UNCERTAINTY OF SOIL COMPRESSIBILITY IN TRANSIENT AND UNSATURATED SEEPAGE ANALYSES

Timothy D. Stark, Ph.D., P.E., D.GE[1]
Navid H. Jafari, NSF Fellow[2]

ABSTRACT

Most levee underseepage and uplift analyses are based on steady-state seepage and can yield conservative results. Transient and unsaturated seepage analyses are more representative of levee seepage conditions because boundary conditions acting on the levee or floodwall and saturation changes with time, which induce pore-water pressure and seepage changes with time in the embankment and foundation strata. In addition, these boundary conditions, e.g., flood surge or storm event, are rapid such that steady-state conditions may not have time to develop in the embankment and some foundation materials. 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 uncertainty analysis shows that the Factor of Safety Most Likely Value (FS_{MLV}) is 1.09 but coefficient of variation is 18%, which indicates high uncertainty. The hydraulic conductivity controls Factor of Safety (FS) standard deviation while soil compressibility does not contribute to overall uncertainty.

INTRODUCTION

Over 100,000 miles of flood protection infrastructure are currently operating in the United States (National Committee on Levee Safety 2009), e.g., along Mississippi, Sacramento, Trinity, Missouri, and American Rivers. The increase in development behind levees and floodwalls poses increased risk to public health and safety. The societal, economic, and environmental risks will consequently play a greater role in assessing the required performance of flood protection. Current performance of urban levees and floodwalls to hurricane and flood events are primarily based on steady-state seepage analyses. 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.

Lambe and Whitman (1969) define transient flow as the condition during water flow where pore-water pressure, and thus total head, changes with time. During transient conditions, changes in hydraulic boundary conditions and boundary total stresses cause: (1) saturated seepage through relatively pervious foundation strata (Casagrande 1937;

[1] Professor, Department of Civil Environmental Engineering, University of Illinois, 205 N. Mathews Ave., Urbana, IL 61801, tstark@illinois.edu
[2] Doctoral Candidate, Department of Civil Environmental Engineering, University of Illinois, 205 N. Mathews Ave., Urbana, IL 61801, njafari2@illinois.edu

1961; Mansur and Kaufman 1957; Turnbull and Mansur 1961); (2) unsaturated seepage through earth embankments (Lam et al. 1987); and (3) shear induced pore-water pressures resulting from changes in boundary total stresses, e.g., flood water or storm surge (Alonso and Pinyol 2011).

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 (Fredlund and Rahardjo 1993) 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.

The impact of steady-state conditions and foundation underseepage on earth structures is well-documented, but the influence of soil compressibility on underseepage and uplift pressures is less understood by geotechnical engineers. This paper demonstrates the importance of soil compressibility in transient seepage analyses, presents methods for evaluating and selecting compatible values of m_v, and investigates the uncertainty of soil parameters in uplift factor of safety. 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

The equation for 3-D transient flow through a saturated anisotropic porous medium is:

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

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 considered incompressible (4.7×10^{-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 1×10^{-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.

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

Because the time variant nature of seepage problems is generally due to movement of the phreatic surface, the compression of the saturated seepage layer can be neglected, i.e., specific storage is negligible. However, m_v in Eq. (1) influences transient flow by controlling the rate of pore-water pressure response in saturated layers. 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.

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. 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 materials (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) 1-D consolidation tests.

$$m_v = \frac{\Delta \varepsilon_v}{\Delta \sigma'_v} = \frac{1}{D} \tag{3}$$

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.

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

Figure 1 provides 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) diameter fixed-piston samples. Figure 1 illustrates the general trend of m_v as a function of in-situ σ'_{vo}. Figure 1 indicates a limited correlation between m_v and σ'_{vo}. Values of m_v range from 2×10^{-4} to 2×10^{-3} kPa^{-1} for the saturated inorganic soils and 7×10^{-4} to 3.5×10^{-3} kPa^{-1} for saturated organic clays located along the IHNC. The HCV, MLV, and LCV in Figure 1 correspond to highest conceivable, most likely, and least conceivable values of the organic clay and is used subsequently in the uncertainty analysis. 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.

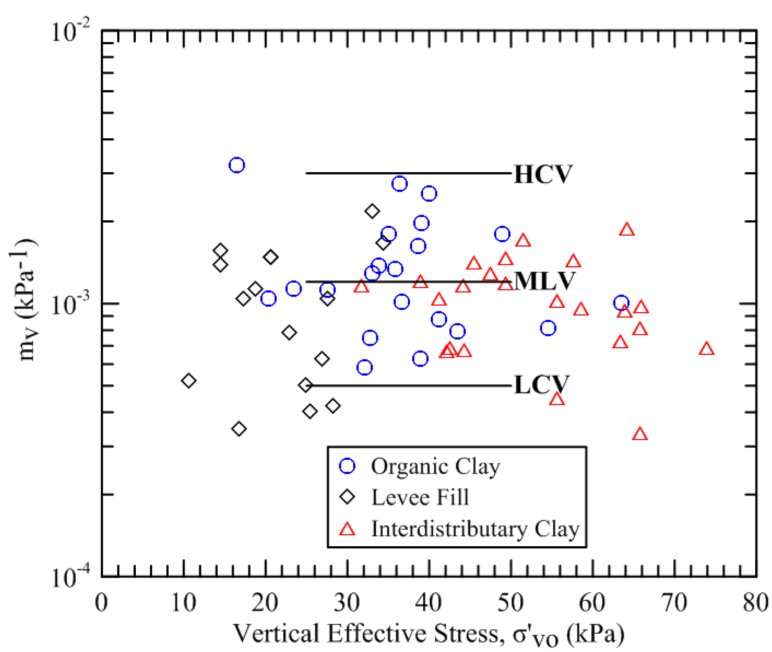

Figure 1. Soil compressibility (m_v) of IHNC clayey soils in terms of effective vertical stress from laboratory consolidation tests (data from Fugro 2012)

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 composition of soil, i.e., mineralogy, controls because any soil that comes to equilibrium at a high void ratio under typical overburden pressures displays high compressibility when subjected to an increase in σ'_v beyond the preconsolidation pressure, σ'_p. 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, e.g., $C_c = w_o/100$ (Mesri and Aljouni 2007; Stark et al. 2014), that may be used to estimate m_v.

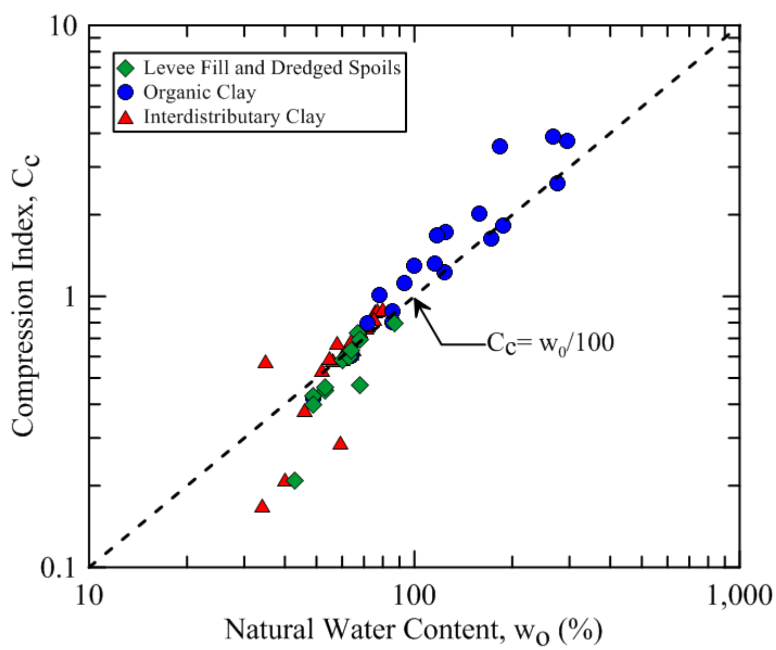

Figure 2. Correlation between compression index and in situ water content for IHNC soils (after Stark et al. 2014)

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. For example, the remedial work along the 17th Street Canal in New Orleans after Hurricane Katrina required installation of piezometers on both the landside and floodside of the levee. Using the recorded changes in canal level before and during Hurricane Gustav in 2010, a transient seepage analysis was developed and calibrated by modeling the piezometric response on both sides of the levee (URS 2011). Before Hurricane Gustav, the steady-state groundwater condition was matched with piezometer readings and the groundwater surface in soil borings. To replicate piezometer readings during Hurricane Gustav, both k_h and m_v were adjusted within reasonable ranges until agreement between the piezometer response and transient seepage model was achieved. As a result, the calibrated transient seepage model can be used to estimate uplift pressures during future storm surges or floods or the effectiveness of remedial measures. The estimated m_v computed for the organic clay at the 17th Street floodwall is 6.3×10^{-4} kPa^{-1}, which is in agreement with the values shown in Table 1 and Figure 1. A similar calibration was performed for the case study using VW piezometer data during Tropical Storm Lee in 2011.

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). The resulting tidal surge rushed from the Gulf of Mexico through the Mississippi River-Gulf Outlet (MR-GO) and the Gulf Intracoastal Waterway (GIWW) to produce a storm surge of about 4.25 m (14 ft) and wind speeds over 160 kph (100 mph) in the IHNC along the Lower Ninth Ward. This storm surge severely loaded and overtopped portions of the IHNC floodwall 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 cross-section 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. A large floodside soil borrow pit in Figure 3 was previously excavated to acquire suitable backfill material for the various excavations created during environmental restoration of the eastern side of the IHNC (referred to as the East Bank Industrial Area (EBIA) herein). 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. The maximum depth of the borrow pit is elevation -3.5 m NAVD88 (current elevation datum in New Orleans, 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. Figure 3 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.

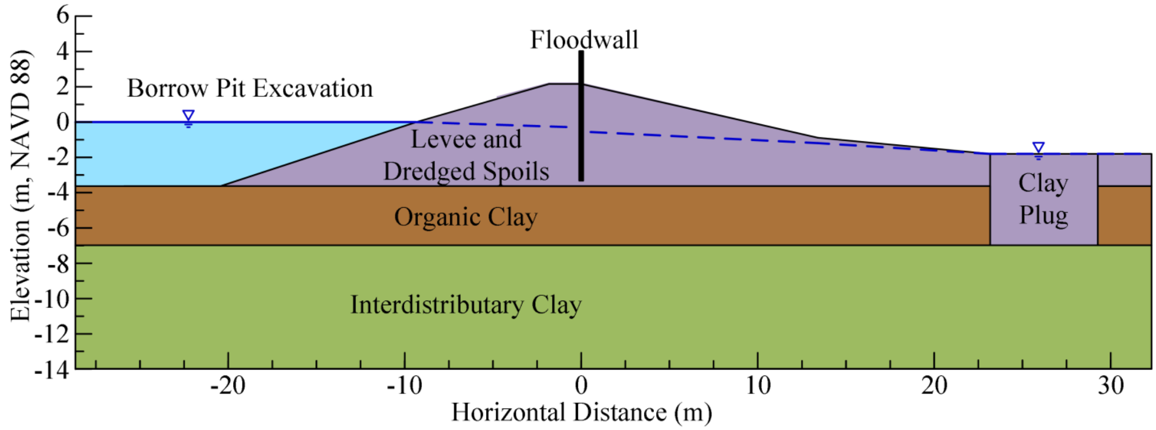

Figure 3. Floodwall cross-section through borrow pit used for parametric seepage analysis (after Stark et al. 2014)

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 3. 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.

SOIL PROPERTIES

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. 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. The fill material consists of a heterogeneous mixture of gray, soft to stiff lean clay, silt, silty sand, and shell fragments. 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 ID clay consists of gray to dark gray, medium to soft clay with lenses of silty sand and silt and medium lean clay.

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 2 also 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.

Table 2. Soil parameters for transient seepage analyses of No-Failure Section in Figure 3 (data from Fugro 2012)

Parameter	**Levee Fill and Dredged Spoils**	**Organic Clay**	**Interdistributary (ID) Clay**
Soil Classification	CH	OH	CH
γ_{sat} (kN/m^3)	16.3	9.9 – 13.7	18.0
w_o (%)	43-68	49-296	34-80
LL (%)	73-103	166	33-97
PI (%)	21-76	111	16-72
k_h (cm/s)	$1x10^{-6}$	$3x10^{-5}$	$3x10^{-7}$
k_v/k_h	0.83	0.33	0.50
m_v (kPa^{-1})	$8x10^{-4}$	$1.2x10^{-3}$	$8x10^{-4}$

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 4). In Figure 4, the hurricane surge increased from +1.4 m NAVD88 after 10 hours to +4.3 m NAVD88 at 25 hours and then precipitously returned to elevation +0 m. Because the top of the floodwall is at +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 4 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 4. Hurricane Katrina is a short-term event while Duncan Point 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.

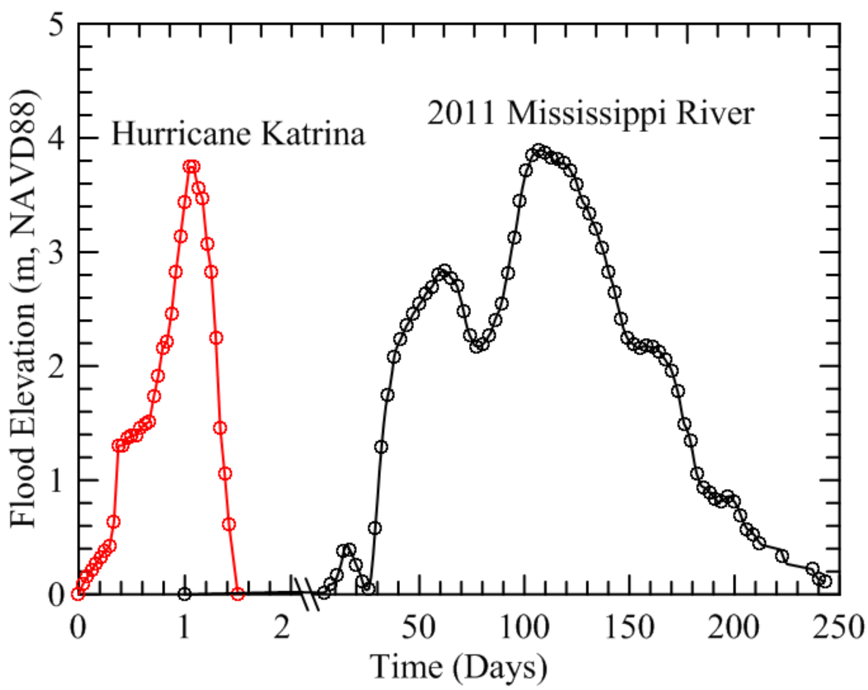

Figure 4. 2005 Hurricane Katrina at IHNC and 2011 Mississippi River Flood at Duncan Point, LA hydrographs applied in parametric study

The SEEP/W program ‘‘seepage exit face’’ 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 m 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 3 is modeled as an impervious boundary due to the low hydraulic conductivity ID clay.

CALIBRATION

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 -2.3 m, -3.8 m, and -4.7 m NAVD88. However, the piezometer at -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.

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. In addition, though an extensive site investigation the organic clay k_h was found to range between $1x10^{-6}$ to $1x10^{-8}$ cm/s with values of $1x10^{-5}$ cm/s because of higher organic content. As a result, the parametric analyses were performed using values of k_h in the 10^{-5} cm/s range.

EFFECT OF SYSTEM COMPRESSIBILITY

Figure 5 presents the factor of safety against uplift developed using parametric values of organic clay k_h, ranging from $1.5x10^{-5}$ 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 5. 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 5 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 5) 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}, Figure 5 suggests that during the flood the difference in FS is 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.

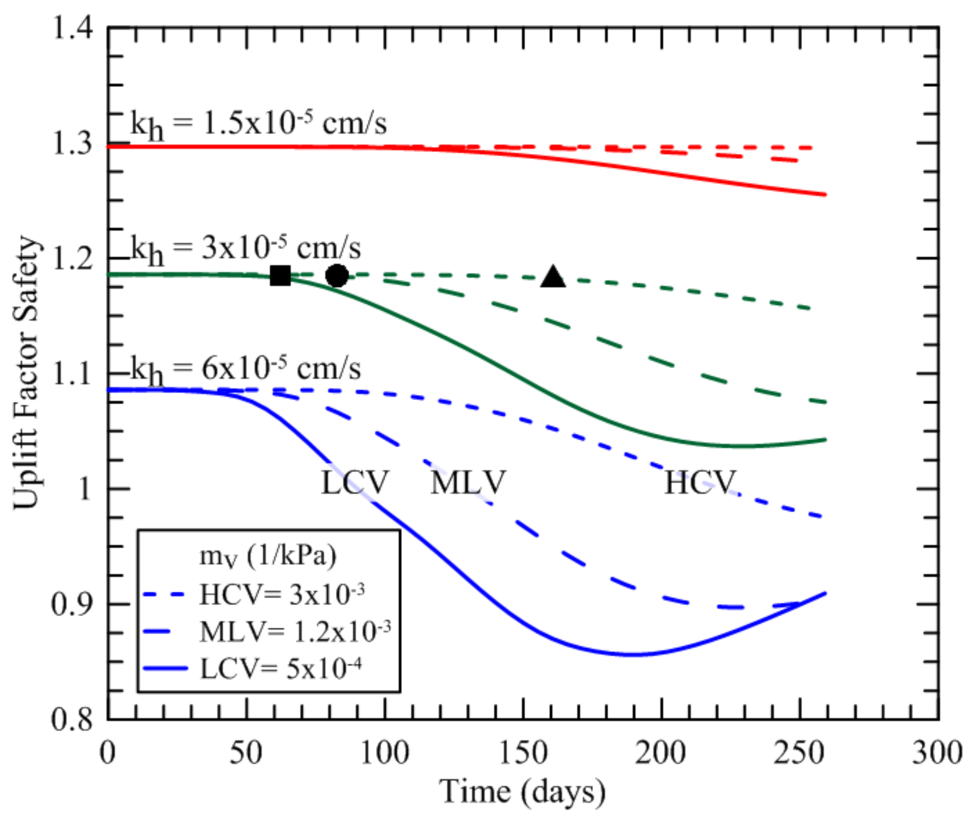

Figure 5. Effect of m_v on factor of safety against uplift at clay plug in Figure 3 using Mississippi River flood hydrograph for various k_h and m_v

UNCERTAINTY IN FACTOR OF SAFETY

The Taylor Series method can be used to capture the effect of soil variability and differences in test procedures on the factor of safety. In this method, the coefficient of variation (C.O.V.) of soil properties, e.g., hydraulic conductivity, soil compressibility, and saturated unit weight, are used to estimate the uncertainty in FS. Duncan (2000) uses slope stability factor of safety to illustrate this method. As a result, the method proposed by Duncan (2000) was modified to capture the variability of soil parameters used in the finite element seepage analyses.

Duncan (2000) proposes several procedures to estimate the variable standard deviations impacting the computed FS. For example, the C.O.V. of saturated unit weight is about 5% (Duncan 2000), so one standard deviation (SD) can be found by multiplying 5% and the most likely value (MLV). Harr (1984) reports the upper limit C.O.V. is 90% for saturate hydraulic conductivity. The soil compressibility SD is evaluated using the Graphical Three-Sigma Rule (Duncan 2000) in Figure 1. Table 3 summarizes the highest conceivable (HCV) and lowest conceivable (LCV) values determined for each variable and subsequently used in the uncertainty analysis in Table 4.

Table 3. Variability of soil parameters used in seepage analyses

Variable	**LCV**	**MLV**	**HCV**	**C.O.V.**	**Reference**
γ_s (kN/m^3)	14.2	15.7	17.2	~5%	Duncan (2000)
k_h (cm/s)	3.8x10^{-6}	3x10^{-5}	2.4x10^{-4}	90%	Harr (1984)
m_v (1/kPa)	5x10^{-4}	1.2x10^{-3}	3x10^{-3}	~42%	This study

The Taylor Series method is used to estimate the standard deviation and variance in FS. The standard deviation in factor of safety (σ_F) is estimated using the following Taylor Series expression:

$$\sigma_F = \sqrt{\left(\frac{\Delta F_1}{2}\right)^2 + \left(\frac{\Delta F_2}{2}\right)^2 + \left(\frac{\Delta F_3}{2}\right)^2} \qquad (5)$$

where ΔF is the change in FS computed for the most likely value (MLV) plus one SD (+1 SD) and the MLV minus one SD (-1 SD) for the parameter in question. The coefficient of variation of factor of safety (V_F) is calculated as:

$$V_F = \frac{\sigma_F}{F_{MLV}} \qquad (6)$$

where F_{MLV} is the FS using the most likely values for each parameter. The uplift FS is computed as the ratio of total overburden stress (σ_v=31.3 kPa) and uplift pore-water pressures from the finite element seepage analysis. Table 4 shows the results from the uncertainty analysis. For each variable, the FS computed using the +1SD and -1SD value is used to determine ΔF. For example, +1SD FS and -1SD FS are 0.92 and 1.27, respectively, for horizontal hydraulic conductivity, and the resulting ΔF is -0.35. After ΔF is calculated for each variable, Eq. (5) can be used to estimate the standard deviation σ_F and Eq. (6) to estimate coefficient of variation V_F.

Table 4. Taylor Series uncertainty analysis for a floodwall

Variable	Values	Uplift FS	ΔF
Saturated unit weight, γ_s			
MLV +1 SD	16.2 kN/m^3	1.13	0.10
MLV -1 SD	15.2 kN/m^3	1.03	
Horizontal hydraulic conductivity, k_h			
MLV +1 SD	6x10^{-5} cm/s	0.92	-0.35
MLV -1 SD	1.5x10^{-5} cm/s	1.27	
Soil compressibility, m_v			
MLV +1 SD	1.7x10^{-3} kPa^{-1}	1.12	0.04
MLV -1 SD	9.1x10^{-4} kPa^{-1}	1.08	

$$\sigma_F=\sqrt{\left(\frac{0.1}{2}\right)^2+\left(\frac{-0.35}{2}\right)^2+\left(\frac{0.04}{2}\right)^2}=0.18$$

$$V_F=\frac{\sigma_F}{F_{MLV}}=\frac{0.18}{1.09}=17\%$$

Based on Table 4, the σ_F and V_F are 0.18 and 17%, respectively. Although the F_{MLV} is 1.09 or marginally stable, the V_F indicates that there is considerable uncertainty in the FS. As a result, V_F provides a method to incorporate the variability in soil properties to evaluate the probability of failure. In addition, Table 4 indicates that hydraulic conductivity plays the most important role in Uplift FS uncertainty and that soil compressibility contribution is small. This result can be attributed to m_v for fine grained soils because they vary within one order of magnitude, i.e., 10^{-4} kPa^{-1} (see Table 1). Therefore, it is necessary to use compatible soil compressibility and hydraulic conductivity values (Stark et al. 2014).

SUMMARY AND CONCLUSIONS

This paper illustrates the importance of soil compressibility on transient and unsaturated seepage analyses using a floodwall case study. This paper also provides guidance on selecting compatible values of soil compressibility and hydraulic conductivity for transient seepage analyses and the potential conservatism with a steady-state analysis. The following information and recommendations were derived from the parametric analyses:

1. The derivation of transient seepage flow indicates that reducing the value of m_v, i.e., making the system incompressible, transforms the transient seepage analysis to a steady-state analysis by essentially eliminating the effect of time on seepage. Although water is considered incompressible, m_v is also a function of the soil skeleton compressibility and hence should appropriately be selected.
2. The increase in landside uplift pressure and effect of m_v are significant for $k_h > 10^{-5}$ cm/s as shown in Figure 5. This shows that selecting a compatible m_v value for saturated fine-grained soils with $k_h > 10^{-5}$ cm/s is critical for developing a representative transient seepage analysis.
3. The parametric analyses show that values of m_v for the seepage layer and system affect the time at which landside uplift pressures start to increase and the magnitude of landside uplift pressures. In particular, the effect of m_v diminishes as the soil becomes more compressible. As expected, seepage flow must be present for uplift pressures to be generated on the landside of the levee or floodwall.
4. General procedures for estimating m_v include laboratory consolidation tests, C_c empirical correlations, field calibration using piezometer data, piezometer slug tests, and field pump tests. Values of m_v vary by about an order of magnitude for the same soil type. To account for uncertainty in m_v, the average or median m_v should be used with additional analyses using the highest and lowest conceivable values, respectively, to develop lower and upper bounds of response. The selected value of m_v should also be representative of the in situ effective vertical stress and compatible with the value of horizontal hydraulic conductivity.

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.

Casagrande, A. (1937). “Seepage through dams.” New England Water Works. Vol. II, No. 2.

Casagrande, A. (1961). “Control of seepage through foundations and abutments of dams.” 1st Rankine Lecture. Géotechnique, 11(3): 161–182.

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.

Duncan, J.M. (2000) “Factors of safety and reliability in geotechnical engineering.” J. Geotechnical and Geoenvironmental Engineering, 126(4), 307-316.

Fredlund, D.G., and Rahardjo, H. (1993). Soil mechanics for unsaturated soils. John Wiley & Sons, Inc., N.J.

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.

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

Harr, M. E. (1987). Reliability-based design in civil engineering. McGraw-Hill, New York.

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

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

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.

National Committee on Levee Safety. (2009). Recommendations for a national levee safety program. Draft to Congress.

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, 51(8), 858-868.

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.

URS. (2011). “Remediation of floodwalls on the 17th Street Canal.” OFC-05. 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.