EFFECT OF SOIL COMPRESSIBILITY IN TRANSIENT SEEPAGE ANALYSES

Timothy D. Stark, F.ASCE, D.GE
Professor of Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
205 N. Mathews Ave.
Urbana, IL 61801
(217) 333-7394
333-9464 Fax
tstark@illinois.edu

Navid Jafari
Graduate Research Assistant of Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
205 N. Mathews Ave.
Urbana, IL 61801
njafari2@illinois.edu

and

Aaron Leopold
Undergraduate Assistant of Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
205 N. Mathews Ave.
Urbana, IL 61801
leopold3@illinois.edu

Paper# GTENG-????
A paper TO BE submitted for review and possible publication in the
ASCE *Journal of Geotechnical and Geoenvironmental Engineering*

October 28, 2012

Effect of Soil Compressibility on Transient Seepage Analyses

Timothy D. Stark[1], F.ASCE, D.GE, Navid Jafari[2], S.M., ASCE, and Aaron Leopold[3], S.M.ASCE

ABSTRACT:
Most analyses of levee underseepage, erosion, piping, heave, and sand boil formation are based on steady seepage flow because the computations are simpler and steady-state seepage parameters are less difficult to determine than the corresponding transient parameters, and yield conservative results. However, transient seepage is more representative of levee seepage conditions because the boundary conditions acting on the levee or floodwall change with time, which induces pore-water pressure 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 foundation materials. This paper presents the large and important affects that the coefficient of compressibility, m_v, can have on levee and floodwall seepage during flood and hurricane events via a parametric study. This paper also presents methods for evaluating and selecting m_v and provides recommendations for performing transient seepage analyses.

Keywords: transient seepage analysis, hydraulic conductivity, compressibility, levee, gradient

[1] Professor, Dept. of Civil and Environmental Engineering, Univ. of Illinois, 205 N. Mathews Ave., IL 61801-2352. E-mail: tstark@illinois.edu
[2] Doctoral Candidate, Dept. of Civil and Environmental Engineering, Univ. of Illinois, 205 N. Mathews Ave., IL 61801-2352. E-mail: njafari2@illinois.edu
[3] Undergraduate Assistant, Dept. of Civil and Environmental Engineering, Univ. of Illinois, 205 N. Mathews Ave., IL 61801-2352. E-mail: leopold3@illinois.edu

INTRODUCTION

Levees are a significant part of the United States flood protection infrastructure. It is estimated that over 161,000 km (100,000 miles) of levees exist in the United States. The vast majority of the levees across the nation are not part of any federal program. There are approximately 14,800 miles of levee enrolled in US Army Corps of Engineers programs (including those built by the Corps and locally maintained) and another 14,000-16,000 miles estimated to be operated by other federal agencies (US Bureau of Reclamation, National Resources Conservation Service).

The reliability and performance of levees, e.g., New Orleans and Sacramento-San Joaquin, to hurricane and flood events are based on steady state analyses. Transient seepage analyses are important to evaluate seepage performance of floodwalls and levees because steady state analyses provide a conservative design scenario. The flood or hurricane conditions typically only act for a period of days to weeks, which may not allow sufficient time to develop steady state conditions. As a result, a transient seepage analysis provides a more realistic approach to evaluating levee seepage and stability especially for failure causation analyses. Although a safe levee design may be correctly analyzed using a transient model, a steady state analysis will yield a conservative result. However, a steady state analysis of an existing levee may indicate unsatisfactory performance or an erroneous failure causation mechanism.

During a transient levee seepage analysis, e.g., a flood event, pore-water pressures are generated by: (1) partially saturated seepage through the levee and (2) underseepage through pervious foundation strata. The first seepage mechanism involves dissipating the negative (suction) pore-water pressure to positive pore-water pressure as the phreatic surface progresses through the levee. The second mechanism is dependent on the hydraulic conductivity, hydraulic conductivity ratio (k_v/k_h), and coefficient of volume compressibility (m_v) of the foundation strata. The impact of hydraulic conductivity and hydraulic conductivity ratio are widely documented, e.g., Cedergren (1989). This paper presents the large and important affects that m_v can have on levee and floodwall seepage during flood and hurricane events. This paper also presents methods for evaluating and selecting m_v and provides recommendations for performing transient seepage analyses. Finally, a hypothetical parametric study is presented to show the relationship between pore-water pressure and time to reach steady state condition for a range of hydraulic conductivity and m_v values.

BACKGROUND

Most analyses of underseepage, erosion, piping, heave, and sand boil formation have been based on steady seepage flow because the computations are simpler and steady-state seepage parameters are less difficult to determine than the corresponding transient parameters. However, transient seepage is more representative of seepage conditions for channel embankments, levees, and floodwalls (Peter 1982).

In general, flow is considered to be transient if changes in water level in wells and piezometers are measurable, i.e., hydraulic gradient is changing in a measurable way (Kruseman and Ritter 1991). Freeze and Cherry (1979) state that transient flow (unsteady or nonsteady flow) occurs when at any point in a flow field the magnitude or direction of flow velocity changes with time. In addition, Lambe and Whitman (1969) define transient flow as the condition during fluid flow where pore-water pressure, and thus total head, changes with time. Based on these definitions, transient seepage analyses are applicable to levees and floodwalls because the boundary conditions acting on the levee or floodwall change with time, which induces pore-water pressure changes with time in the embankment and foundation strata. More importantly, these boundary conditions, e.g., flood surge or storm event, are rapid such that steady state conditions may not have time to develop.

The U.S. Army Corps of Engineers (USACE) design manual for design and construction of levees (2000) details the analysis of underseepage and foundation uplift pressures for levees. The procedure to evaluate the quantity of underseepage, uplift pressures and hydraulic gradients was developed based on closed form solutions to differential equations of seepage flow presented by Bennett (1946). The equations in this Engineer Manual are developed considering a two-layer foundation, which is a typical geological condition in Lower Mississippi River Valley, and steady state conditions. The manual does not require transient seepage analyses for design of levees or floodwalls. Instead, the USACE is currently basing undeerseepage and slope stability analyses on steady state boundary conditions, which can lead to conservative results and results that do not model field conditions.

Hydraulic gradient are a function of soil hydraulic conductivity, hydraulic conductivity ratio, i.e., ratio of vertical to horizontal hydraulic conductivity, and m_v which is now being included in commercial seepage software. The first two soil characteristics are known to greatly affect transient seepage pore pressure generation. However, the coefficient of volume compressibility is relatively new to commercial software and transient seepage.

SEEPAGE THEORY

This section reviews transient seepage theory and provides methods for determining m_v. The law of mass conservation for steady state flow through a saturated porous medium requires that the rate of fluid mass flow into any elemental control volume be equal to the rate of fluid mass flow out of the any elemental control volume. The equation of continuity that translates this law into mathematical form is:

$$-\frac{\partial(\rho v_x)}{\partial x}-\frac{\partial(\rho v_y)}{\partial y}-\frac{\partial(\rho v_z)}{\partial z}=0 \qquad (1)$$

where ρ is fluid density and $\rho\upsilon$ term is the mass rate of flow across a unit cross-sectional area of the elemental control volume. By assuming the fluid is incompressible, ρ is removed from Eq. (1). Substitution of Darcy's law for υ_x, υ_y, and υ_z in Eq. (1) yields the equation for steady-state flow through an anisotropic saturated porous medium:

$$\frac{\partial}{\partial x}\left(K_x\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_z\frac{\partial h}{\partial z}\right)=0 \qquad (2)$$

where *K* is the hydraulic conductivity in *x*, *y*, and *z* directions. The solution of Eq. (2) is a function of h(*x*, *y*, *z*) that describes the value of the total hydraulic head, *h*, at any point in a three-dimensional (3-D) flow field. The law of mass conservation for transient flow in a saturated porous medium requires that the net rate of fluid mass flow into any elemental control volume be equal to the time rate of change of fluid mass stage within the element. The equation of continuity takes the following form:

$$-\frac{\partial(\rho v_x)}{\partial x}-\frac{\partial(\rho v_y)}{\partial y}-\frac{\partial(\rho v_z)}{\partial z}=n\frac{\partial\rho}{\partial t}+\rho\frac{\partial n}{\partial t} \qquad (3)$$

The term $n\frac{\partial\rho}{\partial t}$ is the mass rate of water produced by an expansion of the water under a change in its density ρ and is controlled by the compressibility of the fluid, β. The term $\rho\frac{\partial n}{\partial t}$ is the mass rate of water produced by the compaction of the porous medium as reflected by the change in its porosity, n, and is controlled by the compressibility of the aquifer, α. Because the change in porosity and density are both produced by a change in total hydraulic head, the volume of water produced by porosity and density for a unit decline in total head is S_s, or specific storage. As a result, the right-hand side of Eq. (3) can be expressed as:

$$-\frac{\partial(\rho v_x)}{\partial x}-\frac{\partial(\rho v_y)}{\partial y}-\frac{\partial(\rho v_z)}{\partial z}=\rho S_s\frac{\partial h}{\partial t} \tag{4}$$

Expanding the left-hand side terms, e.g., $\frac{\partial(\rho v_x)}{\partial x}$, by the chain rule, density, ρ, is eliminated from both sides of the equation because the term $\rho\frac{\partial v_x}{\partial x}$ is much greater than $v_z\frac{\partial \rho}{\partial x}$. Inserting Darcy's law, the equation of flow for transient flow through a saturated anisotropic porous medium is:

$$\frac{\partial}{\partial x}\left(K_x\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_z\frac{\partial h}{\partial z}\right)=S_s\frac{\partial h}{\partial t} \tag{5}$$

The specific storage, S_s, volume of water that a unit volume of aquifer releases from storage under a unit decline in hydraulic total head, can be expressed as:

$$S_s=\rho g(\alpha+n\beta) \tag{6}$$

where ρg is unit weight of water (γ_w), and the term $\alpha+n\beta$ is coefficient of volume compressibility, m_v. In Eq. (6), m_v is a function of both fluid (β) and soil (α) compressibility and assuming β is incompressible should not be justification for assuming m_v is also incompressible. Combining Eq. (5) and (6) results in transient flow being defined in terms of m_v:

$$\frac{\partial}{\partial x}\left(K_x\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_z\frac{\partial h}{\partial z}\right)=\gamma_w m_v\frac{\partial h}{\partial t} \tag{7}$$

For a unit decline in total hydraulic head, the right-hand side of Eq. (7) is directly related to the magnitude of m_v. Most m_v values for clays are within a relatively small range, i.e. one or two orders of magnitude. However, if an incompressible m_v, e.g., $2.1\text{x}10^{-8}$ kPa^{-1} ($1\text{x}10^{-9}$ psf^{-1}), is assumed, the right-hand side of Eq. (7) approaches zero, which corresponds to a steady state condition (see Eq. (2)). In other words, an incompressible value for m_v transforms the transient flow equation (Eq. (7)) into the steady state flow equation (Eq. (2)). Consequently, the resulting seepage analysis simulates a steady state condition and generates higher pore-water pressures and gradients than observed in the field.

ESTIMATING SOIL COMPRESSIBILITY

Coefficient of volume compressibility is the change in volume induced in a material under an applied stress, i.e., the ratio of the change in strain to the resulting change in stress ($\Delta\varepsilon_v/\Delta\sigma_v$), or simply the inverse of the modulus of elasticity. The coefficient of volume compressibility is determined from consolidation test data according to:

$$m_v = \frac{1}{M} \tag{8}$$

where M is the modulus of elasticity in confined compression. From consolidation test data, m_v can be expressed in terms of:

$$m_v = \frac{a_v}{1 + e_o} \tag{9}$$

where a_v is the coefficient of compressibility and e_o is the initial void ratio. Finally, m_v can be defined in terms of compression index, C_c, and σ_{va}, average of initial and final stresses:

$$m_v = \frac{0.435 C_c}{(1 + e_o)\sigma_{va}} \tag{10}$$

Table 1 illustrates the range of m_v for soils and rocks, i.e., from 10^{-3} kPa to 10^{-8} kPa, respectively. In comparison, the compressibility of water is similar to sound rock but is also representative of lower end m_v values of gravel and jointed rock. In seepage analyses, sound rock, jointed rock, and clean gravel can be considered incompressible. Table 1 subdivides the broad soil types in Table 1 and provides representative m_v values. Table 2 summarizes values of compressibility for fine grained soils, e.g., soft organic clays and peats to stiffer over consolidated tills. For soils where laboratory consolidation testing is not performed, Table 2 can provide a reasonable first estimate of compressibility for saturated fine grained soils for parametric studies.

Table 1: Range of m_v values for various materials (after Domenico and Mifflin 1965)

Soil Type	m_v (kPa^{-1})	m_v (psf^{-1})
Plastic clay	$2.1x10^{-3}$ to $2.6x10^{-4}$	$1x10^{-4}$ to $1.25x10^{-5}$
Stiff clay	$2.6x10^{-4}$ to $1.3x10^{-4}$	$1.25x10^{-5}$ to $6.25x10^{-6}$
Medium hard clay	$1.3x10^{-4}$ to $6.9x10^{-5}$	$6.25x10^{-6}$ to $3.3x10^{-6}$
Loose sand	$1x10^{-4}$ to $5.2x10^{-5}$	$5x10^{-6}$ to $2.5x10^{-6}$
Dense sand	$2.1x10^{-5}$ to $1.3x10^{-5}$	$1x10^{-6}$ to $6.25x10^{-7}$
Dense sandy gravel	$1x10^{-5}$ to $5.2x10^{-6}$	$5x10^{-7}$ to $2.5x10^{-7}$
Jointed rock	$6.9x10^{-6}$ to $3.3x10^{-7}$	$3.3x10^{-7}$ to $1.6x10^{-8}$
Sound rock	$\geq 3.3x10^{-7}$	$\geq 1.6x10^{-8}$
Water (β)	$4.4x10^{-7}$	$2.1x10^{-8}$

Table 2: Summary m_v for fine grained soils (after Bell 2000)

m_v (10^{-3} kPa^{-1})	m_v (10^{-5} psf^{-1})	Degree of Compressibility	Saturated Fine Grained Soils
Above 1.5	Above 7	Very High	organic alluvial clays and peats
0.3 to 1.5	1 to 7	High	normally consolidated alluvial clays
0.1 to 0.3	0.5 to 1	Medium	varved and laminated clays, firm to stiff clays
0.05 to 0.1	0.2 to 0.5	Low	very stiff or hard clays, tills
Below 0.05	Below 0.2	Very Low	heavily over consolidated tills

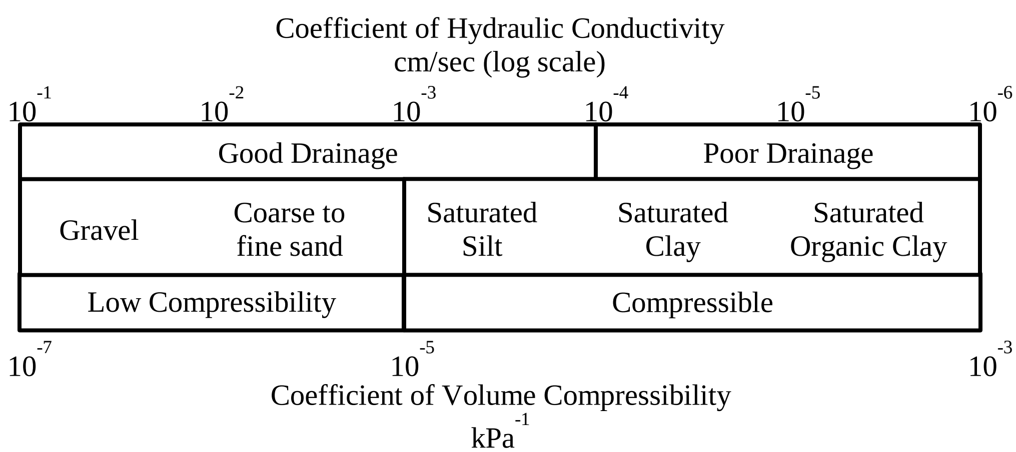


Figure 1: Comparison of saturated hydraulic conductivity and m_v for a range of soils

Fig. 1 relates soil type and saturated hydraulic conductivity to m_v. Hydraulic conductivity is a function of soil drainage and in Fig. 1 the cutoff between good and poor drainage is $k=1x10^{-4}$

cm/sec (Holtz and Kovacs 1981). Similarly, the soil types are divided into low compressibility and compressible based on composition and grain size distribution. Gravels and sands are described as low compressibility (10^{-7} to 10^{-5} kPa^{-1}) while saturated fine grained soils are compressible and range between 10^{-5} to 10^{-3} kPa^{-1}. Based on Fig. 1, as soil hydraulic conductivity transitions from poor to good drainage, i.e., k increases from 10^{-5} to 10^{-3} cm/sec, the saturated soil becomes less compressible, where m_v~$1x10^{-5}$ kPa^{-1}. As a result, evaluating m_v for saturated clays and silts is critical for transient seepage analyses because of its impact on Equation (2).

Fig. 2(a) and Fig 2(b) are a compilation of m_v values for a uniform, fine grained soil layer determined experimentally from incremental load and constant rate of strain consolidation tests. The fine grained soil is normally consolidated, was formed in a deltaic environment, and has natural water content, plastic limit, and liquid limit values of 60%, 26%, and 79%, respectively. This uniform, fine grained soil layer is labeled Deep Foundation Clay in Fig. 3. Fig 2(a) illustrates the general trend of m_v with increasing water content while Fig 2(b) shows compressibility as a function of initial effective vertical stresses (σ'_v). To select an m_v input value for a transient seepage analysis, the soil liquid limit, which is an indication of clay mineralogy, can be used with Fig. 2(c). Most importantly, Fig. 2(c) shows that values of m_v range from $1x10^{-4}$ to $5x10^{-3}$ kPa^{-1} for this saturated and normally consolidated deltaic clay. This means for a transient seepage analysis, the Deep Foundation Clay should not be modeled as incompressible, e.g., $1x10^{-8}$ kPa^{-1}, as is usually assumed in consolidation analyses.

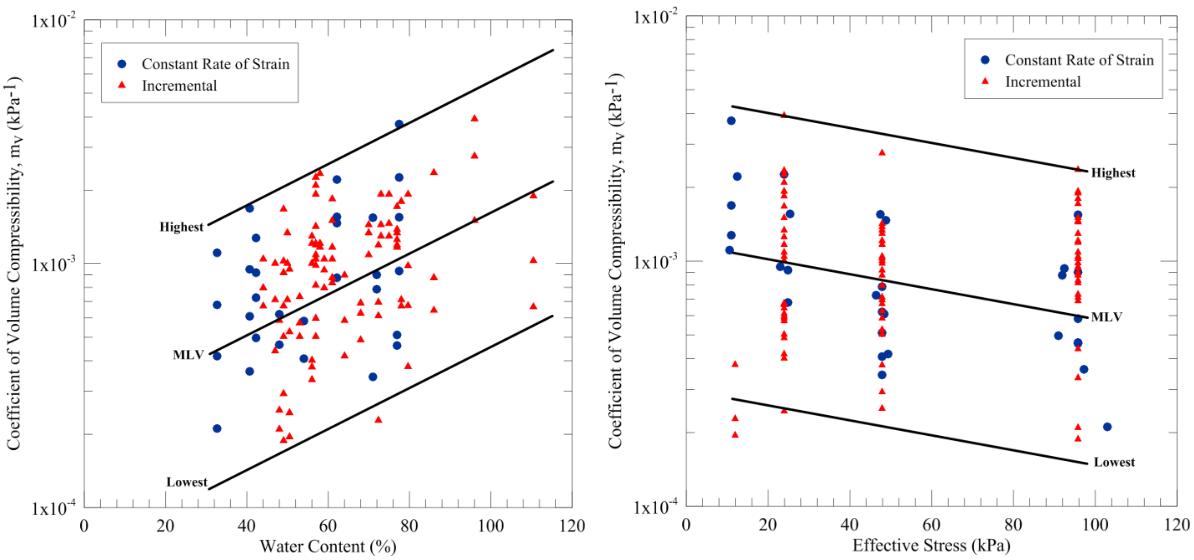
Fig. 2 shows uncertainty is involved in selecting an appropriate value of m_v. To quantify the uncertainty in m_v values, they are assumed to be log-normally distributed (Duncan 2000) because the values are greater than zero and can range several orders of magnitude. Therefore, the median or most likely value (MLV) m_v can be determined graphically, i.e., draw trend lines that represent the highest and lowest m_v values (see Fig. 2). The MLV line is drawn an average distance between the highest and lowest m_v trend lines. Alternatively, if m_v data is available in terms of in-situ effective vertical stress, the following equation can be used to transform the log-normally distributed compressibility to a normal distribution:

$$\lambda = \frac{1}{n}\sum_{i=1}^{n} \ln(m_v) \qquad (10)$$

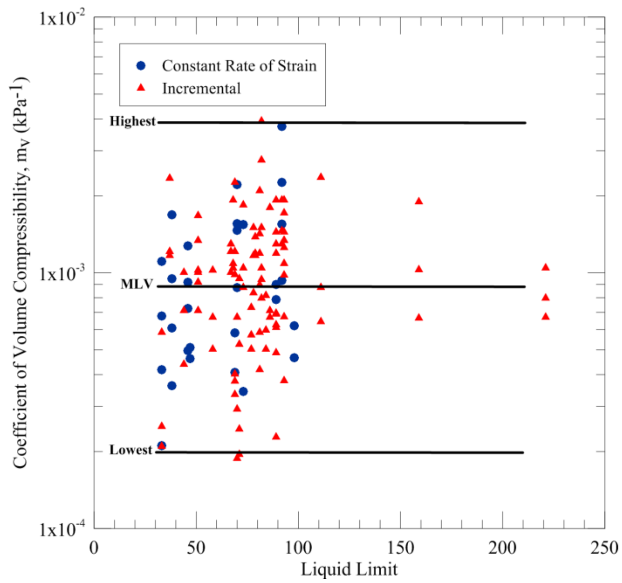
where λ is the mean of $\ln(m_v)$. The relation between median, $x_{0.5}$, and parameter λ is:

$$x_{0.5} = e^{\lambda} \tag{11}$$

As an example, consider data points for m_v at σ'_v of ~95.8 kPa (2,000 psf) in Fig. 2(b). Compute $\ln(m_v)$ for each data point and determine λ and $x_{0.5}$ using Eq. (10) and Eq. (11), respectively. The computed median is 8.6×10^{-4} kPa (4.1×10^{-5} psf^{-1}). The median will always be less than the average value for a log-normal distribution.



(a) (b)



(c)

Figure 2: Compressibility (m_v) of Deep Foundation Clay in Fig. 3 (a) water content, (b) effective vertical stress, and (c) liquid limit

Another method to evaluate m_v is to calibrate the transient seepage model with piezometric response during hurricane events and storm surges. As an example, the remediation of floodwalls along the 17th street canal required transient seepage and stability analyses. The transient seepage analysis was calibrated by modeling the recorded changes in canal levels before and during Hurricane Gustav in 2010 (URS 2011). Before Hurricane Gustav, the steady state condition is matched with piezometer readings by varying the sheet pile wall hydraulic conductivity and thickness. To replicate piezometer readings during Hurricane Gustav, the hydraulic conductivity and coefficient of volume compressibility were adjusted within reasonable ranges until agreement between the piezometer pressure response and transient seepage model was achieved. As a result, the calibrated transient seepage model can be used to estimate the amount of time required for steady-state seepage conditions to develop within the embankment and foundation materials.

HYPTHETICAL FLOODWALL PARAMETRIC ANALYSIS

The derivation of transient seepage flow in Eq. (7) indicates that m_v can be influential in the time required to reach a steady state seepage condition, i.e., increases generation of seepage forces and subsequent probability of erosion and sand boils at the landside levee toe. The USACE

(2000) defines exit gradients of 0.5 to 0.8 to represent conditions favorable for erosion and sand boils. To develop high exit gradients, seepage forces must travel from floodside to landside of the levee to increase pore-water pressures and thus exit gradients. As a result, a hypothetical floodwall system is used to perform parametric analyses of m_v, hydraulic conductivity, and levee geometry to illustrate the importance of m_v on landside pore-water pressures.

The software SEEP\W (Geo-Slope 2007) was used for the two-dimensional (2D) analysis of seepage and hydraulic effects. The CAD-based user interface and automated solver facilitate input of 2D geometries to evaluate field seepage conditions. SEEP/W is a finite element model that can analyze groundwater seepage and excess pore-water pressure dissipation estimated from a stress-deformation analysis within porous materials. SEEP/W can model both saturated and unsaturated flow, which allows it to analyze seepage as a function of time and to consider such processes as infiltration or wetting front migration.

Fig. 3 shows a hypothetical floodwall system consisting of a reinforced concrete floodwall and a supporting sheet pile extending to a depth of -4 m (NAVD88). The sheetpile cuts off seepage or impedes seepage in the upper clay layer so the focus of the seepage analysis is flow through the lower clay layer (see Fig. 3). A deep excavation or burrow pit is modeled 15 m floodside from the floodwall and a clay plug, e.g., clay filled excavation, is modeled 30 m landside of the floodwall. Because the excavation fill is clean sand and hydraulically connected to the lower clay layer, underseepage can occur below the floodwall. The hypothetical excavation represents a possible floodside borrow pit or old river channel which is hydraulically connected to the substratum underlying the levee and clay blanket for this parametric study. The hypothetical excavation is filled with clean sand because preliminary analyses show that excavation filled soils, e.g., $k=10^{-5}$ cm/sec, did not cause landside pore-water pressures to increase, which indicates that seepage flow from the floodside must occur to develop landside uplift pressures.

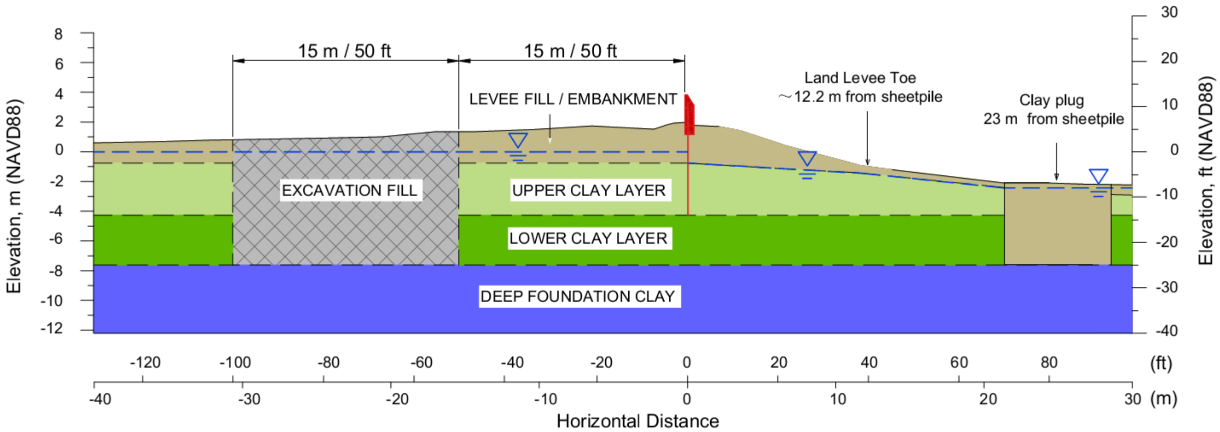


Figure 3: Parametric study soil profile showing floodside excavation, sheet pile wall supported floodwall, and a landside backfilled excavation

Soil Properties

In a transient seepage analysis, three saturated soil properties are required: (1) saturated hydraulic conductivity, (2) saturated anisotropic k_v/k_h ratio, and (3) coefficient of volume compressibility. Table 3 provides the soil properties used for the parametric analysis. These values are representative of soils located in the Inner Harbor Navigation Canal (IHNC) near New Orleans, Louisiana.

Table 3: Soil properties for parametric study

Soil	k (cm/sec)	k_v/k_h Ratio	m_v (kPa^{-1})
Levee Fill/Embankment	$1x10^{-5}$	0.83	$8.35x10^{-4}$
Excavation Fill	$1x10^{-1}$	1	$3.13x10^{-3}$
Canal	$1x10^{-5}$	0.83	$8.35x10^{-4}$

Model Boundary Conditions

The measured storm surge hydrograph measured in the IHNC during Hurricane Katrina is shown in Fig. 4. The hydrograph was modified such that the maximum storm surge does not overtop the floodwall and the maximum flood surge was maintained to evaluate the time required to achieve a steady state condition.

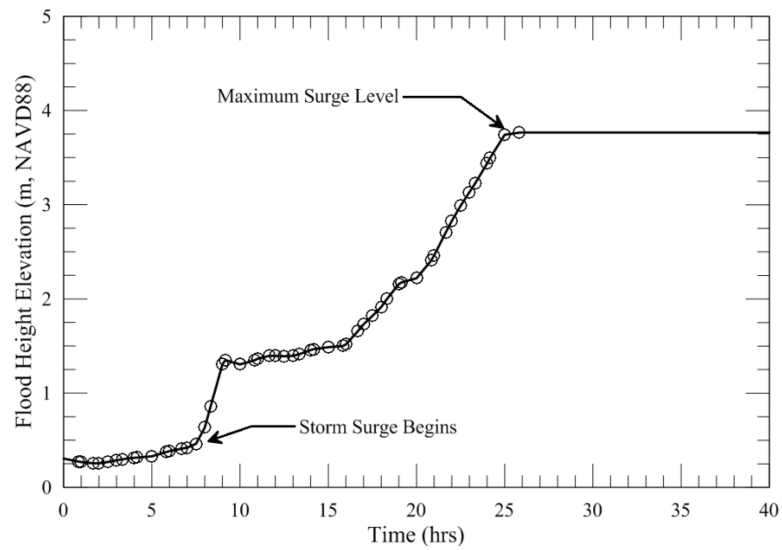


Figure 4: Hydrograph applied in parametric study

The initial floodside steady state boundary condition is assumed to be a total head (h_t) boundary condition of 0 m elevation, which represents the canal water level before the storm surge. The landside boundary conditions are potential seepage face from the floodwall to levee toe and zero pressure head (h_p) boundary condition from landside levee toe to the right hand side of the finite element mesh. The phreatic surface on the landside is unknown but can be estimated using the potential seepage face review boundary. The h_p=0 boundary condition from the levee toe signifies that the groundwater level is at the surface, which is reasonable because of rainfall infiltration into the shallow groundwater surface. 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 Right-Hand Side (RHS) vertical boundary is modeled as a total head boundary (h_t= -2.4 m or about 0.1 m below ground surface) to represent the far field groundwater conditions. Finally, the boundary condition along the bottom of the seepage model in Fig. 3 is modeled as a no flow boundary due to the low hydraulic conductivity overlying clay.

EFFECT OF COEFFICIENT OF VOLUME COMPRESSIBILITY

Fig. 5 shows the time required to reach steady state uplift pressures for a range of hydraulic conductivity and m_v values for the lower clay layer. To determine the time to reach steady state uplift pressures, a steady state seepage analysis at maximum surge level (see Fig. 3) was performed to determine the uplift pressures (shown in Fig. 5). The transient seepage analysis involved using the hydrograph presented in Fig. 3 and maintaining the maximum surge level until the steady state uplift pressures were reached. In Fig. 5, the square, circle, and diamond markers represent time to reach 0%, 50%, and 100% of steady state uplift pressures. For each hydraulic conductivity, Fig. 5 shows analyses performed at m_v values of 2.1x10^{-3} kPa^{-1}, 2.1x10^{-4} kPa^{-1}, and 2.1x10^{-6} kPa^{-1} to represent high, medium, and low compressibility, respectively. The parametric analyses were performed at representative clay hydraulic conductivities of 1x10^{-3} cm/sec, 1x10^{-4} cm/sec, and 1x10^{-5} cm/sec. For k=1x10^{-5} cm/sec, pore-water pressure increase was negligible and so m_v values of 2.1x10^{-3} kPa^{-1} and 2.1x10^{-6} kPa^{-1} are shown.

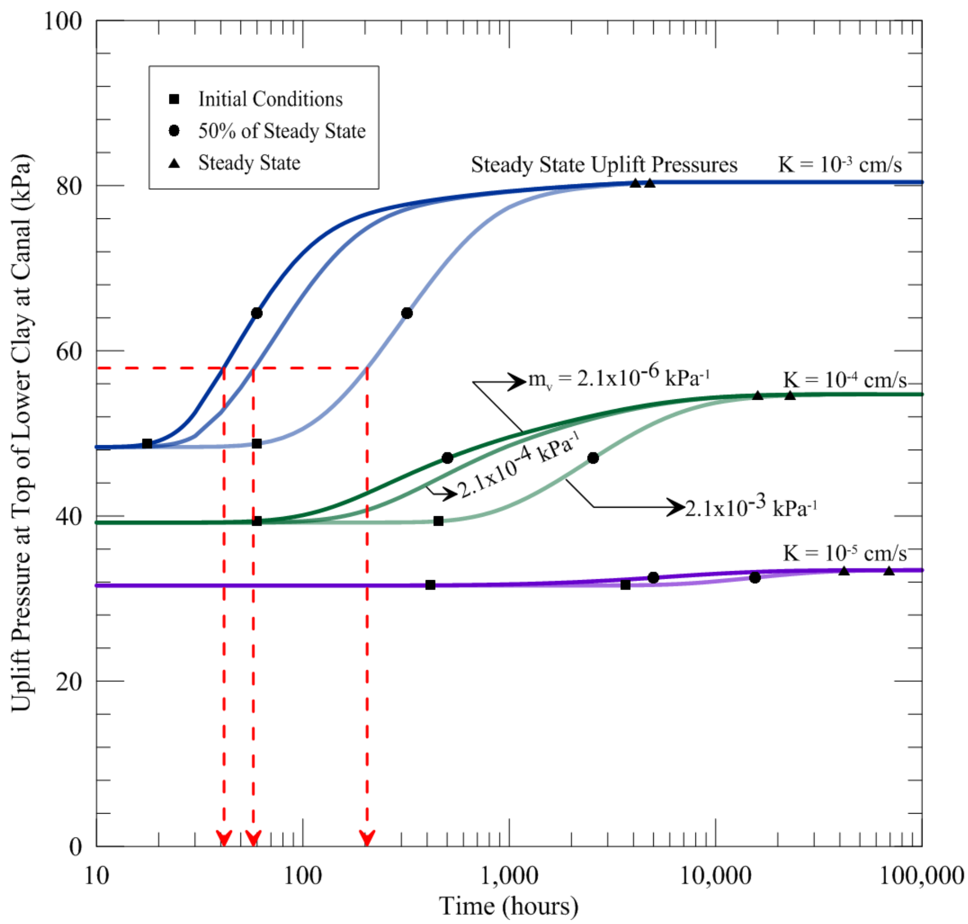


Figure 5: Effect of Lower Clay Layer coefficient of volume compressibility on time to reach steady state seepage

The results of the parametric study are shown in Fig. 5 and three distinct values of m_v are used for each value of hydraulic conductivity to illustrate the impact on seepage through the lower clay layer. The results in Fig. 5 show that as the soil compressibility decreases, i.e., soil becomes less compressible, the time to reach steady state decreases. For a given m_v value, increasing the saturated hydraulic conductivity increases the maximum uplift pressure. However, the increase in uplift pressures and effect of m_v are negligible for a k equal to or less than 10^{-5} cm/sec.

The first effect of m_v is that it delays the onset of uplift water pressure increase at the top of the lower clay layer from initial steady state conditions. For example, the square data points (see Fig. 5) indicate the time at which pore-water pressures start to increase from steady state conditions for a given hydraulic conductivity. Comparing the square data points for a given hydraulic conductivity, increasing the value of m_v increases the time it takes before the rapid increase in uplift pressures. This behavior is reasonable because m_v changes three orders of magnitude (1,000 times) for a given hydraulic conductivity, which means the right-side of Eq. (7) is reduced by the same magnitude. For example, the unit decline in total hydraulic head per time $(\partial h/\partial t)$ means the hydraulic head induced by the storm surge dissipates at a faster rate. On the other hand, a low compressibility soil for the same given time period allows less head to dissipate from the storm surge and thus results in higher uplift pressures at the landside levee toe.

Fig. 5 also shows the effect of m_v diminishes exponentially as compressibility decreases. For instance, using $k=10^{-4}$ cm/sec, the time to reach 50% of the steady state uplift pressures is 2,600 hrs, 700 hrs, and 500 hrs for $m_v=2.1\text{x}10^{-3}$ kPa^{-1}, $2.1\text{x}10^{-4}$ kPa^{-1}, and $2.1\text{x}10^{-6}$ kPa^{-1}, respectively. This highlights the critical range of m_v is from $2.1\text{x}10^{-3}$ $kPa^{-1}$to $2.1\text{x}10^{-4}$ kPa^{-1}, which corresponds to saturated clays in Table 2. This shows that selecting a reasonable m_v value for saturated fine grained soils with $k\geq10^{-4}$ cm/sec is critical to develop a representative transient seepage analysis. More importantly, the necessity of having $k\geq10^{-4}$ cm/sec indicates that fluid seepage must occur for uplift pressures to be generated. In other words, there must be flow to transmit the uplift pressures and seepage forces from the floodside to the landside of the floodwall.

Table 4 presents the vertical hydraulic gradients at a distance of 19.8 m from the floodwall, i.e., at the clay plug to the right of the levee toe. These gradients illustrate the influence of the saturated hydraulic conductivity of the lower clay layer on the vertical hydraulic gradients calculated a distance of 19.8 m from the floodwall. The vertical gradient is calculated by dividing the change in total hydraulic head at the top of the lower clay layer by the distance to the ground surface at the location the gradient is being calculated. In summary, the values of m_v assigned to the landside materials can greatly impact the calculated landside uplift pressures and vertical hydraulic gradients.

Table 4 also shows the vertical gradients for the steady state condition are significantly greater (conservative) than the transient values. In reality, steady state conditions may not develop due to rapid storm surges, e.g., hurricanes, or short duration floods. As a result, for design it is recommended that both transient and steady state analyses be conducted to understand the impact of short duration hydrographs and the coefficient of compressibility.

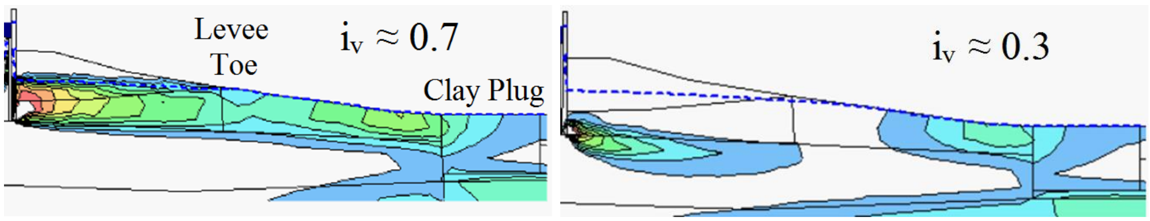
Table 4: Vertical Hydraulic Gradients 19.8 m landside of floodwall

k (cm/sec)	i_v (Initial)	i_v (50% to Steady State)	i_v (Steady State)
10^{-3}	0.54	1.08	1.56
10^{-4}	0.25	0.53	0.76
10^{-5}	0.08	0.09	0.13

Effect of Overlying Confining Layers

Fig. 5 shows the value of m_v selected for the lower clay layer, i.e., the layer below the tip of the sheetpile wall, has a large impact on the landside uplift pressures. As important, or even more important, is the value of m_v selected for the landside materials overlying the lower clay layer on the landside uplift pressures. For example, the value of m_v assigned to the levee fill or embankment material on the landside of the floodwall can impact the landside gradients by a factor of 2. Fig. 6 illustrates this effect by showing the vertical hydraulic gradient contours developed in SEEP/W. The vertical gradients at the clay plug decrease from 0.7 to 0.3 when using a reasonable value of m_v A low value of m_v (2.1×10^{-6} kPa^{-1}) results in the levee fill acting as a cap over the lower clay layer from the floodwall to the levee toe and trapping the uplift pressures at the top of the lower clay layer. Increasing the value of m_v to 2.1×10^{-3} kPa^{-1} allows some of the uplift pressures to dissipate near the floodwall through the levee fill resulting in reduced uplift pressures at a distance of 19.8 m from the floodwall by as much as 50%.

Fig. 5 shows the levee fill material on the landside of the floodwall is above the groundwater surface and thus partially saturated. As a result, the levee fill material should be assigned a compressible value, e.g., to $2.1\text{x}10^{-3}$ kPa^{-1}, of compressibility because the air voids of a partially saturated soil are compressible which is also in agreement with the material compressing when a car was driven on it. When a saturated clay is loaded, it is assumed in geotechnical engineering that water is incompressible so the entire applied load is carried by the pore-water pressures. This assumption is clearly not valid for a partially saturated soil so a compressible value of m_v, e.g., $2.1\text{x}10^{-3}$ kPa^{-1}, should be assigned.



(a) (b)

Figure 6: Effect of landside levee coefficient of compressibility on vertical hydraulic gradient (a) m_v=$2.1\text{x}10^{-8}$ kPa^{-1} and (b) m_v=$8.4\text{x}10^{-4}$ kPa^{-1}

Fig. 7 shows an excavation in the levee fill on the landside of the floodwall near the north end of the IHNC to investigate the soil adjacent to the box culvert in the foreground. The photographs in Fig. 7 show the levee fill in the upper 1.5 to 2 m is lighter color than near the bottom of the excavation indicating a partially saturated condition. Therefore, the levee fill material on the floodside of a floodwall is likely to be partially saturated and should be assigned a compressible value, e.g., to $2.1\text{x}10^{-3}$ kPa^{-1}, of m_v because of the presence of compressible air voids.

Figure 7: Excavation in landside levee fill material at the north end of the IHNC showing partially saturated nature of the fill material

The other materials on the landside of the floodwall materials above the groundwater surface have a similar impact on the uplift pressures created and trapped at the top of the lower clay layer. For example, the compressibility of the soil above the ground water surface from the levee toe to the clay plug and the clay plug also can impact the calculated uplift pressures and resulting vertical hydraulic gradients by a factor of 2. Changing the value of m_v from 2.1×10^{-6} kPa^{-1} to 2.1×10^{-3} kPa^{-1} can decrease the uplift pressures at the clay plug by as much as 50% (see Fig. 6). The higher compressibility allows the material to drain some of the pressures. Of course, increasing the saturated hydraulic conductivity of the clay plug or not applying a boundary condition across the top of the clay plug will also result in the clay plug acting as a drain and dissipating some of the uplift pressures. As a result, modeling the field conditions on the landside of the floodwall is important for estimating realistic analytical seepage results.

Location of Zone of Interest

In general, the zone of interest for erosion, and subsequently piping, is the levee toe because it exhibits the thinnest cover soil and is closest to the floodwall. However, Wolff (2002) reports that the location of sand boils can also be influenced by local geologic conditions. For example, high exit gradients and concentrations of seepage are usually found along the landside at thin or weak spots in the top stratum and adjacent to clay filled swales or channels. One such example is the large sand boils near Sun Plus Road at Mississippi River Right Levee at River Mile 228 (N 30° 25’ 45.18” W 91° 14’ 05.64”, 2 June, 2011 at 18:50) near Baton Rouge, Louisiana that developed several 100 m (Alfortish et al. 2011) from the Mississippi River levee (see Fig. 8). More than likely, local geology, e.g., preferential flow paths, caused these sand boils. Sand boils also tend to occur between levees and parallel clay-filled plugs and landside ditches. The parametric analysis also shows that a clay plug, which models a clay filled excavation or impermeable culvert, can generate higher vertical gradients in the vicinity of the clay plug. This is caused by the seepage flow being impeded by the impermeable plug and forced upwards creating large uplift pressures and vertical gradients under the clay blanket.

(a) (b)

Figure 8: (a) Overview of large sand boil and (b) close up of one of large sand boils at Sun Plus Road at Mississippi River Right Levee at River Mile 228 (N 30° 25’ 45.18” W 91° 14’ 05.64”, 2 June, 2011 at 18:50)

RECOMMENDATIONS FOR TRANSIENT SEEPAGE ANALYSES

The state of practice for levee design and remediation is still is steady state seepage conditions. However, there is interest in performing transient seepage analyses to investigate the level of conservatism with a design based on steady state seepage conditions and to calibrate the seepage model with piezometric data. The following procedure is recommended for a transient seepage analysis:

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 used to establish the initial phreatic surface through the levee via a steady state analysis.

2. Transient seepage: The initial steady state pore-water pressure regime is used as a "parent analysis" for the transient analysis and the boundary conditions are no longer constant with time. For example, Fig. 4 shows a storm hydrograph modified from Hurricane Katrina in 2005. The hydrograph is applied as the boundary condition to the floodside surface nodes. Because the flood hydrograph is not known at the time of design, i.e., the hazard event has not occurred, it is recommended to use an agreed upon maximum storm surge level and a reasonable hydrograph. By raising the flood level to maximum and then maintaining the maximum storm surge until steady state conditions develop (see Fig. 4), a parametric study similar to the one in Fig. 5 can be developed. This analysis is performed using the median or most likely value (MLV) of m_v using site specific data or values from Tables 1 and 2. Additional analyses using highest conceivable and lowest conceivable m_v values can also be performed to develop low and high bounds, respectively, of the time required to reach steady state and magnitude of uplift pressures. 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 design measures.

3. Underseepage and exit gradients: An exit hydraulic gradient of 0.85 measured in the vertical direction on the landside of a levee is commonly considered sufficient to initiate sand boil formation. Other field measurements show that sand boils may occur with exit hydraulic gradients in the range of 0.54–1.02 (Daniel 1985). Therefore, using a vertical hydraulic gradient of 0.85, the uplift pressures required to induce heave and sand boils can be back-calculated. Using a graph similar to the shown in Fig. 5 and developed in Step 2, the calculated uplift pressure for a given hydraulic conductivity and m_v can be used to estimate the time at which sand boils may develop assuming maximum flood level is approximated. For $k=1x10^{-3}$ cm/sec in Fig. 5, the uplift pressure that induces

heave and sand is 58 kPa and the time at which sand boils may develop are 42 hrs, 58 hrs, and 205 hrs for m_v=2.1x10^{-3} kPa^{-1}, 2.1x10^{-4} kPa^{-1}, and 2.1x10^{-6} kPa^{-1}, respectively. This permits levee owners and communities to monitor the levee for seepage distress and plan remedial measures.

4. Slope stability: If a drained stability analysis is being performed for the steady state condition, the uplift pressure regime developed in the transient seepage analysis can be used in a coupled analysis to evaluate levee stability. For example, the pore-water pressure regime can be imported into a 2-D slope stability program and the Factor of Safety (FS) computed. If an undrained stability analysis is being performed because of the short duration of loading and/or a low value of hydraulic conductivity or coefficient of consolidation, C_v, the transient seepage analysis results are not used in the stability analysis because an undrained shear strength is used. The methodology proposed by Duncan and Wright (2005) and Fig. 9 can be used to determine whether or not a drained or undrained stability analysis should be used.

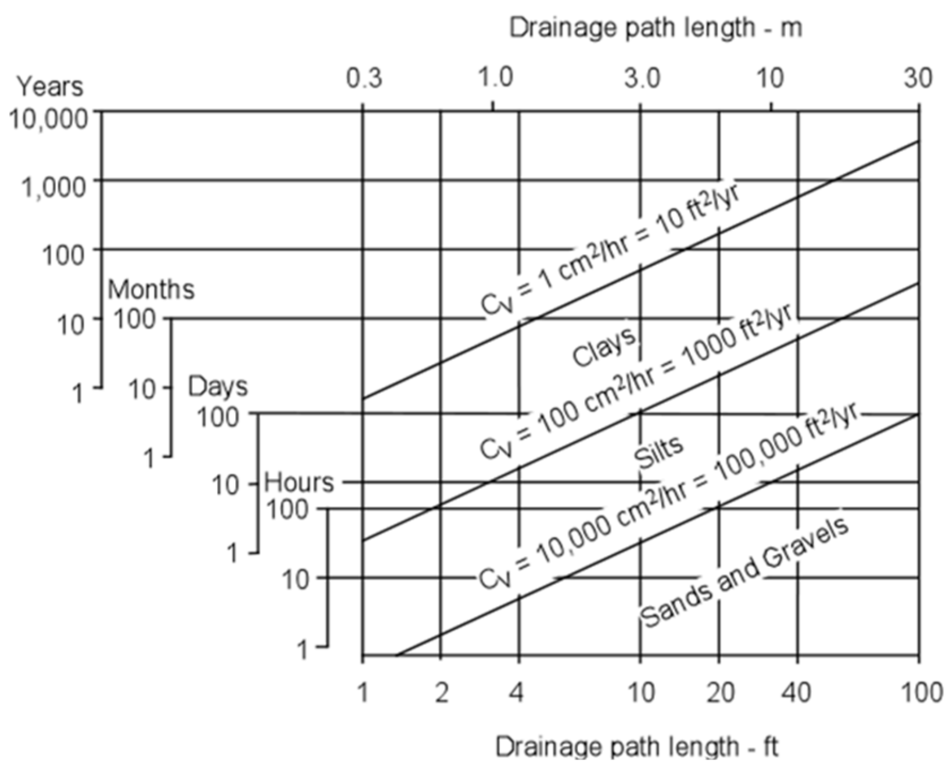


Figure 9: Time required for 99% of pore-water pressure to be dissipated (after Duncan and Wright 2005)

In summary, steady state and transient seepage each reflect certain soil, e.g., high hydraulic conductivity and compressibility, and boundary conditions, e.g., short duration flood or storm surge. The engineer must determine the appropriate soil conditions and boundary conditions and then determine which method of analysis, e.g., transient or steady state, best simulates field conditions to estimate realistic uplift pressures and hydraulic gradients.

CONCLUSIONS

This paper reviews the importance of the coefficient of compressibility on transient seepage analyses using a parametric analysis. The paper also provides guidance on selecting the value of m_v and performing transient seepage analyses. The following conclusions were derived from this analysis:

1. The derivation of transient seepage flow indicates that reducing the value of m_v, i.e., making the system incompressible, implicitly transforms the transient seepage analysis to steady state. Also, most seepage analyses assume water is incompressible. The value of m_v should not be assumed to be incompressible because m_v is a function of both soil skeleton and water compressibility. Even if water is assumed to be incompressible, the soil skeleton usually has a high compressibility especially if it is partially saturated. Therefore, a transient seepage analysis should not be converted to steady state analysis with an erroneously low value of m_v.
2. General guideline for estimating the coefficient of volume compressibility include laboratory consolidation tests, empirical correlations, soil type, field calibration using piezometers, and field pump tests. In addition, m_v is shown to vary, e.g., by an order of magnitude, for the same soil type. To account for uncertainty in the most likey m_v value, the highest and lowest values should be used in the analyses. The selected value of m_v should also be representative of the in-situ effective vertical stress.
3. The parametric analyses show that m_v affects the time at which landside uplift pressures start to increase and the magnitude of the uplift pressures. In particular, the effect of m_v diminishes as the soil becomes more compressible. As expected, fluid

flow must be present for uplift pressures to be generated on the landside of the levee or floodwall.

4. Current state of practice for levee underseepage does not require transient seepage analyses, thus making designs potentially conservative and costly. A design procedure for performing transient seepage analyses is provided that incorporates how to estimate material properties, develop initial steady state conditions before applying the flood hydrograph, and using the results to predict an approximate time at which underseepage distress may begin and the zone of interest.

ACKNOWLEDGMENTS

This material is based upon work supported by the National Science Foundation through a Graduate Research Fellowship to Navid H. Jafari. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect the views of the NSF.

REFERENCES

Alfortish, M., Brandon, T. L., Gilbert, R. B., Stark, T. D., and Westerink, J. (2011). "Geotechnical Reconnaissance of the 2011 Flood on the Lower Mississippi River," Geo-engineering Extreme Event Response (GEER) Report, National Science Foundation, 40.

Bell, F. G. (2000). *Engineering Properties of Soils and Rocks*. 4th ed., Blackwell Science, UK.

Bennett, P. T.(1946). “The effect of blankets on seepage through pervious foundations.” Trans. Am. Soc. Civ. Eng. 11, Paper No. 2270, 215–252.

Cedergren, H. R. (1989). Seepage, Drainage, and Flow Nets, John Wiley and Sons, NY, p 489.

Daniel, D. E. (1985). Review of piezometric data for various ranges in the rock island district, USACE Waterways Experiment Station, Vicksburg, Miss.

Domenico, P. A., and Mifflin, M. D. (1965). “Water From Low-Permeability Sediments and Land Subsidence.” *Water* Resources Research, American Geophysical Union, vol. 1, no. 4, 563-576.

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

Duncan, J.M. and Wright, S.G. (2005). Soil strength and slope stability. John Wiley & Sons, 297

Freeze, R. A., and Cherry, J. A.(1979). Groundwater, Prentice-Hall Inc., Englewood Cliffs, N.J.

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

Holtz, R. D. and Kovacs, W. D. (1981). An Introduction to Geotechnical Engineering, Prentice-Hall Inc., Englewood Cliffs, N.J.

Kruseman, G. P., and Ridder, N. A. (1990). *Analysis and Evaluation of Pumping Test Data*. 2nd ed., International Institute for Land Reclamation and Improvement (ILRI), Wageningen, 377.

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

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

URS JEG JV (2011). "Remediation of Floodwalls on the 17th Street Canal." OFC-05. Dept. 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, Dept. of the Army, Washington, D.C.

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

Wolff, T. F. (1986). "Design and Performance of Underseepage Controls: A Critical Review." *Report ERDC/GSL TR-02-19*, U.S. Army Corps of Engineers, Engineer Research and Development Center, (2002), Vicksburg, MS.

FIGURE CAPTIONS:

Figure 1: Comparison of saturated hydraulic conductivity and m_v for a range of soils

Figure 2: Compressibility (m_v) of deltaic clay near New Orleans (a) water content, (b) effective vertical stress, and (c) liquid limit

Figure 3: Parametric study soil profile showing floodside excavation, sheet pile wall supported floodwall, and a landside backfilled excavation

Figure 4: Hydrograph applied in parametric study

Figure 5: Effect of coefficient of compressibility on time to reach steady state seepage

Figure 6: Effect of landside levee coefficient of compressibility on vertical hydraulic gradient (a) m_v=2.1x10^{-8} kPa^{-1} and (b) m_v=8.4x10^{-4} kPa^{-1}

Figure 7: Excavation in landside levee fill material at the north end of the IHNC showing partially saturated nature of the fill material

Figure 8: (a) Overview of large sand boil and (b) close up of one of large sand boils at Sun Plus Road at Mississippi River Right Levee at River Mile 228 (N 30° 25' 45.18" W 91° 14' 05.64", 2 June, 2011 at 18:50)

Figure 9: Time required for 99% of pore-water pressure to be dissipated (after Duncan and Wright 2005)

TABLE CAPTIONS:

Table 1: Range of m_v values (after Domenico and Mifflin 1965)

Table 2: Summary m_v for clays (after Bell 2000)

Table 3: Soil properties for parametric study

Table 4: Vertical Hydraulic Gradients 19.8 m landside of floodwall