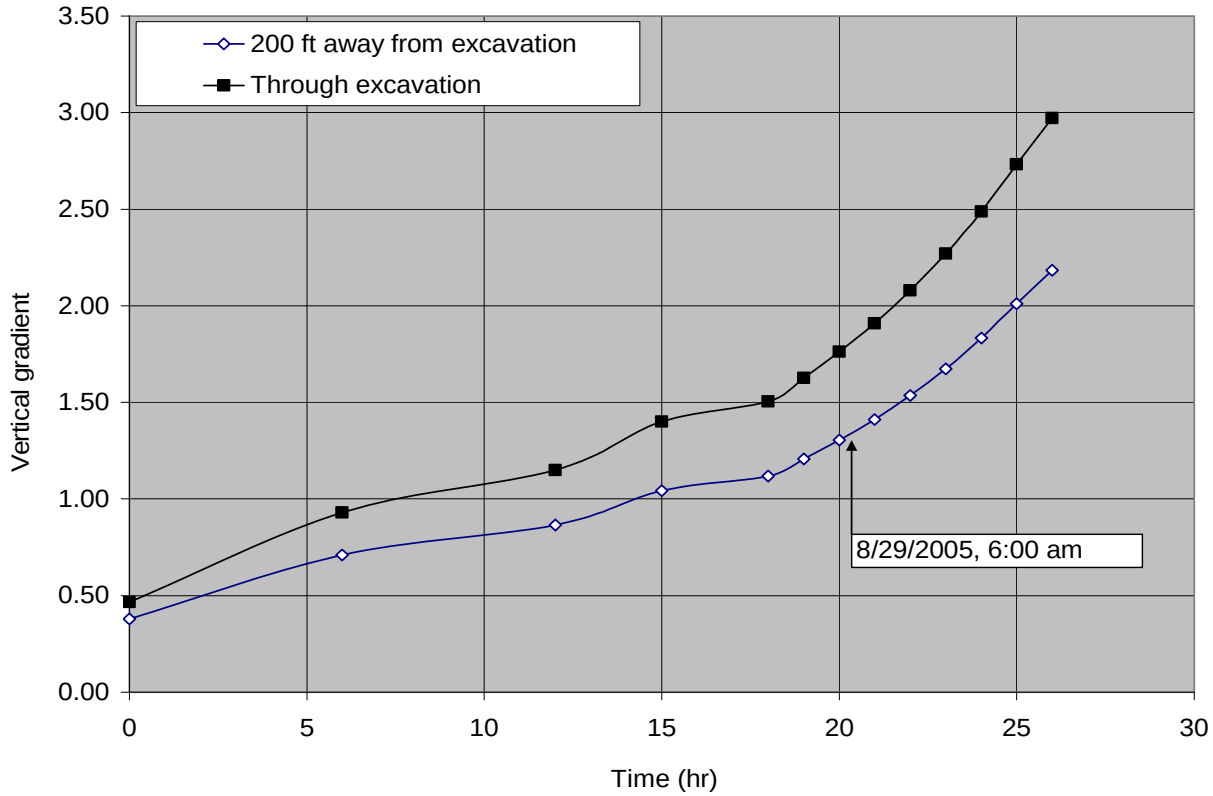


Figure 10: Gradients along two lines normal to the levee centerline, Time: between 5:00 and 6:00 am on August 29, 2005.

South Breach

Results from the forensic engineering investigations indicate that a deeper-longer translational failure surface was the most likely failure mechanism in the development of the South breach along the east bank of the IHNC-Lower Ninth Ward. This was a 900 ft long failed section. Similarly to the breach to the north, this failure was influenced by formation of a gap between the sheet pile and waterside fill, a nearby waterside excavation filled with permeable material cutting the marsh layers and sheet pile with insufficient length to cut flow paths through permeable marsh deposits helped develop this breach.

This breach is believed to have developed between 7:30 and 9:00 am August 29, 2005. Figure 11 shows the interpreted profile for a representative section through this breach. Similar to the north section, a clay layer covers the surface extending from the embankment fill landward. The top of the Marsh deposit is located at about elevation –5 ft, but a thin clay layer appears to divide the marsh layer.

Sheet pile extends from +12.0 ft to -10.5 ft (+4 to -3.5 m), and again, the sheet piles fail to cut through the Marsh deposit. The cross section in Figure 20 shows another waterside excavation; this time narrower than the one found in the North section, but at the same depth to the bottom and backfill material. Levee slopes are one vertical to three horizontal.

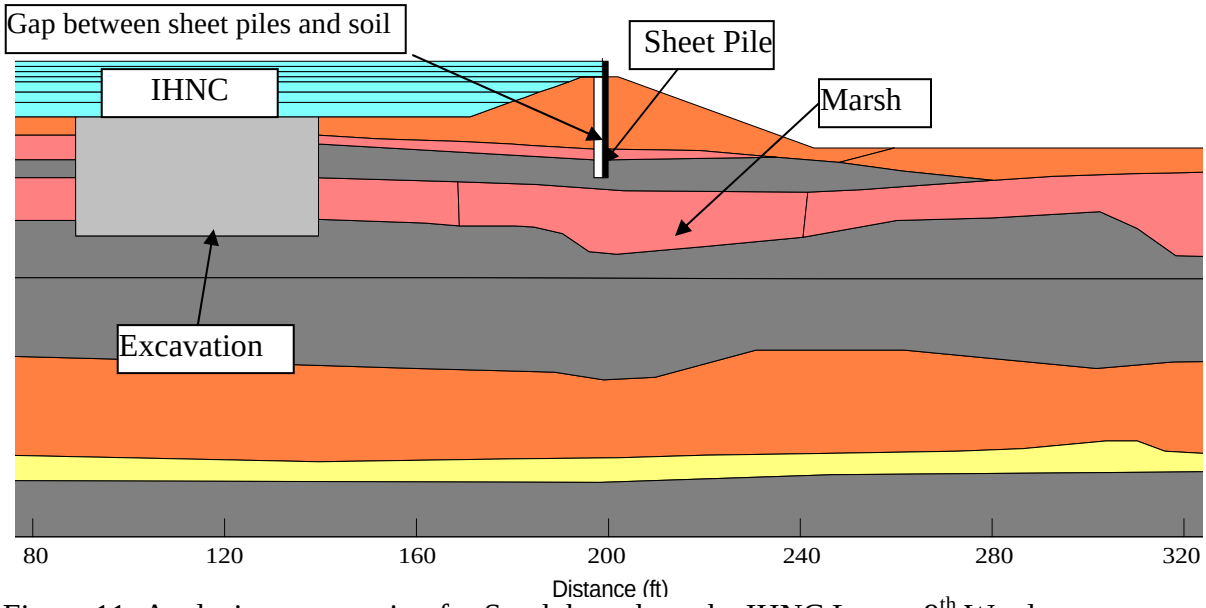


Figure 11: Analysis cross-section for South breach at the IHNC Lower 9th Ward

2D Seepage Analyses

Transient seepage analyses were performed for the rising water level in the IHNC according to the measured surge elevation hydrographs in this area (Figure 3). The surge hydrograph was input into the seepage models which generated analytical results beginning at a water elevation +2 ft. Material properties used for this section were identical to those used for the north breach (Table 2).

The results of the seepage analyses (Figures 12 and 13) show that after 33 hours of surge development (August 28, 2005 12:00 am to August 29, 2005, 9:00 am), high pore pressures and hydraulic gradients developed along the Marsh deposit. Figures 14 and 15 summarize the results of the transient seepage analyses for the south breach section at 7:00 am on August 29, 2005, in terms of total heads and hydraulic gradients, respectively. Even though the waterside half of the levee embankment was not saturated, the phreatic surface on the landside did raise enough to saturate the shallow clay layers and produce high uplift pressures to cause potential instability.

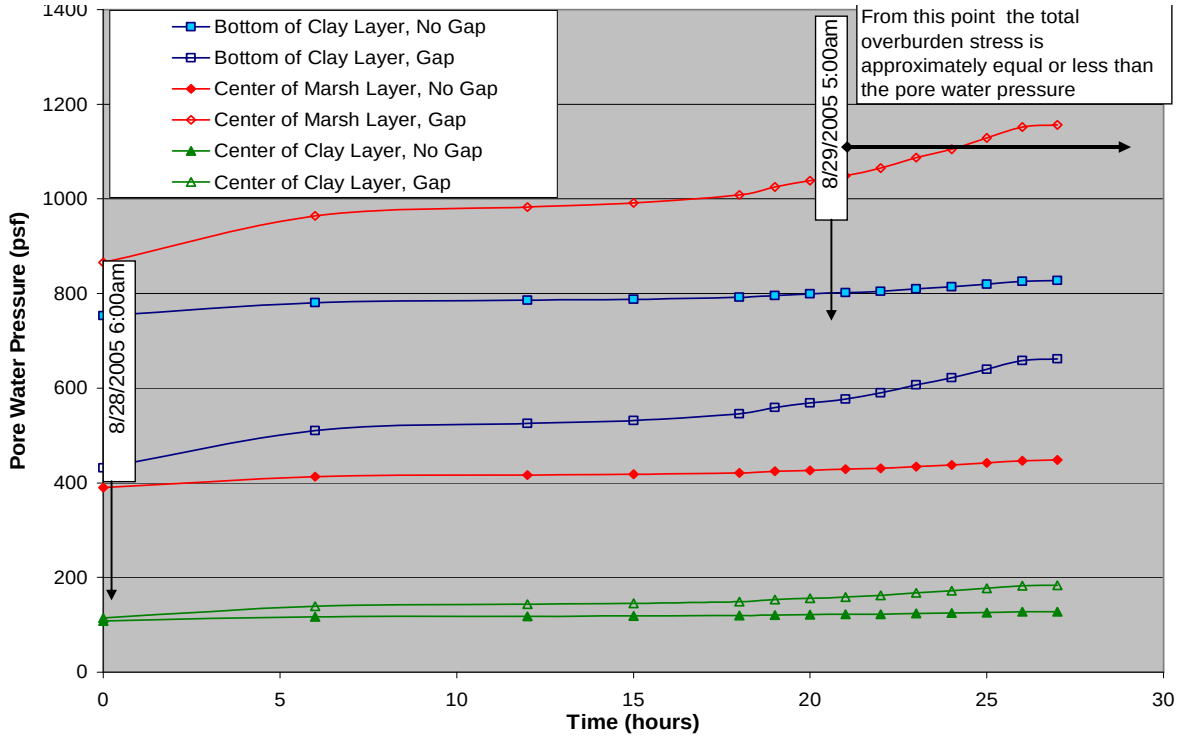


Figure 12: Pore pressure versus time near the toe of the levee for Marsh horizontal permeability of kx = 10^{-3} cm/s

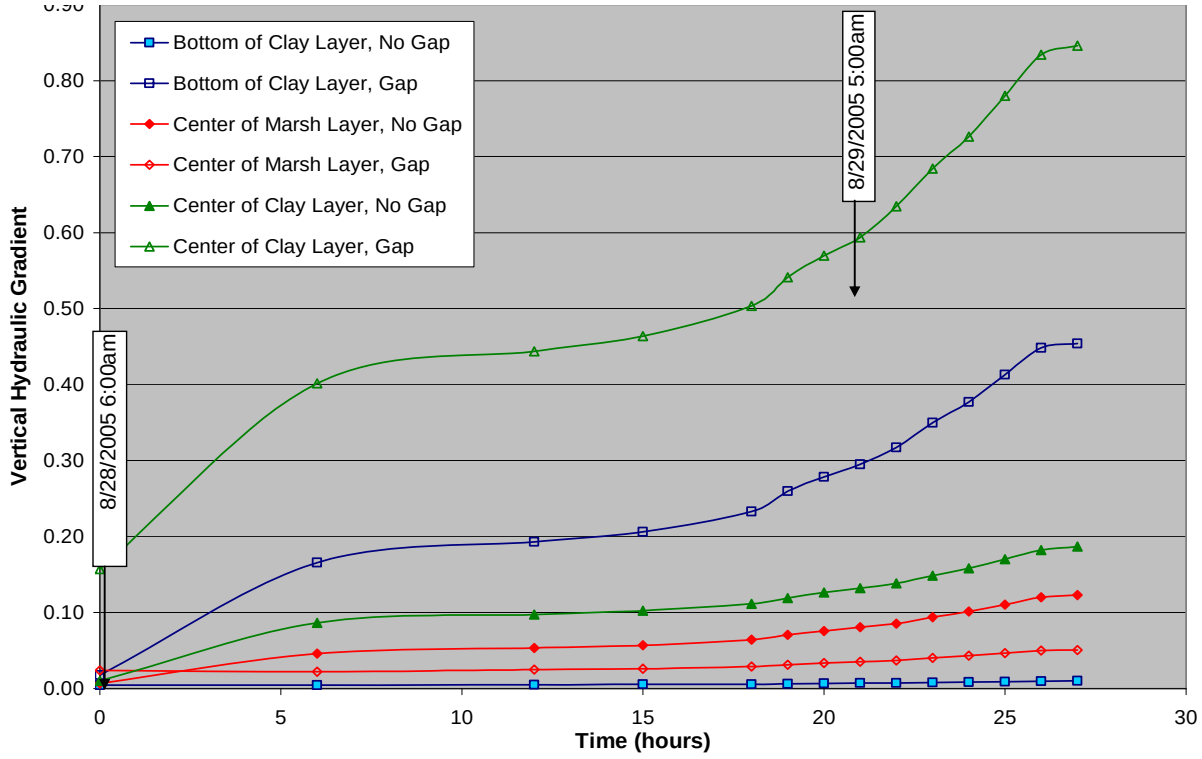


Figure 13: Vertical hydraulic gradients near the toe of the levee for Marsh horizontal permeability of kx = 10^{-3} cm/s

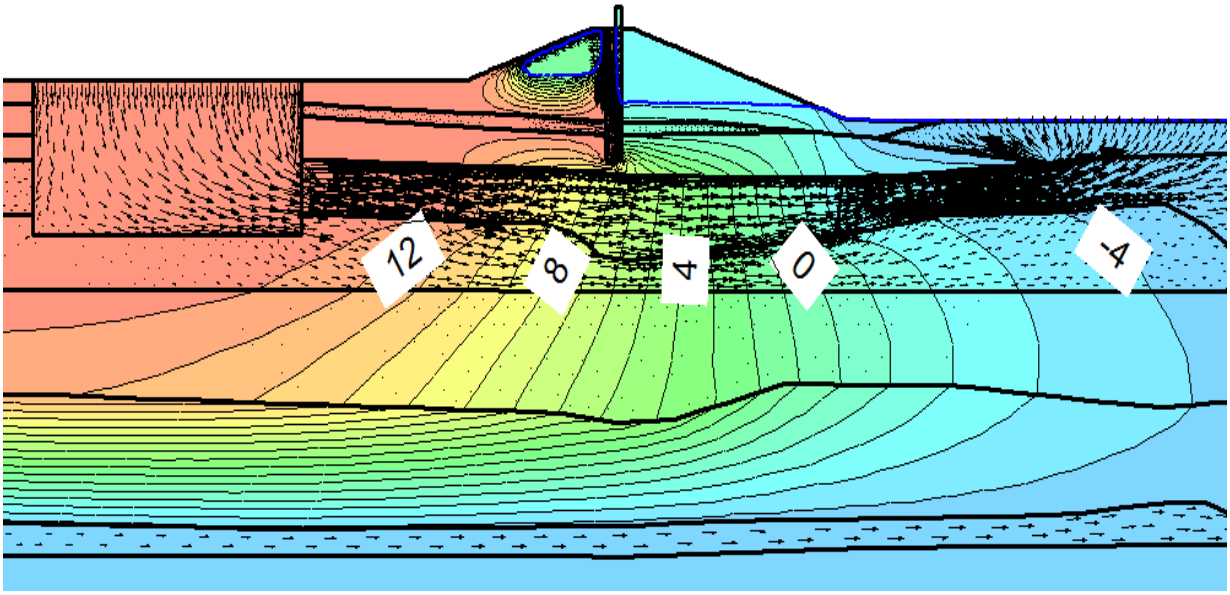


Figure 14: Transient seepage analysis results. Total Heads in ft (1 ft = 0.3 m). Time: between 7:00 and 8:00 am on August 29, 2005.

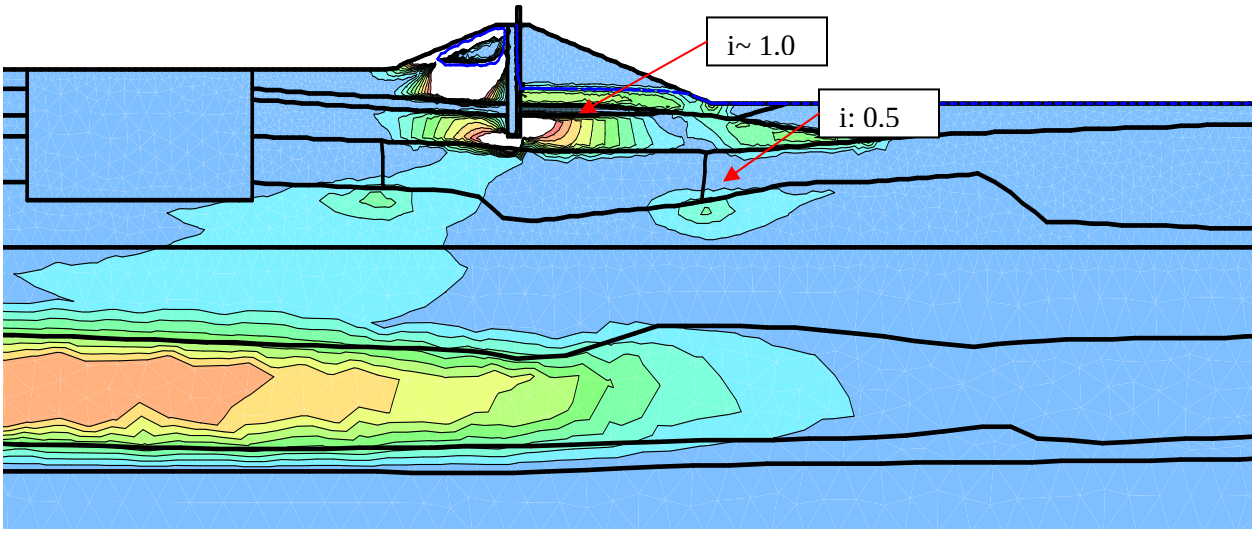


Figure 14: Hydraulic gradients from transient seepage analysis (contour interval = 0.1). Time: between 7:00 and 8:00 am on August 29, 2005. Maximum hydraulic gradient in the clay layer above the marsh deposit is approximately i = 1.0.

3D Seepage Analyses

As with the previous analyses, the cross section used for the 2D model was used as a starting point for the 3D analyses. The excavation was again modeled with vertical walls, approximately 60 ft away from the centerline of the levee. Figure 15 shows the 3D model used for the South breach.

Figure 16 shows the total head contours computed using the 3D model. The plot shows a similar distribution of heads as the 2D model, with minor losses in the soil layers upstream of the sheet pile located near the vicinity of the excavation. Figure 17 shows the computed hydraulic gradients concentrating several tens of feet landward of the toe. Figure 18 shows vertical gradient values versus time on the nodes beneath the landside toe, along two cross sections, or 'slices' of the 3D model, one through the filled excavation, and another one outside the excavated area. There is a 30 to 70% increase in gradients near the landside toe in the cross section through the filled excavation, compared to the section outside this area.

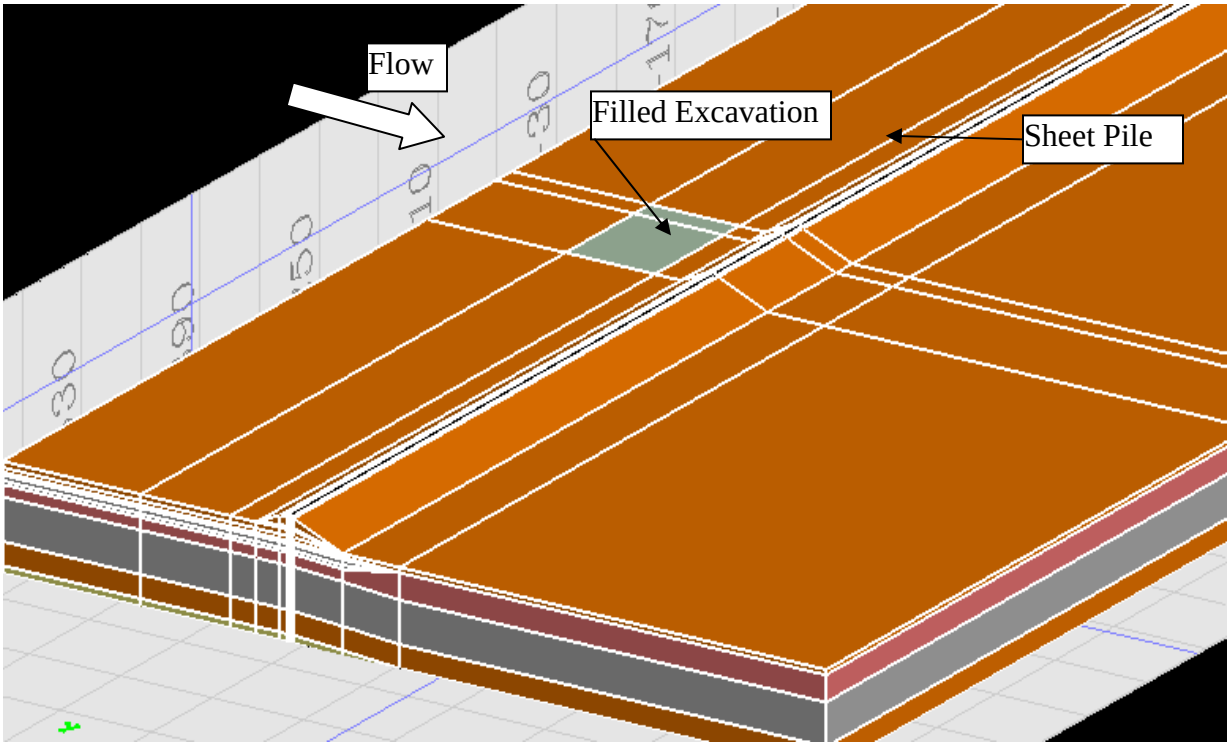


Figure 15: 3D seepage analysis model, IHNC-Lower Ninth Ward South breach

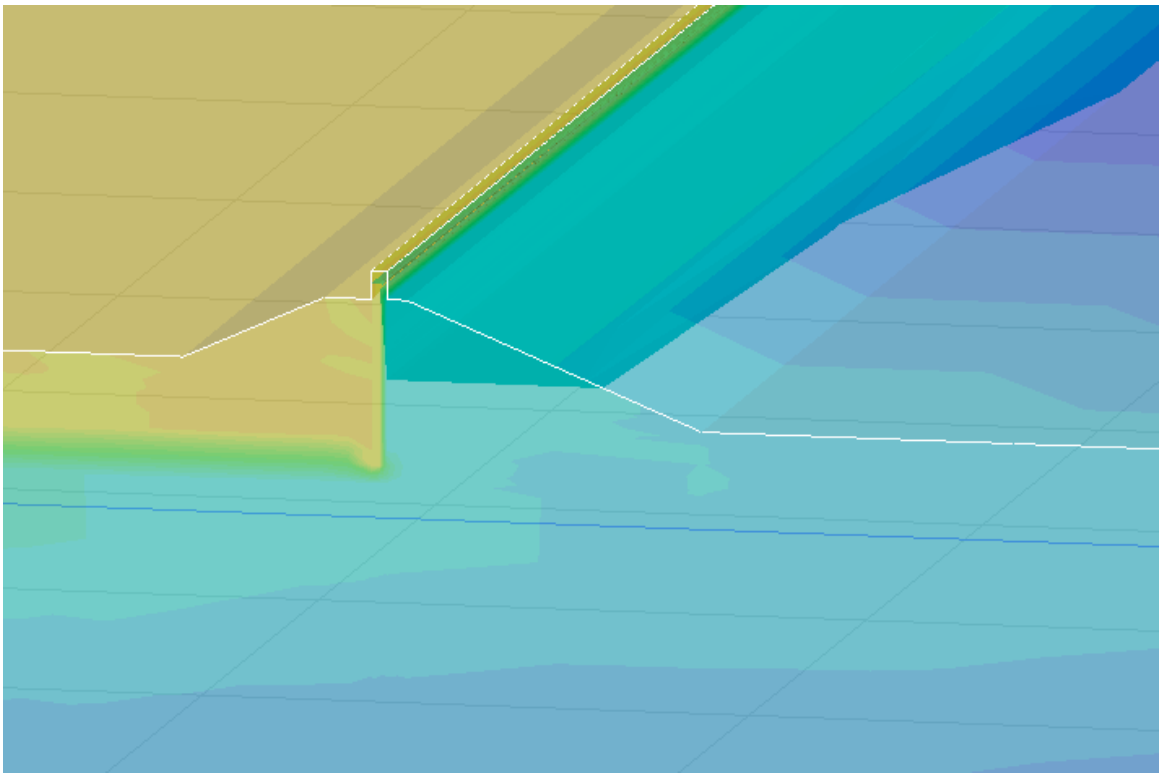


Figure 16: Total heads near landside toe for the south breach. Time: between 7:00 and 8:00 am on August 29, 2005

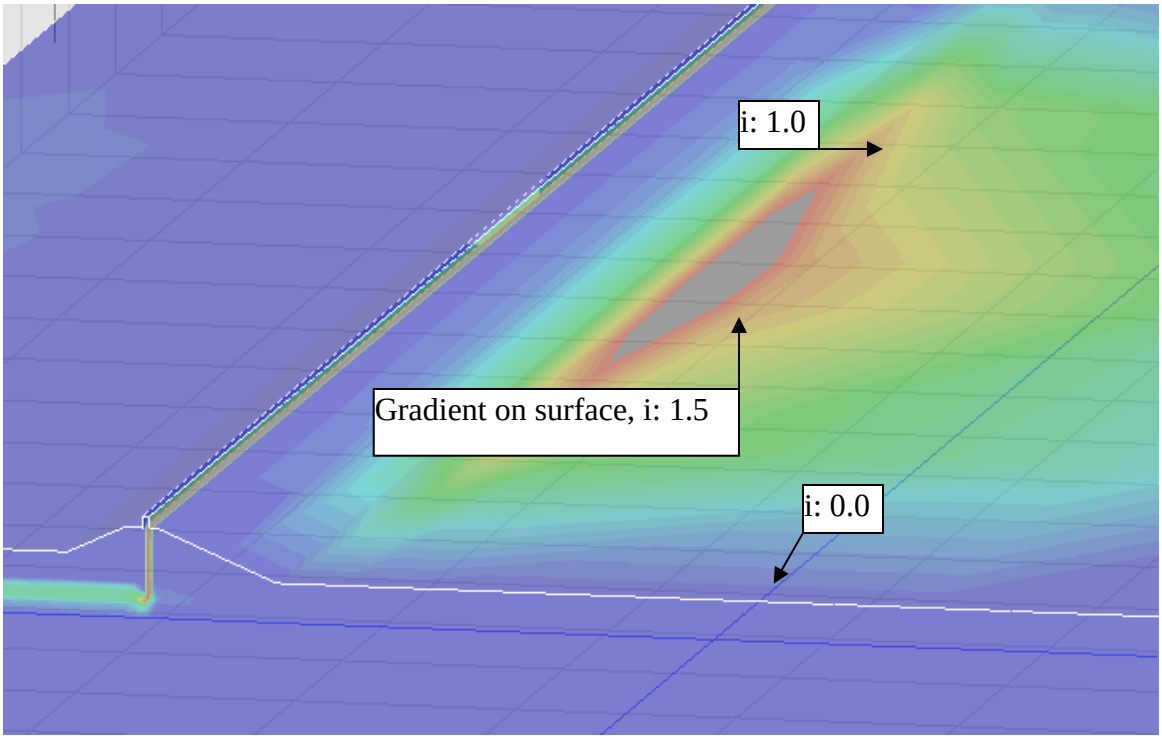


Figure 17: Hydraulic gradients near landside toe for the South breach. Time: between 7:00 and 8:00 am on August 29, 2005.

From the plot in Figure 18 it is clear that the hydraulic gradients computed by the 3D model are higher compared to adjacent areas where the waterside shallow layer is clay. The behavior of this section is different than the North Breach, showing lower gradients in both 2D and 3D models. This difference can be explained because the marsh layer is found below a clay layer which produces more head losses compared to the North Breach section. Also, the sheet pile is tipped inside the clay layer; augmenting the head losses and minimizing the amount of flow in the marsh layer coming from the gap between the sheet pile and fill.

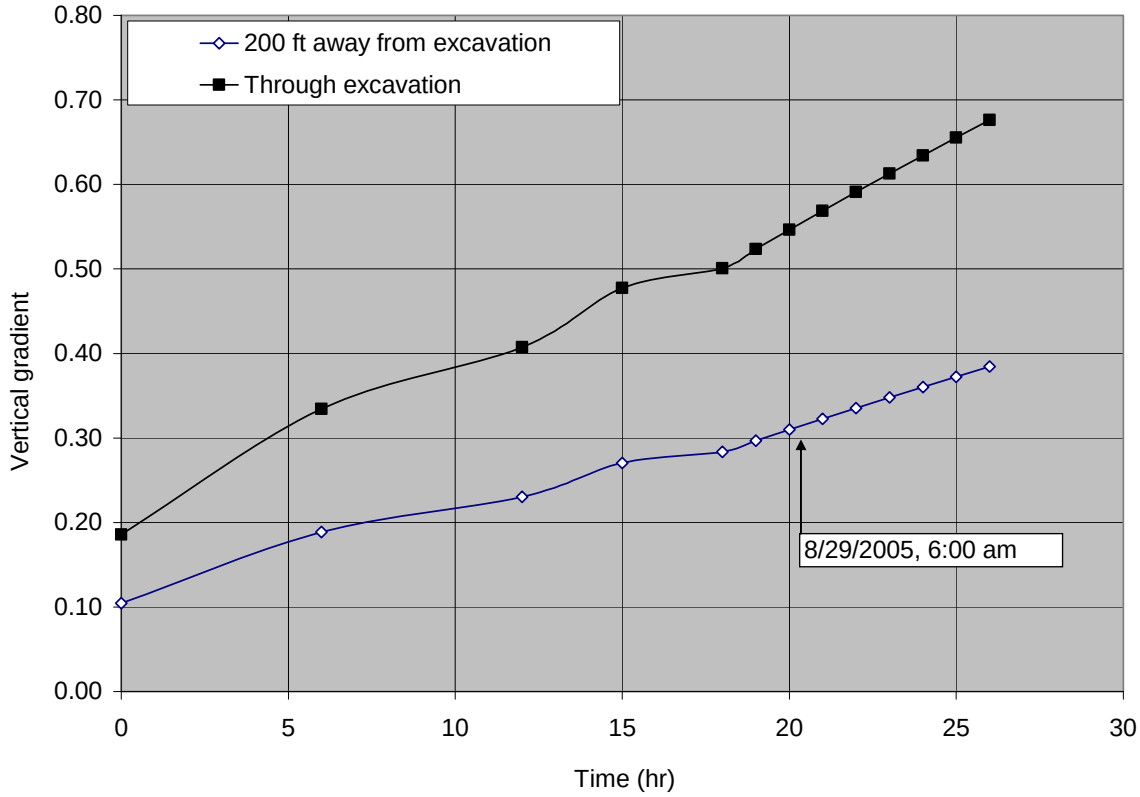


Figure 18: Gradients along two lines normal to the levee centerline, Time: 8:00 am on August 29, 2005.

Summary

Compared with the 2D seepage analysis models, the 3D seepage analysis models more accurately characterize the influences of localized excavations on development of the breaches at the Lower 9th Ward during Hurricane Katrina. Money (2006) performed similar analyses and found that vertical gradients increased between 50 and 100% when a narrow excavation was analyzed near the landside toe of an idealized levee. The difference in gradients, which is directly proportional to the increase in water pressures near the area affected, can be attributed to the fact that flow concentrates in the direction parallel to the levee centerline (Figure 19).

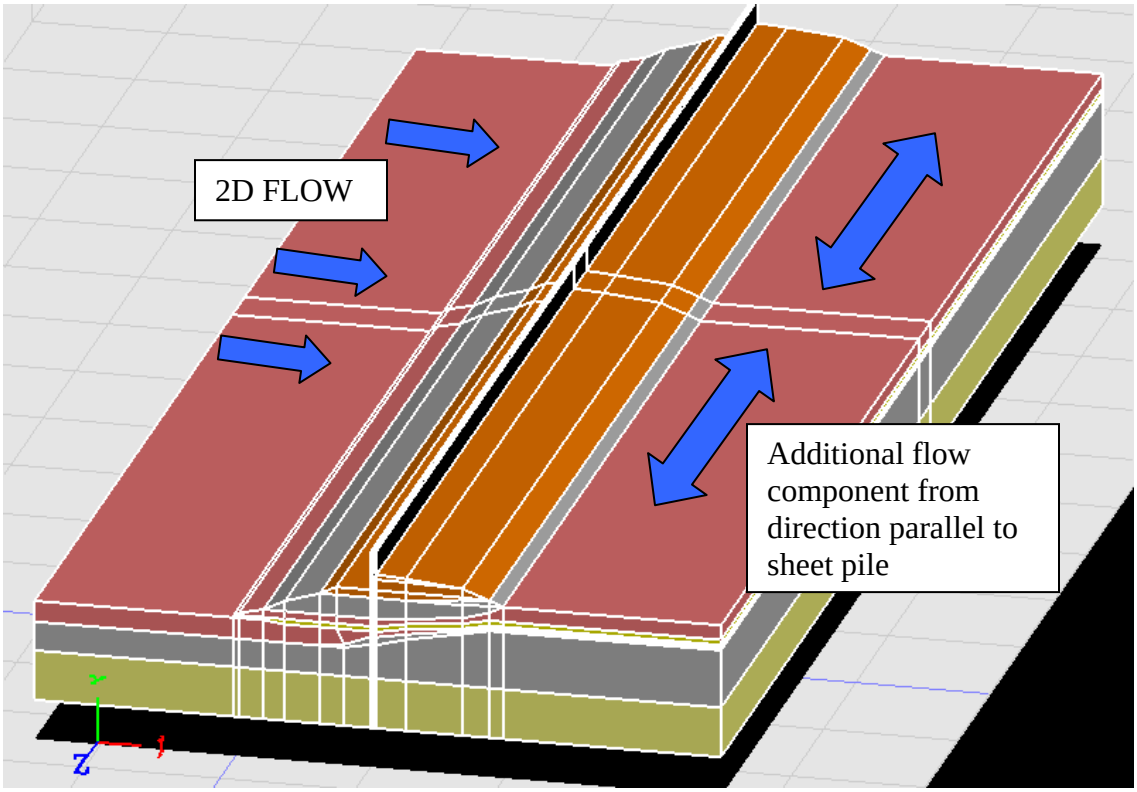


Figure 19: Contributions from additional flow component introduced by 3D seepage and hydraulic conductivity analyses.

The two IHNC-Lower 9th Ward models included the presence of waterside excavations filled with permeable materials near the levee toe. The plots of total head and gradients clearly show a concentration of flow near the landside toe, opposite to the waterside excavations. Computed internal hydraulic gradients on fine grained layers increased by 30 and 70% for the south and north breach sections, respectively. Increased gradients on the surface were also observed opposite to the waterside excavations.

Hydraulic blowout may have been the principal player in the development of the IHNC Lower 9th Ward North Breach section, which occurred before overtopping (storm surge between +9 and +10 ft or +2.8 and +3.1 m), approximately at 5:00 am on August 29, 2005. The South Breach at the IHNC Lower 90th Ward developed after overtopping, between 7:30 am to 8:00 am on the same day. The analyses summarized here further demonstrate that seepage and hydraulic uplift pressures played key roles in the initiation of these three failures.

Increased pore pressures, flow quantities and hydraulic gradients are the end result of analyzing three-dimensional effects of localized excavations near levee toes. For the two case histories analyzed, the effects were consistently negative, increasing gradients by 30 to 60% compared to 2D values. Table 3 summarizes the 2 and 3D gradients at the time of failure.

Table 3: Summary of results

Analysis Section	**Storm surge el. (ft, NAVD88)**	**2D vertical gradient**	**3D vertical gradient**
IHNC-North Breach	+9.0	1.0 – 1.5	1.5 - 2.0
IHNC-South Breach	+12.0	0.6	0.7 (internal) – 1.5 (shallow)

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