Soil Compressibility in Transient Unsaturated Seepage Analyses

Timothy D. Stark[1], Navid H. Jafari[2], Aaron L. Leopold[2], and Thomas L. Brandon[3]

[1]Professor, Dept. of Civil and Environmental Engineering, University of Illinois, 205 N. Mathews Ave., Urbana, IL 61801-2352; email: tstark@illinois.edu
[2]Doctoral Candidate, Dept. of Civil and Environmental Engineering, University of Illinois, 205 N. Mathews Ave., Urbana, IL 61801-2352; email: njafari2@illinois.edu
[3]Graduate Research Assistant, Dept. of Civil and Environmental Engineering, University of Illinois, 205 N. Mathews Ave., Urbana, IL 61801-2352; email: leopold3@illinois.edu
[4]Associate Professor, Dept. of Civil and Environmental Engineering, Virginia Tech, 22 Patton Hall, Blacksburg, VA 24061, tbrandon@vt.edu

ABSTRACT: Most levee underseepage and uplift analyses are based on steady-state seepage and can yield conservative results. Although computations are simpler and steady-state seepage parameters are easier to determine and readily available, transient and unsaturated seepage analyses are more representative of levee seepage conditions. Transient seepage analyses using a case study floodwall indicate that as soil compressibility of the underseepage layer decreases, rapid landside pore-water pressures increase and can approach steady-state values. The transient results for this case study also indicate that uplift factor safety during the flood event are about 25% higher than steady-state. The effect of soil compressibility can delay or accelerate the onset of uplift water pressure increase from the initial steady-state conditions.

INTRODUCTION

Over 100,000 miles of flood protection infrastructure are currently operating in the United States, e.g., along Mississippi, Sacramento, Trinity, Missouri, and American Rivers. Current performance of urban levees and floodwalls to hurricane and flood events are primarily based on steady-state seepage analyses. By assuming steady-state seepage, the U.S. Army Corps of Engineers (USACE) design manuals EM 1110-2-1901 and EM 1110-2-1913 (USACE 1993, 2000) represent the state-of-practice for evaluating landside hydraulic gradients and uplift pressures from levee underseepage. Hurricane and flood conditions typically only act for a period of hours to weeks, which may not allow sufficient time to develop steady-state conditions (Peter 1982). As a result, a transient seepage analysis, e.g., wetting front or other movement of water in unsaturated soil, provides a more realistic approach to evaluating levee seepage and slope stability, especially in failure causation analyses (Lam et al. 1987;

Li and Desai 1983; Lane and Griffiths 2000).

During transient conditions, changes in hydraulic boundary conditions and boundary total stresses cause (i) saturated seepage through relatively pervious foundation strata, (ii) unsaturated seepage through earth embankments, and (iii) shear-induced pore-water pressures resulting from changes in boundary total stresses, e.g., flood water or storm. The first change (saturated seepage) depends on the saturated horizontal hydraulic conductivity (k_h), hydraulic conductivity anisotropy (ratio of vertical k to horizontal k or k_v/k_h), and coefficient of volume compressibility (m_v; hereafter referred to as soil compressibility) of embankment and foundation strata through which underseepage will occur. The second mechanism (unsaturated seepage) relies on unsaturated hydraulic conductivity function and soil-water characteristic curves and causes delays in seepage and propagation of pore-water pressures. This effect is less influential for the floodwall case study presented herein because seepage occurs in the initially saturated foundation strata and hence is not discussed further. In the third mechanism (total stresses), positive pore-water pressures resulting from changes in shear and normal stresses are not calculated in a transient analysis. For example, shear-induced pore-water pressures from sudden drawdown after a prolonged flood stage can result in the upstream slope becoming unstable. This problem requires a coupled analysis of flow and stress deformation analysis using a constitutive soil model. The transient seepage analyses performed using commercial finite element analysis (FEA) pro-grams are uncoupled from changes in total stress. Therefore, shear and normal stress induced pore-water pressures are not considered in this paper.

The impact of steady-state conditions and foundation underseepage on earth structures is well-documented (e.g., Cedergren 1989; Daniel 1985; Gabr et al. 1996; Turnbull and Mansur 1961; Mansur and Kaufman 1957). However, the influence of soil compressibility on underseepage and uplift pressures is less understood by geotechnical engineers. Browzin (1961), Brahma and Harr (1962), and Desai and Sherman (1971) present transient seepage analyses but the models do not consider m_v and assume only saturated flow. As a result, this paper demonstrates the importance of soil compressibility in transient seepage analyses, presents methods for evaluating and selecting compatible values of m_v, and provides recommendations for performing transient unsaturated seepage analyses for levees and floodwalls. This paper uses a calibrated floodwall case study to show the influence of m_v on landside uplift pressures during flood and hurricane events.

SEEPAGE THEORY

Lambe and Whitman (1969) define transient flow as the condition during water flow where pore-water pressure, and thus total head, changes with time. The equation for 3-D transient flow through a saturated anisotropic porous medium is:

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial h_t}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial h_t}{\partial y}\right) + \frac{\partial}{\partial z}\left(k_z \frac{\partial h_t}{\partial z}\right) = S_s \frac{\partial h_t}{\partial t} \quad (1)$$

where k is the hydraulic conductivity in the x, y, and z directions, h_t is the total

hydraulic head, t is time, and S_s is the specific storage. Specific storage is expressed as $S_s = \gamma_w(m_v+n\beta)$, where γ_w is the unit weight of water, n is porosity, and β is compressibility of water. Because water is incompressible ($4.7x10^{-7}$ kPa^{-1}) for seepage purposes, specific storage reduces to $S_s = \gamma_w m_v$. Time is introduced in the seepage analysis via the right-hand side (RHS) of Eq. (1). For a unit decline in total hydraulic head, the RHS is directly related to the magnitude of m_v. If an incompressible value of m_v of $1x10^{-7}$ kPa^{-1} is assumed, the RHS approaches zero, which corresponds to a steady-state seepage condition shown in Eq. (2). Consequently, the steady-state seepage analysis becomes independent of time and generates landside pore-water pressures and gradients significantly higher, e.g., equivalent to a steady-state analysis, by only decreasing the parameter m_v. Therefore, applying reasonable values of m_v for the saturated seepage layer is important and determines if pore-water pressures, and thus gradients, can be rapidly transmitted to the landside.

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial h_t}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial h_t}{\partial y}\right) + \frac{\partial}{\partial z}\left(k_z \frac{\partial h_t}{\partial z}\right) = 0 \qquad (2)$$

Table 1 summarizes m_v values for soils and rocks, which generally range from $1x10^{-3}$ to $1x10^{-8}$ kPa^{-1}, respectively. The compressibility of sound rock as well as sandy gravel are similar to water and thus assumed to be incompressible. Representative m_v values for fine-grained soils, e.g., soft organic clays and peats to stiffer over-consolidated clays and tills, are also provided in Table 1. The compressibility of soils found near floodplains, i.e., normally consolidated alluvial clays given in Table 1, fall within a range of $3x10^{-4}$ to $1.5x10^{-3}$ kPa^{-1}. This narrow range of m_v influences pore-water pressure transmission through saturated foundation strata and is investigated herein using a floodwall case study.

Table 1. m_v for various soils (after Domenico and Mifflin 1965 and Bell 2000)

Material	m_v (kPa^{-1})
Organic alluvial clays and peats	$\geq 1.5x10^{-3}$
Normally consolidated alluvial clays	$3x10^{-4}$ to $1.5\ x10^{-3}$
Varved and laminated clays, firm to stiff clays	$1x10^{-4}$ to $3x10^{-4}$
Very stiff or hard clays, tills	$5x10^{-5}$ to $1x10^{-4}$
Heavily over consolidated tills	$\leq 5x10^{-5}$
Loose sand	$1x10^{-4}$ to $5.2x10^{-5}$
Dense sand	$2.1x10^{-5}$ to $1.3x10^{-5}$
Dense sandy gravel	$1x10^{-5}$ to $5.2x10^{-6}$
Sound and jointed rock	$\leq 6.9x10^{-6}$
Water (β)	$4.7x10^{-7}$

ESTIMATING SOIL COMPRESSIBILITY

Soil compressibility describes the strain induced under an applied effective vertical stress (σ'_v) and is related to the constrained modulus (D) of the soil, see Eq. (3). The m_v value can be determined from constant rate of strain (ASTM D4186) or

incremental loading (ASTM D2435) one-dimensional (1-D) consolidation tests.

$$m_v = \frac{\Delta\varepsilon_v}{\Delta\sigma'_v} = \frac{1}{D} \quad (3)$$

Soil compressibility can also be expressed in terms of σ'_v, initial void ratio (e_o), and slope of the e-log σ'_v relationship, i.e., the compression index, C_c, as shown in Eq. (4). Values of C_c and e_o can be easily determined from results of 1-D consolidation tests on high quality specimens.

$$m_v = \frac{0.434 C_c}{(1 + e_o)\sigma'_v} \quad (4)$$

Figures 1(a) and 1(b) provide m_v values for three fine-grained soils located in the Inner Harbor Navigation Canal (IHNC) along the Lower Ninth Ward in New Orleans, Louisiana. These values were computed from incremental load and constant rate of strain 1-D consolidation tests conducted on large diameter (125 mm) fixed-piston samples. Figure 1(a) illustrates the general trend of m_v with increasing natural water content (w_o), and Figure 1(b) shows compressibility as a function of in-situ σ'_{vo}. Values of m_v range from $2x10^{-4}$ to $2x10^{-3}$ kPa^{-1} for the saturated inorganic soils and $7x10^{-4}$ to $3.5x10^{-3}$ kPa^{-1} for saturated organic clays located along the IHNC. The levee fill along the IHNC and Interdistributary (ID) clay underlying the IHNC floodwall exhibit a similar range of m_v because the levee fill material was generated from dredging of the organic and ID clays to create the IHNC. The material was spread along the side of the canal to create the original levee and subsequent floodwall. The data in Figure 1 illustrate the variability of m_v and uncertainty in selecting an appropriate value for seepage analyses. As a result, a range of m_v values should be used in transient and unsaturated seepage analyses as discussed below.

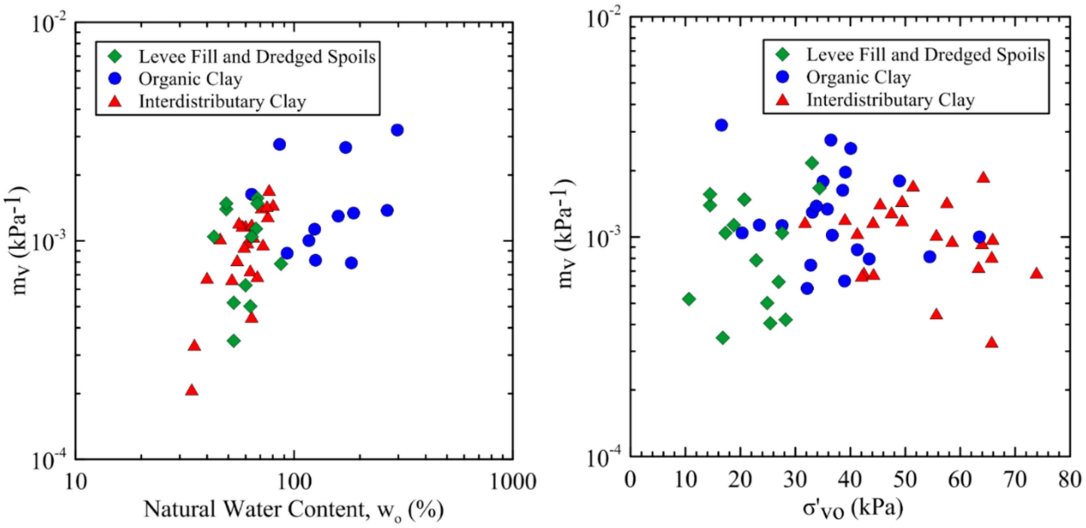

FIG 1. m_v of clayey soils in terms of: (a) natural water content (b) effective vertical stress from laboratory consolidation tests (data from Fugro 2012)

If a planning level seepage analysis is desired, m_v can be estimated by obtaining an estimate of compression index (C_c) from an empirical correlation with in-situ water content. For example, Figure 2 shows compression index C_c of IHNC soils as a function of in situ water content. A direct relationship between the compression index and in-situ water content exists in Figure 2 because both are controlled by soil composition and structure (Terzaghi et al. 1996). The IHNC organic clay deposits come to equilibrium at water contents of 100 to 400% and display values of C_c typically in the range of 1 to 5 because a large amount of water is held within and among the organic particles (Mesri and Aljouni 2007). The levee fill and ID clay (inorganic clays and silts) exhibit typical in-situ water contents below 100 and C_c below 1. In the absence of laboratory testing on high quality samples, Figure 2 provides an empirical correlation between C_c and w_o for IHNC soils, i.e., $C_c = w_o/100$, which is also reported in Terzaghi et al. (1996).

Another method to evaluate m_v is to calibrate a transient seepage model with vibrating wire (VW) piezometric data during hurricane events, storm surges, or floods. To replicate piezometer readings during a storm or flood event, both k_h and m_v can be adjusted within reasonable ranges until agreement between the piezometer response and transient seepage model was achieved.

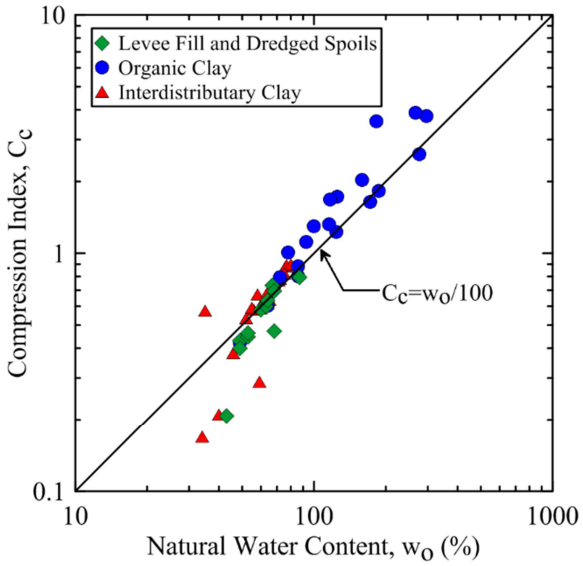

FIG 2. Correlation between compression index and in situ water content for IHNC soils (data from Fugro 2012)

CASE HISTORY AND PARAMETRIC ANALYSIS

Hurricane Katrina was a Category 3 hurricane when it made landfall on the Gulf Coast in 2005 with maximum sustained surface winds of 282 kilometers/hour (kph; 165 mph) (IPET 2007). This storm surge severely loaded and overtopped portions of the IHNC floodwall wall that was constructed to protect the adjacent Lower Ninth Ward. This storm surge contributed to two failures of the eastern portion of the floodwall along Jourdan Avenue in the Lower Ninth Ward: (1) the north breach

located at the north end of the Lower Ninth Ward and directly south of the Florida Avenue Bridge and (2) the south breach located north of the Claiborne Avenue Bridge near the middle of the Lower Ninth Ward.

The floodwall shown in Figure 3 is located immediately south of the north breach and is denoted the "no-failure section" herein because this area of the floodwall did not fail during Hurricane Katrina. The large borrow pit excavation in Figure 3 filled with water prior to Hurricane Katrina, was previously excavated to acquire suitable backfill material for the various excavations created during environmental restoration of the eastern side of the IHNC. These excavations were created to remove contaminated soils, building foundations, utilities, and other infrastructure from abandoned industrial activities to allow future expansion of the IHNC. Topographic surveys (WGI 2001; Wink 2005) show the borrow pit is about 23 m (75 ft) from the floodwall and has a 3H:1V sideslope on the floodwall side of the excavation (see Figure 4). The maximum depth of the borrow pit is 3.5 m, i.e., elevation -3.5 m NAVD88, which is about 0.3 m (1 ft) lower than the tip of the floodwall sheet pile (elevation -3.2 m NAVD88) under the concrete portion of the floodwall. The cross-section in Figure 4 depicts the sheet pile-supported floodwall and three fine-grained soil layers located along the IHNC. The cross-section is located in the area between the north and south floodwall breaches and through the soil borrow pit used to backfill floodside excavations.

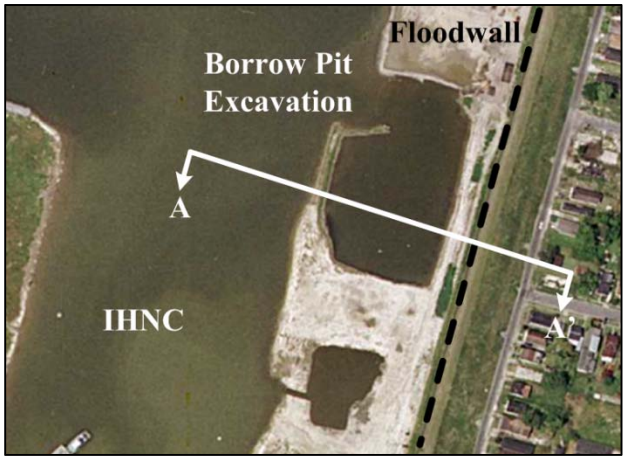

FIG 3. Aerial view of large water filled soil borrow pit excavation within 23 m of the floodwall prior to Hurricane Katrina between the North and South Breaches [(29°58'41.50"N, 90°01'14.14"W); photo from GCAM 2005)]

The levee fill and dredged spoils are comprised of dredged organic and ID clays from creation of the IHNC that were used to construct the levee that parallels the east side of the IHNC. The fill material consists of a heterogeneous mixture of gray, soft to stiff lean clay, silt, silty sand, and shell fragments. The levee fill exhibits a w_o, liquid limit (LL), and plasticity index (PI) of 43–68%, 73–103%, and 21–76%, respectively, based on an extensive subsurface investigation conducted in 2011 (see Table 2). The organic clay underlying the levee fill and dredged spoils is a soft, gray to dark gray soil formed in this deltaic environment. The organic content ranges from 2 to 62%, predominantly consisting of roots and pieces of wood. The organic clay w_o, LL, and

PI values range from 57–296%, 70–309%, and 46–196%, respectively. The ID clay consists of gray to dark gray, medium to soft clay with lenses of silty sand and silt and medium lean clay. The ID clay w_o, LL, and PI range from 34–80%, 33–97%, and 16–72%, respectively. Table 2 summarizes the soil classification, saturated unit weight (γ_{sat}), w_o, LL, and PI values of these three soil layers that are used in subsequent transient seepage analyses.

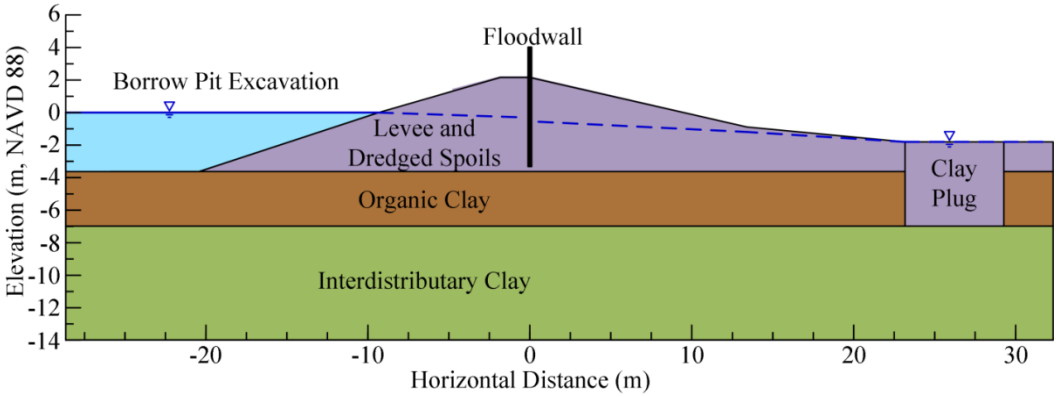

FIG 4. Floodwall cross-section through borrow pit used for parametric seepage analysis that shows sheet pile supported floodwall, initial phreatic surface, and a landside backfilled excavation

Table 2. Engineering index properties for soils (data from Fugro 2012)

Soil Type and Classification		γ_{sat} (kN/m^3)	w_o (%)	LL (%)	PI (%)
Levee Fill and Dredged Spoils	CH	16.3	43-68	73-103	21-76
Organic Clay	OH	9.9 – 13.7	110-451	166	111
Interdistributary (ID) Clay	CH	18.0	34-80	33-97	16-72

The derivation of transient seepage flow in Eq. (1) indicates that m_v can influence the time required to reach increased pore-water pressures, which increases generation of seepage forces and uplift at the landside levee toe. Thus, the no-failure case study is used herein to establish the role of soil m_v on landside pore-water pressure by performing a parametric analysis using k_h, m_v, and levee geometry. The parametric analyses use uplift factor of safety (FS) – the ratio of total stress and pore-water pressure at the top of organic clay layer to evaluate floodwall performance. This site consists mainly of fine-grained soils so there is limited potential for uplift failure at the toe. However, the procedure used to calculate the uplift pressures is instructive for other sites containing fine-grained soils overlying pervious foundations.

TRANSIENT MODEL

The two-dimensional SEEP/W finite element program (Geo-Slope International 2007) was used to model groundwater conditions and underseepage along the IHNC, including the cross-section in Figure 4. SEEP/W is a general seepage analysis

program formulated to model saturated and unsaturated transient flow through soil and excess pore-water pressure dissipation estimated from a stress-deformation analysis within porous materials. SEEP/W can model different hydraulic conductivities, water contents, and changes in water content as a function of pore-water pressure.

Figure 4 shows the floodwall system in the no-failure section consists of a reinforced concrete floodwall and a supporting sheet pile extending to an elevation of -3.2 m NAVD88. The sheet pile impedes seepage through the levee fill and dredged spoils so the focus of the seepage analysis is flow through the organic clay layer below the sheet pile tip. The soil borrow pit or deep excavation is modeled 23 m from the floodwall. A clay plug on the landside is modeled about 25 m from the floodwall. The clay plug represents an excavation or ditch on the landside along Jourdan Avenue that was retrofitted with a reinforced concrete storm water box culvert to bury Jourdan Avenue Canal prior to Hurricane Katrina. This concrete box culvert parallels the floodwall in the Lower Ninth Ward along the length of the IHNC and was backfilled with clayey soil.

The transient model was calibrated using the 2011 Tropical Storm Lee hydrograph and field data from four landside VW piezometers located 7.6 m (25 ft) landside from the floodwall. The piezometers were installed at elevations of -2.3 m, -3.8 m, -4.7 m, and -8.4 m NAVD88. Rainfall infiltrating and saturating the levee fill caused pore-water pressures to increase at piezometers of el. -2.3 m, -3.8 m, and -4.7 m NAVD88. However, the piezometer at el. -8.4 m NAVD88 (located in the ID clay) did not measure pore-water pressure increase, which corroborates that low hydraulic conductivity and high compressibility of the organic clay does not instantaneously transmit floodside pore-water pressure to the landside toe. As a result, the calibrated transient model is used to perform a parametric analysis using k_h, m_v, and levee geometry.

Seepage Soil Properties

In a saturated transient seepage analysis, three soil properties are required: (1) saturated k_h, (2) saturated k_v/k_h ratio, and (3) m_v. Table 3 provides the soil properties used for the parametric analysis of the no-failure section. The k_h and k_v/k_h were estimated from small (25.4 mm; 1 inch) and large (127 mm; 5 inch) diameter laboratory hydraulic conductivity tests (ASTM D5048) that were trimmed in horizontal and vertical orientations to estimate hydraulic conductivity anisotropy. The organic clay k_h was found to range between 10^{-6} to 10^{-8} cm/s with values of 10^{-5} cm/s because of higher organic content. As a result, parametric analyses were performed using values of k_h in the 10^{-5} cm/s range. The values of m_v were computed from 1-D consolidation test results shown in Figure 1. For each soil, average values of m_v were selected. The parametric analyses also investigated soil variability by using the lowest and highest values of m_v from Figure 1. This represents upper and lower bounds of m_v that can influence landside uplift pressures. Observations during borrow pit excavation, numerous soil borings, cone penetration soundings, vane shear tests, and other evidence (IPET 2007; ILIT 2006) show the native soils in the borrow pit area are representative of the IHNC soils summarized in Tables 2 and 3.

Table 3. Soil parameters for transient seepage analyses (data from Fugro 2012)

Soil Type	k_h (cm/s)	k_v/k_h	m_v (kPa^{-1})		
			Lowest	Average	Highest
Levee Fill	$1x10^{-6}$	0.83	$3x10^{-4}$	$8x10^{-4}$	$2x10^{-3}$
Organic Clay	ranges	0.33	$5x10^{-4}$	$1.2x10^{-3}$	$3x10^{-3}$
Interdistributary Clay	$3x10^{-7}$	0.50	$3x10^{-4}$	$8x10^{-4}$	$2x10^{-3}$

Model Boundary Conditions

For a transient analysis, it is essential to define the initial groundwater conditions. The initial floodside steady-state boundary condition is assumed to be a total head (h_t) boundary condition of elevation +0 m NAVD88, which represents the canal water level inside the borrow pit excavation before the 2005 hurricane. This is consistent with canal water levels measured prior to Hurricane Katrina. The initial landside steady-state boundary condition is assumed to be 0.3 m (1 ft) below the ground surface based on borings and excavations in 2011 and the Florida Avenue Pump Station. The phreatic surface and total head loss across the sheet pile wall was estimated using a steady-state analysis and the floodside and landside boundary conditions described above. These steady-state results become the parent analysis, i.e., starting point, for the transient analyses.

The transient total head conditions applied to the floodside for this study are the 2005 Hurricane Katrina storm surge and the 2011 Mississippi River flood level (Figure 5). The data points shown in Figure 5 were used to model these hydrographs in SEEP/W by inputting sets of data points and modeled as a floodside total head boundary. In Figure 5(a), the hurricane surge increased from el. +1.4 m NAVD88 after 10 hours to elevation +4.3 m NAVD88 at 25 hours and then precipitously returned to elevation +0 m. Because the top of the floodwall is at el. +4 m NAVD88 (IPET 2007), the maximum surge level was decreased to correspond with the top elevation of the floodwall and thus prevent overtopping for the parametric analyses. The 2011 Mississippi River flood – obtained from USACE river gauge at Duncan Point, Louisiana, USA – represents a long-duration flood event and is used to understand the difference in seepage conditions between a rapid storm surge and a long-duration flood event. Elevations of the Mississippi River at Duncan Point are higher than the IHNC so the flood levels were normalized such that the peak flood elevation also corresponds to the top elevation of the floodwall or el. +4 m NAVD88. The flood level in Figure 5(b) reaches a first peak at El. +2.7 m NAVD88 after 65 days and then reaches the maximum flood level of El. +4 m NAVD88 after 100 days. For comparison purposes, the Hurricane Katrina storm surge is superimposed on the 2011 flood event in Figure 5(b). Figure 5(a) is a short-term event and Figure 5(b) is a long duration flood so both hydrographs represent scenarios that may be present in extreme events that can impact levee design and are used to illustrate the range of landside hydraulic response.

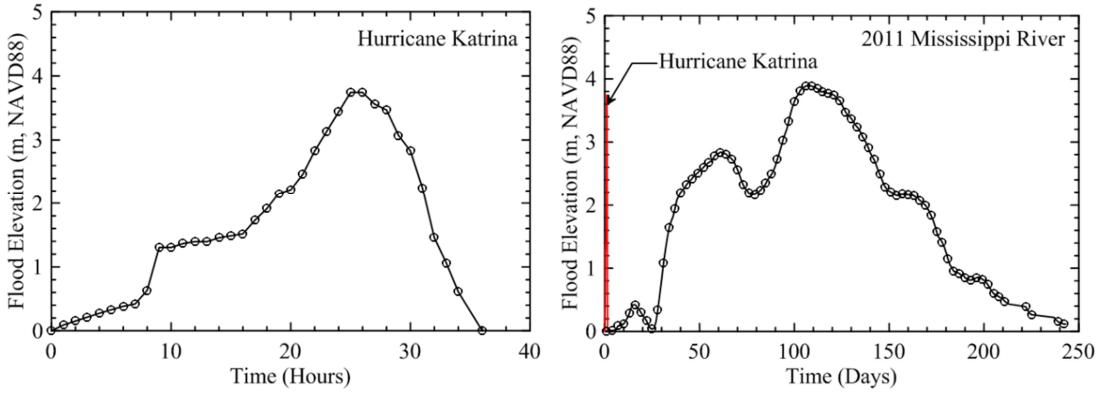

FIG 5. Hydrographs applied in parametric study (a) 2005 Hurricane Katrina at IHNC and (b) 2011 Mississippi River Flood at Duncan Point, LA

The SEEP/W program ‘‘potential seepage face review’’ option was selected for the ground surface from the floodwall to landside levee toe because the phreatic surface on the landside levee slope is unknown and the near surface soil is unsaturated based on landside borings and excavations. If the phreatic surface increases above the elevation of landside toe, then SEEP/W treats the slope face water flow as runoff. The landside boundary condition from the landside levee toe to the RHS of finite element mesh is zero pressure head (h_p). In SEEP/W, the h_p=0 boundary condition from the levee toe to the RHS of the mesh signifies that the groundwater level is at the ground surface, which is a reasonable assumption because of the normally high groundwater surface in the area coupled with the rainfall infiltration associated with the flood events. The Left-Hand Side (LHS) vertical boundary is characterized as zero flow, which occurs at a groundwater divide. The IHNC channel was considered a groundwater divide because of symmetry of the canal channel. The RHS vertical boundary is modeled as a total head boundary (h_t = -2.0 m or about equal to the ground surface) to represent the far-field groundwater conditions. Finally, the boundary condition along the bottom of the seepage model in Figure 4 is modeled as an impervious boundary due to the low hydraulic conductivity ID clay.

EFFECT OF COEFFICIENT OF VOLUME COMPRESSIBILITY

Figure 6 presents the factor of safety against uplift developed using parametric values of organic clay k_h, ranging from $1x10^{-5}$ to $6x10^{-5}$ cm/s, and values of m_v. The organic clay m_v values represent the average, highest, and lowest values of $1.2x10^{-3}$, $3x10^{-3}$, and $5x10^{-4}$ kPa^{-1}, respectively, in Figure 6. Transient seepage analyses performed using the Hurricane Katrina hydrograph exhibit almost no change in landside pore-water pressures and gradients because of the short duration of the hydrograph. Thus, the parametric analyses focus on the 2011 Mississippi River flood hydrograph. The results of varying saturated k_h are shown in Figure 6 and the values of m_v are used to illustrate the impact on seepage through the organic clay layer. For a given m_v value, increasing the saturated k_h increases the maximum uplift pressure. The increase in uplift pressure and effect of m_v are negligible for $k_h \leq 10^{-5}$ cm/s, i.e., seepage induced uplift pressures in the landside are generated when $k_h > 10^{-5}$ cm/s.

This shows that selecting a compatible m_v value for saturated fine-grained soils with $k_h > 10^{-5}$ cm/s is critical to develop a representative transient seepage analysis. If a compatible, i.e., realistic value of m_v is used, water must flow from the floodside to the landside of the floodwall to transmit the uplift pressures and seepage forces.

The first effect of m_v is that it can delay or accelerate the onset of uplift water pressure increase at the clay plug from the initial steady-state conditions. The time for underseepage to reach the clay plug for $m_v=3x10^{-3}$ kPa^{-1} (triangle symbol in Figure 6) is about 161 days. By decreasing the value of m_v to $1.2x10^{-3}$ kPa^{-1} and $5x10^{-4}$ kPa^{-1}, the time for underseepage to reach the clay plug decreases to about 83 days (circle symbol) and 62 days (square symbol), respectively. As a result, the time for underseepage to reach the clay plug decreases by a factor of about 2.5 times when m_v ranges from highest to lowest value and k_h remains constant. This behavior is reasonable because large m_v values increase the RHS of Eq. (1), i.e., total hydraulic heads induced by the flood dissipate at a faster rate. On the other hand, a low compressibility soil for the same time period allows less head to dissipate from the flood and thus results in uplift pressures developing quicker at the landside levee.

The second effect is that decreasing m_v increases maximum uplift pressures during the flood. For $k_h = 3x10^{-5}$ cm/s, the initial steady-state uplift FS is 1.18 and it decreases during the flood to 1.15 and 1.03 for m_v of $3x10^{-3}$ and $5x10^{-4}$ kPa^{-1}, respectively. When $k_h = 6x10^{-5}$ cm/s, FS decreases with time from a steady-state value of 1.08 to 0.97 and 0.85 for m_v of $3x10^{-3}$ and $5x10^{-4}$ kPa^{-1}, respectively, which indicates heave may develop according to USACE (2005). By decreasing the organic clay m_v from the highest to lowest value, i.e., $3x10^{-3}$ to $5x10^{-4}$ kPa^{-1}, FS is observed to decrease by about 10% for k_h of $3x10^{-5}$ cm/s and $6x10^{-5}$ cm/s. Based on this case study, the effect of m_v increases significantly as k_h increases to $1x10^{-4}$ cm/s. The results for $k_h=1x10^{-5}$ cm/s also indicate that vertical hydraulic gradients are unaffected, which is reasonable due to the low hydraulic conductivity of the organic clay.

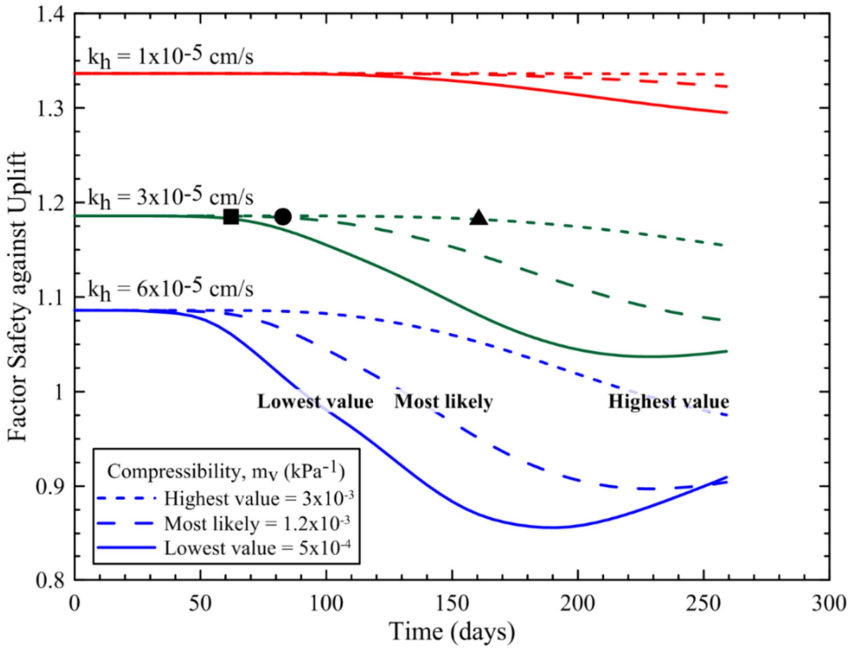

FIG 6. Effect of m_v on factor of safety against uplift at clay plug in Figure 4 using Mississippi River flood hydrograph for various k_h and m_v

Table 4 compares the FS against uplift at the clay plug for average m_v under transient conditions as well as assuming maximum flood surge at a steady-state condition. These gradients illustrate the influence of the saturated k_h of the organic clay layer on the vertical hydraulic gradients calculated a distance of 20 m from the floodwall. Table 4 also shows the FS for the steady-state condition are significantly lower (about 22%) and thus more conservative than the transient FS values. Steady-state conditions may not develop during rapid events, e.g., hurricanes, storms, and river floods, so transient unsaturated seepage analyses are recommended for design of seepage control measures to understand the impact of short duration hydrographs and values of m_v on the design of the remedial measures.

Table 4. Summary of vertical hydraulic gradients at clay plug using average of m_v and elevation +4 m (NAVD88) flood level for steady-state conditions and 2011 Mississippi River Flood hydrograph

k_h (cm/s)	Factor Safety against Uplift		
	Initial conditions (El. +0 m NAVD88)	Transient, average m_v (Figure 5(b))	Steady-state (El. +4m NAVD88)
$1x10^{-5}$	1.34	1.32	1.12
$3x10^{-5}$	1.18	1.08	0.84
$6x10^{-5}$	1.08	0.90	0.69

RECOMMENDATIONS FOR TRANSIENT SEEPAGE ANALYSES

The state of practice for levee design and remediation is performing a steady-state seepage analysis (USACE 1993, 2005). However, there is interest in performing transient unsaturated seepage analyses to calibrate transient seepage models with piezometric data and investigate the level of conservatism with a design based on steady-state conditions. As a result, the following procedure is recommended for such transient unsaturated seepage analyses:

1. **Initial steady-state conditions:** Before performing a transient analysis, the initial pore-water pressure regime near the levee must be determined. The floodside and landside groundwater surface before flooding or storm surge should be determined from subsurface information and used to establish the initial phreatic surface through the levee via a steady-state analysis. The initial groundwater conditions can be established via piezometer, boring, or excavation data.
2. **Selecting soil parameters:** For fine-grained, erodible soils such as silts and silty sands through which flow will occur, laboratory 1-D consolidation testing, C_c and w_o empirical correlation, and piezometer readings can be used to initially estimate m_v for transient seepage studies. Values of k_h and the anisotropy ratio of k_h/k_v should be measured using 127 mm (5 inch) laboratory permeameter tests (ASTM D5048) and high quality specimens. Values of k_v and m_v can be validated for

compatibility using Stark et al. (2014) and then used to determine if drained conditions control the accompanying stability analyses.

3. **Transient seepage:** The initial steady-state pore-water pressure regime is used as a "parent analysis" or starting point for the transient analysis. The appropriate floodside hydrograph should be used for the transient boundary conditions. The analysis is performed using the median or average value of m_v based on site-specific data. Additional analyses using the highest and lowest m_v values should be performed to develop upper and lower bounds, respectively, of the magnitude of uplift pressures and vertical hydraulic gradient. In addition, the location or zone of interest for the calculated uplift pressures and vertical gradients can be determined and compared with initial estimates, e.g., levee toe, to ensure reasonable responses and design measures.
4. **Underseepage and exit gradients:** Using a graph similar to Figure 6, the calculated uplift pressures or vertical gradients for a given k_h and m_v can be used to estimate the FS against uplift or heave, respectively. An exit hydraulic gradient of 0.85 calculated in the vertical direction on the landside of a levee is commonly considered sufficient to initiate heave and subsequent erosion, i.e., sand boils, in granular soils (USACE 2005). Other field measurements show that sand boils may occur with exit hydraulic gradients in the range of 0.54–1.02 (Daniel 1985). In cohesive soils, e.g., IHNC organic clay, sand boils will not develop but a gradient greater than 0.85 may be sufficient to cause some heave provided there is a pervious layer underlying the fine-grained layer.
5. **Seepage control measures:** Using steady-state analysis results for design of cutoff walls and/or relief wells is conservative (see Table 4), so the introduction of a transient analysis may preclude the use of a cutoff wall and only installation of relief wells.
6. **Slope stability:** To accurately model a transient condition in an effective stress stability analysis, a fully-coupled stress/seepage analysis is required. The contribution of changes in total stress and shear stress on the generated pore pressures in fine-grained soil have to be included in the analysis because these are not calculated in a transient seepage analysis. Unfortunately, this fully-coupled type of analysis is not common in geotechnical engineering practice (Alonso and Pinyol 2011). Transient analyses do provide guidance on when a conventional steady-state seepage stability analysis is warranted, or when a "rapid flood loading" analysis is required. Transient analyses can also provide useful information regarding pore-water pressures in pervious layers that can apply destabilizing uplift pressures to the layers above.

REFERENCES

Alonso, E.E., and Pinyol, N.M. (2011). "Landslides in reservoirs and dam operation." Dam Maintenance and Rehabilitation II, 3-27.

ASTM. (2010). "Standard test methods for measurement of hydraulic conductivity of saturated porous materials using a flexible wall permeameter (D5084)." In 2010 Annual Book of ASTM Standards, Volume 04.08. American Society for Testing and Materials, Philadelphia.

ASTM. (2011). "Standard test methods for one-dimensional consolidation properties of soils using incremental loading (D2435)." In 2011 Annual Book of ASTM Standards, Volume 04.08. American Society for Testing and Materials, Philadelphia.

ASTM. (2012). "Standard test method for one-dimensional consolidation properties of saturated cohesive soils using controlled-strain loading (D4186)." In 2012 Annual Book of ASTM Standards, Volume 04.08. American Society for Testing and Materials, Philadelphia.

Bell, F.G. (2000). Engineering properties of soils and rocks. 4th edition, Blackwell Science, UK.

Brahma, S.P., and Harr, M.E. (1962). "Transient development of the free surface in a homogeneous earth dam." Géotechnique, **12**: 283–302.

Browzin, B.S. (1961). "Non-steady flow in homogeneous earth dams after rapid drawdown." *In* Proceedings of the 5th International Conference on Soil Mechanics and Foundations Division, Paris, Vol. 2, pp. 551–554.

Cedergren, H.R. (1989). Seepage, drainage, and flow nets. John Wiley and Sons, NY.

Daniel, D.E. (1985). "Review of piezometric data for various ranges in the rock island district." USACE Waterways Experiment Station, U.S. Army Corps of Engineers, Vicksburg, MS.

Desai, C.S. and Sherman, W.C. (1971). "Unconfined transient seepage in sloping banks." Journal of Soil Mechanics and Foundations Division, 97(2): 357-373.

Domenico, P.A., and Mifflin, M.D. (1965). "Water from low-permeability sediments and land subsidence." Water Resources Research, American Geophysical Union, 1(4): 563-576.

Fugro (2012). Geotechnical Data Report, Inner Harbor Navigational Canal, East Bank Industrial Area (IHNC-EBIA), Field & Laboratory Testing Program. Final Report, New Orleans, Louisiana. Report No. 04.57114007-2.

Gabr, M.A., Wolff, T., Taylor, H. and Brizendine, A. (1986). "Underseepage analysis of levees on two-layer and three-layer foundation." Computers and Geotechnics, 18(2): 85-107.

Geo-Slope. 2007. Seep/W software Users Guide. Geo-Slope International Ltd., Calgary, Canada.

Independent Levee Investigation Team (ILIT). (2006). Investigation of the performance of the New Orleans regional flood protection systems during Hurricane Katrina. Final Report. Available from http://www.ce.berkeley.edu/new_orleans/ [accessed 31 July 2006].

Interagency Performance Evaluation Task Force (IPET) (2007). Performance evaluation of the New Orleans and southeast Louisiana hurricane protection system. Final Report of the Interagency Performance Evaluation Task Force, U.S. Army Corps of Engineers. Available from http://media.nola.com/hurricane_katrina/other/060106corps_vol7.pdf

Lane, P. and Griffiths, D. (2000). "Assessment of stability of slopes under drawdown conditions." Journal of Geotechnical and Geoenvironmental Engineering. 126(5): 443-450.

Lambe, T.W. and Whitman, R.V. 1969. Soil mechanics. John Wiley & Sons, N.Y.

Li, G.C. and Desai, C.S. 1983. Stress and seepage analysis of earth dams. Journal of Geotechnical Engineering, 109(7): 946-960.

Lam, L., Fredlund, D.G. and Barbour, S. L. (1987). "Transient seepage model for saturated-unsaturated soil systems: a geotechnical engineering approach." Canadian Geotechnical Journal, 24(4): 565-580.

Mansur, C.I., and Kaufman, R.I. (1957). "Underseepage-Mississippi River Levees." St. Louis District. Trans. ASCE, 122.

Mesri, G. and Ajlouni, M. (2007). "Engineering properties of fibrous peats." Journal of Geotechnical and Geoenvironmental Engineering, 133(7): 850–866.

Peter, P. (1982). "Canal and river levees. Developments of Civil Engineering." Vol. 29, Elsevier/North-Holland, Inc., New York.

Stark, T.D., Jafari, N.H., Leopold, A.L., Brandon, T.L. (2014). "Soil compressibility in transient unsaturated levee seepage." Canadian Geotechnical Journal, in press.

Terzaghi, K., Peck, R.B., and Mesri, G. (1996). Soil mechanics in engineering practice. John Wiley & Sons, N.Y.

Turnbull, W.J. & Mansur, C.I. (1961). "Investigation of underseepage – Mississippi River Levees." Trans. ASCE, 126: 1486-1539.

U.S. Army Corps of Engineers (USACE). (1993). "Seepage analysis and control for dams." EM 1110-2-1901, Department of the Army, Washington, D.C.

U.S. Army Corps of Engineers (USACE). (2000). "Engineering and design—design and construction of levees." EM 1110-2-1913, Department of the Army, Washington, D.C.

U.S. Army Corps of Engineers (USACE). (2005). Design guidance for levee underseepage. ETL 1110-2-569, Department of the Army, Washington, D.C.

Washington Group International (W.G.I.) (2001). RECAP submittal report-borrow pit report. Inner Harbor Navigation Canal-East Bank Industrial Area New Orleans, LA. Prepared for the U.S. Army Corps of Engineers, New Orleans District, LA.

Wink, Incorporated. (2005). Map of survey – East Bank Industrial Area. Prepared for Washington Group International.