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Integrating Field Monitoring and Numerical Modeling to Evaluate Performance of a Levee under Climatic and Tidal Variations

Xavier A. Rivera-Hernandez, M.ASCE¹; Ghada S. Ellithy, M.ASCE²; and Farshid Vahedifard, M.ASCE³

Abstract: Several short-duration and extreme hydraulic loadings impose time-dependent variably saturated seepage conditions on earthen slopes and embankments. Difficulty assigning appropriate input parameters and lack of full-scale validation are among the main factors introducing uncertainty and lack of confidence when performing numerical transient seepage analysis. This case study demonstrates how to effectively use field-monitoring data to improve the numerical analysis of a levee under climatic and tidal variations. The case study includes a silty sand setback levee located near Seattle. An array of tensiometers and soil moisture sensors installed within the levee's embankment and foundation, along with a nearly real-time data acquisition system, were used to collect and process the in situ data for a period of about 15 months. Climatic and weather data, including precipitation, temperature, humidity, and wind speed, were collected from a weather station at the site, and tidal water fluctuations were monitored using a water level sensor. The field-measured matric suction and water content were used along with a suction stress-based representation of effective stress to compute suction stress and effective stress profiles versus time. A finite-element model of transient seepage under saturated-unsaturated conditions was developed. The numerical model was calibrated and then validated using the measured pore-water pressures and the piezometric surface. The application of the numerical model was illustrated by modeling the seepage and stability of the levee during a 100-year flood event. The results highlight the need to consider climatic variables and soil-atmosphere interaction when performing transient seepage analysis. DOI: 10.1061/(ASCE)GT.1943-5606.0002134. © 2019 American Society of Civil Engineers.

Author keywords: Levee; Case study; Transient seepage; Unsaturated soil; Field monitoring; Numerical modeling; Effective stress.

Introduction and Background

Assessing the performance and stability of earthen structures during their service life warrants an improved understanding of the behavior of variably saturated soils under transient seepage conditions. The need is more pronounced considering recent climatic trends, which have led to more frequent and severe extreme events. Climate change has caused considerable changes in intensity, frequency, and duration of extreme precipitation events in several parts of the world (IPCC 2013; USGCRP 2009). In the United States, an increase of 20% in the amount of heavy precipitation was reported from 1958 to 2007 (USGCRP 2009). In several regions, heavier rainfalls occur over shorter periods, posing a real flash-flood threat to the levees in those regions due to the huge quantity of runoff produced in a short time. Ragno et al. (2018) used a nonstationary model and showed that major cities in the United States might experience extreme precipitation events up to 20% more intense and twice as

frequent compared with historical records. The aforementioned changes in the statistics of extreme precipitation and flood probability can affect the stability of earthen structures through changes in degree of saturation (e.g., Vahedifard et al. 2016b; Robinson et al. 2017; Jasim et al. 2017; Vahedifard et al. 2017a, b).

Several short-duration and extreme hydraulic loadings (e.g., flooding, extreme precipitations, and rapid drawdown) can impose time-dependent seepage conditions on earthen slopes and embankments. Numerical modeling of an earthen structure subjected to such scenarios warrants performing transient seepage analysis under variably saturated conditions. Previous attempts in this area primarily involved transient saturated seepage analyses of rapid drawdown in earthen dams (e.g., Freeze 1971; Stark and Duncan 1991). Improved understanding of the theory along with continuous advances in numerical modeling of flow in porous media have extended the realm of transient seepage analysis to unsaturated soils. Some example applications include modeling rainfall-triggered landslides in natural slopes (e.g., Godt et al. 2012; Leshchinsky et al. 2015), analysis of saturated-unsaturated rapid drawdown in earthen dams (e.g., Stark et al. 2016), and simulation of the hydromechanical behavior of unsaturated earthen structures under extreme precipitation and flood events (e.g., Jasim et al. 2017; Vahedifard et al. 2018).

A group of previous transient seepage analyses was performed by solving the governing equation of unsaturated flow (i.e., Richards' equation) using an analytical solution or numerical method (e.g., finite element, finite difference) and then incorporating the seepage analysis results into a limit equilibrium or limit analysis of the earthen structure (e.g., Godt et al. 2012; Stark et al. 2016). This group required fewer input parameters and computational resources but still provided reasonable results for several applications (e.g., Godt et al. 2012; Stark et al. 2016). Alternatively,

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coupled hydromechanical simulations are performed using more-advanced numerical models to account for the two-way interaction between solid and fluid (e.g., [Jasim et al. 2017](#); [Vahedifard et al. 2017b](#)). This coupling aspect becomes particularly important for models in which the soil compressibility is considerable or the change in total stress is significant. The coupled models involve more-rigorous computational efforts, but with appropriate input parameters and model settings they can lead to more-accurate results.

Considerable research advances have been made in various aspects of variably saturated transient seepage. Some of these research findings have been translated to the state of the practice. For example, several commercial software programs have been developed which are designated to facilitate variably saturated transient seepage analysis. However, the geotechnical engineering community has been relatively hesitant to fully embrace transient seepage analysis in practice. Difficulty assigning appropriate input parameters and boundary conditions is among the main factors introducing uncertainty and lack of confidence when performing numerical transient seepage analysis. Furthermore, lack of full-scale validation of results and poor connection to field monitoring data undermine the validity and accuracy of such numerical analyses for practicing engineers. There is also the general belief that the steady-state condition constitutes the most severe design case. However, sound engineering practice requires site-specific analyses to determine the circumstances and loading conditions under which transient conditions represent a more critical case.

This case study paper demonstrates how to effectively use field-monitoring data to improve the numerical analysis of a levee under climatic and tidal variations. The case study includes a silty sand setback levee near Seattle. The study area, levee section, and instrumentation plan are discussed. The field-measured matric suction and water content are used along with a suction stress-based representation of effective stress to compute suction stress and effective stress profiles versus time. A finite-element model of transient

seepage under saturated-unsaturated conditions is then developed and calibrated to reasonably match the measured pore-water pressures and the piezometric surface. The application of the numerical model is illustrated by modeling the seepage and stability of the levee during a 100-year flood.

Study Site: Qwuloolt Levee

Location and Geology

Approximately 162 ha (400 acres) of restored estuarine wetland in Washington form one of the largest ecosystem restoration projects in the United States. The project site is located about 64 km (40 mi) north of Seattle. As part of this project, a 1,219-m-long (4,000 ft) setback levee, the Qwuloolt levee, was constructed in 2014 to protect an industrial area while allowing for inundation of the site with Puget Sound water. Fig. 1 shows the study area, located in the city of Marysville, Snohomish County, Washington, with the Puget Sound and the Central Cascade mountain range bounding it to the east and west, respectively. The project borders include the city's sewage treatment plant to the west, an industrial park to the northwest, and residential neighborhoods along 61st Street NE and Sunnyside Boulevard to the north and east, respectively.

In general, the regional ground surface elevation gradually slopes from 48 m (160 ft) in the north to 1.5 m (5 ft) in the southern parts of Marysville. The study area has an average ground surface elevation of about zero. The climate is defined as temperate/mesothermal, with an average annual rainfall of 1,043 mm (41 in.). The topsoil extends down to 1 m (3.3 ft) is classified as a hydric Puget silt loam ([Anderson et al. 1947](#)), and is representative of the levee and the surrounding area. The initial soil horizon extends to 152 mm (6 in.) depth and is a brownish-gray to gray smooth friable silt loam with relatively high organic matter content ([Anderson et al. 1947](#)). The subsoil has a noticeable laminated structure and

Fig. 1. Map of study area and levee section. (Map data © 2018 Google.)

Fig. 2. Instrumented section of levee (Station 36 + 10).

is a light gray silt loam to a depth of 508 mm (20 in.). Between 508 and 1,016 mm (20–40 in.), the soil is a brownish-gray silty clay loam with a layered structure. Stratified layers of yellowish-brown sandy loams and fine sands with a greenish-gray tint underlie the silty clay loam horizon (Anderson et al. 1947).

Field Instrumentation and Monitoring Plan

The construction of the levee started in 2012 and ended in 2014. The instruments were installed August 10–17, 2015. Fig. 2 depicts a cross section of the levee (Station 36 + 10), which is instrumented with different sensors for field-monitoring purposes. The sensors were manufactured by METER Group. Sensors were installed by placing them at the desired depth in hand-augured boreholes and surrounding them with compacted soil. A PVC pipe was used to keep the borehole stable. After installing the sensors, the boreholes were sealed with bentonite slurry and PVC caps. Soil samples were collected at different intervals from all holes.

Fig. 2 shows the instrumentation points within the levee embankment, consisting of (1) six points labeled Q1–Q6 identifying the colocation of tensiometers (T8) and volumetric water content sensors (GS3), (2) one point labeled Piezo4.75 identifying the location of a vibrating wire piezometer, and (3) one point labeled CDT-10 identifying the water gauge. Each instrumentation point includes the instrument elevation in parenthesis.

The T8 tensiometer consists of a combined sensor for matric suction and temperature for long-term monitoring purposes. This tensiometer has a working range of -85 to 100 kPa pore-water pressure with a soil water tension accuracy of ± 0.5 kPa (METER Group 2018). A porous ceramic cup filled with water is in direct contact with the soil water. The soil water tension is measured by a pressure transducer. The GS3 sensor measures the volumetric water content, electrical conductivity, and temperature of soil. The GS3 sensor uses an electromagnetic field to measure the dielectric permittivity of the surrounding medium, which is converted to water content by a calibration equation.

Water Gauge CTD-10 is placed in a trench close to the levee toe on the water side. The water gauge was placed in a perforated 10-cm PVC pipe wrapped in a protective textile sleeve below the low water table. The trench was then backfilled with filter rock. The CDT-10 sensor measures the tidal water level, electrical conductivity, and water temperature. The water depth resolution is about 2 mm, and accuracy is $\pm 0.05\%$ of full scale at 20°C (METER

Group 2018). In addition, a weather station at the site is used to obtain the weather parameters, consisting of precipitation, temperature, relative humidity, and wind speed. Solar radiation data were obtained from a publicly available database operated by Washington State University from the Langley weather station located on Whidbey Island about 24 km from the site.

The data from weather sensors are collected and reported by their data logger every 1 and 6 h. Both sensors and data logger are powered by a solar panel. The soil sensors are connected to the data loggers by underground cables to protect them from any damage. The soil data loggers (METER Em50) have their own solar-charged batteries which help to maintain the power in the system. A modem in each data logger is used to collect and transmit the data using METER's software.

Instrumentation Readings

Figs. 3–6 show the soil sensor recordings and climatic data from September 2016 to November 2017. Pore-water pressure and volumetric water content versus time are shown in Fig. 3. Fig. 4 depicts climatic data collected at the site, including air temperature Fig. 4(a), relative humidity Fig. 4(b), precipitation Fig. 4(c), solar radiation Fig. 4(d), and wind speed Fig. 4(e). Fig. 5 shows the data obtained from Piezo4.75. Fig. 6 shows the tidal hydrographs with 6- and 24-h data. The tidal and piezometer readings were converted to total head for comparison purposes.

Fig. 3 depicts the pore-water pressures measured using T8 tensiometers and the volumetric water contents measured using GS3 sensors at instrumentation points Q2 Fig. 3(a), Q5 Fig. 3(b), and Q6 Fig. 3(c). These sensors were deployed at elevations of 3.20 m for Q2, 2.90 m for Q5, and 1.07 m for Q6. The water table on the downstream side is close to the ground surface (i.e., elevation zero) most of the time.

The pore-water pressures measured using the T8 tensiometers and the volumetric water contents measured using the GS3 sensors exhibited similar trends. The initial 2 months were the end of a drought period [Fig. 4(c)] during which both pore-water pressure and water content were low. From October 2016 to June 2017, the measured water content increased in all three GS3s, which can be attributed to the rainy season during winter and spring of 2017. Pore-water pressures within the same period remained almost near zero or slightly higher in all three T8 tensiometers, demonstrating an increase from the previous drought period. The second

Fig. 3. Pore-water pressure and volumetric water content versus time measured using T8 tensiometers and GS3 sensors at (a) Q2; (b) Q5; and (c) Q6.

dry period was from June 2017 to September 2017, during which both pore-water pressure and water content values decreased at all three instrumentation points. The time lag in the matric suction response was estimated to be 2 to 3 weeks.

The GS3 sensors recorded a change in volumetric water content during the period between September 2016 and November 2017, when precipitation [Fig. 4(c)] caused a change in the volumetric water content from 0.22 to 0.38 (m^3/m^3) (Fig. 3). The water content recorded at Q5 remained constant during that period until October 2016, when it changed from 0.21 to 0.38 (m^3/m^3). All volumetric water content measurements decreased during the drought period of June 2017 to September 2017. For the period of study, the saturated volumetric water content was the highest value recorded by the sensors ($0.39 \text{ m}^3/\text{m}^3$), and the lowest recorded volumetric water content was $0.18 \text{ m}^3/\text{m}^3$.

The five climatic data sets in Fig. 4 were used to define the land-climate interaction boundary condition in saturated-unsaturated transient seepage analysis. The air temperature overall fluctuated, decreasing and increasing during the study period [Fig. 4(a)]. The

relative humidity [Fig. 4(b)] fluctuated substantially during the day but returned to the same baseline. Fig. 4(c) shows the precipitation data during the study period of September 2016 to November 2017. Peak recorded precipitation was as high as 12 mm, with an average of 2 mm over 6 h. Fig. 4(d) depicts the solar radiation, defined as the radiant energy emitted by the sun, at the site during the study period. The radiation varied from 300 to $1,000 \text{ W}/\text{m}^2$ in the higher range, with a lower value of $100 \text{ W}/\text{m}^2$.

Fig. 5 shows the total head versus time obtained from the piezometer deployed within the levee's embankment (Piezo4.75) for the period of study. The trend shows that the piezometer readings are affected substantially by changes in the tide and precipitation, particularly after December 7, 2016. Fig. 6 depicts the 6- and 24-h data collected from Water Gauge CDT-10 from September 2016 to November 2017. The 24-h data are presented as a clearer trend line of the water tide. The tide elevations varied during the day but followed a constant pattern during the 15-month period, with high tide ranging from 2 to 2.5 m, and low tide approximately 1 m.

Fig. 4. Climatic data: (a) air temperature; (b) relative humidity; (c) precipitation; (d) solar radiation; and (e) wind speed.

Laboratory Testing

Representative soil samples were collected from instrumented locations of the levee's embankment and were shipped to the U.S Army Engineer Research and Development Center (ERDC) in

Fig. 5. Total head versus time obtained from the piezometer within the levee's embankment (Piezo4.75).

Fig. 6. Tidal hydrograph with 6 and 24h data.

Vicksburg, Mississippi for testing. The soil samples were generally classified as silty sand (SM) as per the Unified Soil Classification System (USCS).

The soil water retention curves (SWRCs) of the embankment soil were obtained by running multiple tests using the transient water release and imbibition method (TRIM) on soil samples collected from Instrumentation points Q2–Q6. Wayllace and Lu (2012) developed TRIM to provide a fast, accurate, and simple testing tool for obtaining the SWRC for wetting and drying states under no confinement conditions. The method uses an inverse modeling technique to develop the full curve from two matric suction pressures (10 and 290 kPa for this study) that are applied to the soil. The TRIM results were used to obtain the van Genuchten (1980) SWRC model, which is defined as

$$\left(\frac{\theta - \theta_r}{\theta_s - \theta_r} \right) = S_e = \left[1 + \left(\frac{\psi}{a} \right)^n \right]^{\left[\left(\frac{n}{n-1} \right) - 1 \right]} \quad (1)$$

where θ , θ_r , and θ_s = total volumetric water content, residual volumetric water content, and saturated volumetric water content, respectively; S_e = effective saturation; ψ = matric suction (kPa); and a (kPa) and n (unitless) = fitting parameters which are dependent on the air entry suction and pore size characteristics, respectively.

Fig. 7 shows the SWRCs for the tested soils under drying and wetting paths. Using the TRIM results, the a parameter had large variability, with a minimum of 13 kPa, a maximum of 250 kPa, and an average of 64 kPa. The n values obtained from TRIM varied within a narrow range, with a minimum of 1.2, a maximum of 2.2, and an average of 1.6. An average saturated hydraulic conductivity of $k_{sat} = 3 \times 10^{-5}$ cm/s was obtained from the tests.

Fig. 7. SWRCs of tested soils under drying and wetting paths.

Fig. 8. Suction stress and effective stress versus time at Q2 and Q6.

Monitoring Suction Stress and Effective Stress Using Field Data

This section demonstrates the use of in situ data consisting of pore-water pressure and water content to compute effective stress versus time. Effective stress is a critical parameter that dominates the shear strength and, subsequently, the stability of earthen structures (e.g., Lu and Likos 2004; Vahedifard et al. 2016b). Pioneered by Bishop (1959), several studies have tried to properly characterize the effective stress of unsaturated soils. The present study used the suction stress-based effective stress representation (Lu and Likos 2006; Lu et al. 2010), along with the field-measured data, to compute the suction stress and effective stress time series within the unsaturated zone of the levee embankment under seasonal and tidal changes.

Lu and Likos (2004) extended Bishop's expression for the effective stress of unsaturated soils by adopting the suction stress concept. The unified effective stress for both saturated and unsaturated soil conditions can be defined as (Lu and Likos 2004, 2006)

$$\sigma' = (\sigma - u_a) - \sigma^s \quad (2)$$

where σ' = effective stress; σ = total stress; u_a = pore-air pressure; and σ^s = suction stress. Suction stress for unsaturated soils can be determined by Lu et al. (2010)

$$\sigma^s = -\frac{\theta - \theta_r}{\theta_s - \theta_r} (u_a - u_w) = -S_e (u_a - u_w) \quad (3)$$

where u_w = pore-water pressure; and the difference $(u_a - u_w)$ represents the matric suction, ψ .

The saturated volumetric water content (porosity) of the levee embankment was taken as $0.39 \text{ m}^3/\text{m}^3$, which is the highest value recorded by GS3 sensors. Due to the process of sensor installation, the soil surrounding the sensors is looser and has a higher porosity than the embankment's soil, which is about $0.2 \text{ m}^3/\text{m}^3$. The residual volumetric water content was determined by laboratory testing to be $0.08 \text{ m}^3/\text{m}^3$. These values for the saturated and residual volumetric water contents were used along with the volumetric

water contents recorded by G3 sensors to obtain the effective degree of saturation. The computed effective degree of saturation and the matric suction values recorded by T8 tensiometers in Eq. (3) were used to generate suction stress time series (Fig. 8). The computed suction stress values were used along with Eq. (2) to generate the effective stress time series at Q2 and Q6 (Fig. 8). The total stress was calculated using the unit weight and the depth of the soil, which were 21.13 kN/m^3 and 0.91 m , respectively.

September–November 2016 was the end of the first drought period and the beginning of the rainy season, which increased suction stress from -70 to 0 kPa while decreasing the effective stress from 85 kPa to approximately 20 kPa (Fig. 8). In the dry period from end of June 2017 to the end of September 2017, the suction stress decreased substantially, and therefore the effective stress increased from approximately 20 to 80 kPa at Q2 and from 20 to 100 kPa at Q6. This significant increase of 4–5 times contributed to an increase in the shear strength of the soil. In these calculations, the total stress was assumed to be constant over the monitoring period. Both sensors had a consistent response, because they are located at similar depths from the ground surface.

The suction stress-based effective stress can be used in the Mohr–Coulomb failure criterion to determine the shear strength of both saturated and unsaturated soils. Lu et al. (2010) validated the suction stress-based representation of effective stress with experimental data of 20 different soils from the literature and mathematically showed that it continuously and smoothly transitions from the unsaturated state to the saturated state and vice versa. For a saturated soil, the suction stress-based effective stress representation degenerates to Terzaghi's effective stress equation. Using the suction stress-based effective stress approach avoids the need for developing new shear strength failure criteria for unsaturated soils.

Incorporating the suction stress-based effective stress into the Mohr–Coulomb failure criterion defines the shear strength of unsaturated soils as

$$s = c' + \left[\sigma - u_a + \frac{\theta - \theta_r}{\theta_s - \theta_r} (u_a - u_w) \right] \tan \phi' \quad (4)$$

where s = shear strength; c' = effective cohesion; and ϕ' = effective angle of friction. Using two independent stress state variables, Vanapalli et al. (1996) proposed a similar equation to quantify the shear strength of unsaturated soils by using the SWRC and saturated shear strength parameters (c' and ϕ'). Several slope stability software packages, such as SLOPE/W used in the present study, use this relationship to determine the shear resistance along the unsaturated part of slip surfaces. In these limit equilibrium-based slope stability programs, one can specify a SWRC model (e.g., van Genuchten 1980) to be used in the shear strength determination based on the Eq. (4). Water content and suction can be obtained from computed values imported from a seepage analysis program (e.g., SEEP/W) or user-input values (e.g., from field-measured data). This paper used the former approach (i.e., computed values imported from SEEP/W) along with the Vanapalli et al. (1996) shear strength equation to perform a set of saturated-unsaturated slope stability analyses using SLOPE/W. However, the results in Fig. 8 are still of practical interest, because they demonstrate how field monitoring data can be directly used to compute suction stress and effective stress, which are key parameters for performing stability analysis of variably saturated soils. Godt et al. (2012) and Lu et al. (2012) provided examples for the direct use of such profiles in slope stability analysis.

Numerical Modeling of Transient Seepage

Theory and Governing Equation

The commercial 2D finite-element software package (SEEP/W) was used to develop the numerical model. SEEP/W is a general seepage analysis program that can model saturated-unsaturated transient seepage with land-atmospheric coupling at the ground surface. Derived based on the conservation of mass, the following governing equation is used in the SEEP/W formulation

$$\frac{\partial}{\partial x} \left(k_x \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left(k_y \frac{\partial h}{\partial y} \right) + Q = m_w \gamma_w \frac{\partial h}{\partial t} \quad (5)$$

where k_x and k_y = hydraulic conductivity in the x - and y -directions, respectively; h = total head (i.e., elevation head plus pressure head); Q = applied boundary flux; m_w = coefficient of volume compressibility; γ_w = unit weight of water; and t = time. This study assumed the hydraulic conductivity to be isotropic (i.e., $k = k_x = k_y$). For unsaturated soils, the van Genuchten (1980) SWRC was used to relate the matric suction head (pressure head) to the water content under climatic and tidal variations. We used the following hydraulic conductivity function (HCF) (Mualem 1976; van Genuchten 1980) to define the hydraulic conductivity at different levels of saturation:

$$k = k_s S_e^{0.5} \left[1 - \left(1 - S_e^{n/(n-1)} \right)^{(1-1/n)} \right]^2 \quad (6)$$

A key aspect of the present study is demonstrating how to use field monitoring data to perform saturated-unsaturated transient seepage analysis while considering atmospheric coupling. For this purpose, climatic data collected at the site were incorporated into SEEP/W's land-climate interaction (LCI) boundary condition to simulate water transfers at the surface given the collected climate data. The input climate data needed to use the LCI coupling are five data sets of air temperature, precipitation, relative humidity, wind speed, and net radiation. SEEP/W considers the water fluxes due to

rainfall, snow melt, evaporation, and runoff in a mass balance equation to obtain the infiltration flux. The fluxes due to evaporation and runoff go out of the domain and have negative sign in the mass balance equation. The infiltration is the residual of the mass balance equation and forms the boundary condition of the water transfer equation. The Penman-Wilson method (Wilson et al. 1997) is used to calculate the evaporation flux as a function of relative humidity, net radiation, and wind speed.

The LCI boundary condition was defined by using the climatic data in Fig. 4. Fig. 4(d) shows solar radiation, whereas SEEP/W requires the user to input net radiation. One can calculate the net radiation using the solar radiation resulting from the balance between the incoming and reflected solar radiations as

$$R_{ns} = R_s - \alpha R_s = (1 - \alpha) R_s \quad (7)$$

where R_{ns} = net radiation; α = albedo or canopy reflection coefficient; and R_s is the solar radiation. The ASCE *Standardized Reference for Evapotranspiration* (Allen et al. 2005) recommends an albedo reflection coefficient value of 0.23. This α value was used in this study to calculate the net radiation from the solar radiation in Fig. 4(d). Allen et al. (1994) stated that albedo varies somewhat with time of day, time of season, and latitude due to the change in sun angle. However, because the solar intensity is lower during these periods, the error introduced in fixing the albedo reflection coefficient at 0.23 is relatively small (Allen et al. 1994).

Geometry, Boundary Conditions, and Modeling Stages

Fig. 9 shows the geometry and boundary conditions of the model used in the simulation. Table 1 presents the parameters used in the seepage analysis. The model consists of three materials. The foundation of the levee is a silt layer, which is assumed to be saturated at all times due to the high ground water table elevation. The silty sand layer, which comprises the levee embankment, is assumed to be the only layer that has variable saturation conditions depending on the hydraulic loading and climatic conditions. The gravel layer between the silt and silty sand represents the gravelly road section built to support the construction equipment (Fig. 9). The finite-element seepage model was meshed using 7,011 four-node elements.

A steady-state seepage analysis was performed to set the initial pore-water pressures and the piezometric surface for the subsequent transient seepage analyses. The upstream side was assigned a total head of 1.68 m, which reflects the water tide level on September 7, 2016. The piezometer located in the levee's embankment (Piezo4.75) was assigned a total head of 1.46 m at the same time. The downstream toe was set to a total head of zero, which is the ground surface elevation. The bottom boundary of the model was set as a no-flow boundary condition. Fig. 10 shows the pore-water pressure contours generated using the steady-state seepage analysis.

After performing the steady-state seepage analysis, transient seepage analyses were performed by imposing the time-dependent hydraulic boundary loads including the LCI boundary condition (Fig. 4) and the 6-h water tide data (Fig. 6). Precipitation, temperature, wind speed, relative humidity, and net radiation time series were used to define the LCI boundary condition versus time.

The transient seepage modeling efforts include three distinct stages: (1) calibration of the seepage model using the measured data for a period of 10 months from September 7, 2016 to July 7, 2017 (0–7,000 h); (2) validation of the seepage model using the measured data from July 7, 2017 to November 27, 2017 (7,000–10,704 h); and (3) application of the calibrated model to perform seepage and slope stability analysis of the levee under a 100-year flood event (10,700–10,797 h). Results from each

Fig. 9. Geometry and boundary conditions of numerical model.

Table 1. Parameters used in seepage analysis

	Material	k_s (cm/s)	m_v (kPa ⁻¹)	θ_s	θ_r	a (kPa)	n
T1:1	Silty sand	Varies	2.09×10^{-5}	0.2	0.08	Varies	1.6
T1:2	(embankment levee)						
T1:3	Silt (foundation)	10.0×10^{-6}	2.1×10^{-4}	0.4	—	—	—
T1:4	Gravel	10.0×10^{-8}	1×10^{-6}	0.3	—	—	—

modeling stage are presented and discussed in the following sections. In the third stage, the 100-year flood hydrograph was applied as a hydraulic boundary load.

Calibration of Numerical Model

The pore-water pressure generated by the transient seepage analysis was used to calibrate the model parameters based on the measured data from the piezometer (Piezo4.75) and T8 tensiometers at Q2, Q5, and Q6. The measured data for about 10 months from September 7, 2016 to July 7, 2017 (7,000 h) were used for model calibration purposes.

The calibration effort started by varying the following four parameters for the levee embankment in an attempt to reasonably match the field measurements: the SWRC fitting parameters (a and n), the coefficient of volume compressibility (m_v), and the saturated hydraulic conductivity (k_s). The parameters of the foundation soil and the gravel layer were kept constant. The different values for

n , a , and k_s examined in the calibration process were selected based on the TRIM results. For m_v , we used the recommended values for similar soil types reported in the literature. For the range of parameters examined, the calibration results showed that varying n and m_v did not have considerable effects on the results. This can be attributed to the limited range of values used for these two parameters. Subsequently, their values were set to be constant ($n = 1.6$ and $m_v = 2.09 \times 10^{-5}$ kPa⁻¹) and the calibration continued by varying a and k_{sat} . In each calibration attempt, only one parameter was varied and the rest of parameters were kept constant. The base model used $a = 13$ kPa and $k_s = 3 \times 10^{-5}$ cm/s, which were obtained by averaging the laboratory test results for each variable. In each calibration attempt, only one parameter was varied and the rest of parameters were kept constant.

The base model parameters were used along with different boundary conditions to illustrate the importance of considering the LCI boundary condition. Three scenarios for boundary conditions were examined: (1) only tide, (2) tide plus precipitation, and (3) tide plus LCI boundary condition. The pore-water pressures generated using the numerical model were compared with the field data recorded using Piezo4.75 [Fig. 11(a)]; the T8 tensiometer, and the GS3 sensor at Q2 [Fig. 11(b)]; the T8 tensiometer and GS3 sensor at Q5 [Fig. 11(c)]; and the T8 tensiometer and GS3 sensor at Q6 [Fig. 11(d)]. The van Genuchten (1980) SWRC model was used to calculate the pore-water pressures corresponding to the volumetric water content data recorded by GS3 sensors (Ellithy 2017).

Fig. 10. Initial pore-water pressures generated using steady-state seepage analysis.

Fig. 11. Effect of different boundary conditions on the performance of the numerical model to match the field data measured from September 7, 2016 to July 7, 2017 using (a) Piezo4.75; (b) T8 tensiometer and GS3 sensor at Q2; (c) T8 tensiometer and GS3 sensor at Q5; and (d) T8 tensiometer and GS3 sensor at Q6.

The inclusion of the LCI boundary condition in the numerical model resulted in pore-water pressures better matching the measured field compared with the tide-only and the tide-plus-precipitation boundary conditions (Fig. 11). The model results began to conform to the field measurements after approximately 3 months (2,000 h). This could be attributed to the time required by the numerical model to generate the adequate initial conditions for the subsequent periods. The piezometer (Piezo4.75) was installed in the beginning of the modeling period, and it took approximately this long to acclimate to the levee embankment conditions.

There was a dry event before the beginning of the modeling period, which resulted in an initially high matric suction measurements by the tensiometers (Fig. 11). The model prediction did not match the tensiometer readings in the first 1,000 h (41 days), which was primarily the result of the selected initial pore-water pressure. However, the comparison of the predicted and measured pore-water pressures improved significantly toward the latter part of the simulation. This emphasizes the importance of selecting the model duration period for calibration and adequate initial conditions. Toward the end of the calibration period there was a longer dry period, which resulted in a significant decrease in the pore-water pressure at all monitored locations.

The rest of the calibration simulations were performed using the model with tide-plus-LCI boundary conditions (Figs. 12 and 13). Three values for the SWRC parameter a were examined: 13, 64, and 150 kPa. Fig. 12 shows the effect of a on the performance of the numerical model to match the pore-water pressures recorded at Piezo4.75, Q2, Q5, and Q6. A value of $a = 13$ kPa, which was the minimum value obtained from the TRIM results, led to a closer match with the field measurements. Higher a values of 64 and 150 kPa indicate a less drainable soil, leading to lower simulated pore-water pressures than the measured values most of the time. In general, the field measurements indicated higher matric suctions and pore-water pressures during dry and wet periods, respectively, than did the numerical model.

Fig. 13 shows the effect of the saturated hydraulic conductivity on the predicted pore-water pressures recorded at Piezo4.75, Q2, Q5, and Q6. The calibration results showed no significant difference in the predicted pore-water pressure. A value of $k_s = 3 \times 10^{-5}$ cm/s was selected for use in the validation period.

Validation of Numerical Model

To assess the predictive capability of the model, we used the calibrated model to predict the field-measured data during the period from June 7, 2017 to November 27, 2017 (7,000–10,704 h). Fig. 14 shows the validation results for the selected parameters. Based on the results of the calibration process, $a = 13$ kPa and $k_s = -3 \times 10^{-5}$ were assigned to the embankment soil in the validation process. The rest of the parameters are listed in Table 2.

Fig. 14(a) shows the predicted pore-water pressure against the recorded values at Piezo4.75. The measured data indicated higher pore-water pressures than did the predicted values during the validation period. The model considers the LCI boundary condition for the dry period, whereas Piezo4.75 did not read below zero and remained closer to the zero value. The model prediction matched the trend of the pore-water pressure measured at Q2 and Q5 during the drought period, during which a sudden decrease in the pore-water pressure occurred [Figs. 14(b and c)]. The maximum suction value predicted by the model was relatively close to the measured value but with some time lag. The model underpredicted the measured suctions from Q6 [Fig. 14(d)]. The high suctions measured at Q6 were possibly due to a drainage layer (gravel layer) under the sensor that may be draining the water away. This layer was not considered in the numerical model.

The model seems to match the trend and the approximate value of pore-water pressure during the dry period. However, the measured pore-water pressures suddenly decreased over a short period, which was not consistent with the gradual response of the pore-water pressures predicted by the numerical model. Overall, the validation results suggest that there is promise for using saturated-unsaturated transient flow analyses to more accurately predict pore-water pressure representing real field conditions under various loadings. The predictive accuracy of the numerical model possibly

Fig. 12. Effect of the SWRC parameter a on the performance of the numerical model to match the field data measured from September 7, 2016 to July 7, 2017 using (a) Piezo4.75; (b) T8 tensiometer at Q2; (c) T8 tensiometer at Q5; and (d) T8 tensiometer at Q6.

Fig. 13. Effect of saturated hydraulic conductivity on the performance of the numerical model to match the field data measured from September 7, 2016 to July 7, 2017 using (a) Piezo4.75; (b) T8 tensiometer at Q2; (c) T8 tensiometer at Q5; and (d) T8 tensiometer at Q6.

could be improved, especially for the cases in which a sharp decrease in the pore-water pressure occurred, by incorporating the effects of some additional factors not considered in the model. Those include, but are not limited to, anisotropy in hydraulic conductivity, looser soil density around sensors, and layers with higher hydraulic conductivity.

Application: Modeling Levee under 100-year Flood

The application of the calibrated model was illustrated by modeling the seepage and stability of the levee during a 100-year flood.

Transient seepage analysis was first performed by applying the 100-year flood hydrograph (Fig. 15) as a boundary load. The pore-water pressure values obtained from the seepage analysis were then used in a set of limit-equilibrium analysis to obtain the factor of safety of the levee.

Seepage Analysis

To simulate an extreme flood even, a 100-year flood hydrograph model for the levee site was constructed using the data reported in Tetra Tech (2013) for Snohomish County and FEMA flood maps. The 100-year event hydrograph (Fig. 15) increased the tide

Fig. 14. Model validation results from June 7, 2017 to November 27, 2017.

Table 2. Parameters used in slope stability analysis

Property	Silty sand (embankment levee)	Silt (foundation)	Gravel
Unit weight (γ)	21.1 kN/m ³	18.1 kN/m ³	20.4 kN/m ³
Cohesion (c')	0	0/21.5 ^a kPa	0
Friction angle (ϕ')	38°	25°/0° ^a	32°

^aShear strength parameters used for undrained loading of 100-year flood.

to an elevation of 4.25 m within 62 h, whereas the regular hydrograph used in the model had a high elevation 2.5 m. A 100-year precipitation depth of about 165 mm (6.5 in.) was used for the event period, which was about 4 days.

The 100-year flood was introduced in the model immediately after the 15-month calibration and validation periods. The model

Fig. 15. 100-year flood hydrograph for the study area.

predicted pore-water pressures before and during the 100-year flood at different locations (Fig. 16). A sudden increase occurred in the pore-water pressure in all the monitored locations, including the piezometer. This increase was a result of the high 100-year event tide, which was in addition to a moderate precipitation of about 1.8 mm/h for the duration of the event (93 h).

Stability Analysis

A limit-equilibrium slope stability model using Spencer's method was built in SLOPE/W to analyze the stability of the upstream and downstream slopes of the levee. The pore-water pressure results were imported from the SEEP/W model and used in the stability analyses to determine the factor of safety (FOS).

Table 2 lists the parameters for the three layers of the model used in the slope stability analysis. The slope stability parameters were obtained from a prior analysis performed during the design phase of the setback levee. For the saturated foundation silt layer, a friction angle (ϕ') of 25° and a cohesion (c') of 0 kPa were used for the study period. A friction angle of 0 and a cohesion of 21.5 kPa were used to represent the undrained loading during the 100-year extreme flood for the silt layer. Due to their drainage properties, both the levee embankment and the gravel material properties were assumed to be the same under drained and undrained loading. Only the embankment soil was assumed to be saturated-unsaturated for the slope stability analysis. Several slip surfaces for both upstream and downstream slopes were evaluated. The slip surfaces passed through both the unsaturated and saturated zones of the embankment soil, as well as the foundation soils. This indicates the need to determine the shear resistance along both saturated and unsaturated parts of the slip surface to perform the slope stability analysis. The equation proposed by Vanapalli et al. (1996) was used within SLOPE/W to determine the shear strength of unsaturated parts by employing a user-defined SWRC model along with the saturated shear strength parameters (c' and ϕ'). The van Genuchten (1980) SWRC model was used in the slope stability analysis.

Fig. 17 shows the variation of FOS for the entire modeling period for the upstream and downstream slopes. For the upstream slope, FOS varied from 2.2 to 3.0 with no significant change in FOS for the 100-year flood. For the downstream slope, the value of FOS varied during the wet period (winter period) from 2.15 to 1.75, then increased to about 2.5 during the dry period. The 100-year flood event was plotted at the end of the modeling period, and produced a FOS = 1.55 for the downstream slope. The FOS at the end of the 100-year flood for the upstream and downstream slopes is indicated with dots in Fig. 17. The sudden increase in pore-water pressure in a short period during the 100-year flood event occurring after a drought event caused a substantial decrease in FOS. The FOS of the upstream slope increased during the extreme event to a FOS = 3.0.

Fig. 16. Predicted pore-water pressures before and during 100-year flood at different locations.

647 Conclusions

648 This case study demonstrated how to effectively integrate field-
 649 monitoring data and numerical analysis to better evaluate the per-
 650 formance of a levee during normal and extreme hydraulic loading

conditions considering atmospheric coupling. In situ data collected
 from an array of field instruments and a weather station were used
 along with a finite-element model to study the response of a silty
 sand setback levee near Seattle under seasonal and tidal changes.
 The numerical model was calibrated and validated against field

Fig. 17. Factor of safety for upstream and downstream slopes.

data. The calibrated model was then used to simulate the evolution of pore-water pressure and factor of safety during a 100-year flood event.

Including the land-climatic boundary condition (which accounts for the effect of air temperature, precipitation, relative humidity, wind speed, and solar radiation) was essential for prediction of the pore-water pressures. Including only the precipitation as a boundary condition resulted in a close match during wet periods; however, in dry events the decrease in pore-water pressure could be matched only when considering the climatic boundary conditions that account for evaporation of moisture from the embankment soil.

The pore-water pressure obtained from the model had a gradual and subtle change in response to the wet and dry periods, whereas the field instruments had a more abrupt and larger response, i.e., higher pore-water pressures during wet periods and higher suctions during dry periods. This response could possibly depend on where the instruments are installed. For example, the upper layers in the levee are more affected by the climatic variations, whereas the lower layers are more affected by the piezometric line, which the model uses to estimate the pore-water pressure.

In general, the results demonstrated that there is promise for use of saturated-unsaturated transient flow analyses to more accurately predict pore-water pressures representing real field conditions under various loadings. The predictive capability of the numerical model could be improved by incorporating the effects of some additional factors not considered in the current study. Those include, but are not limited to, anisotropy in hydraulic conductivity, looser soil density around sensors, and layers with higher hydraulic conductivity.

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