EFFECT OF LANDSIDE EXCAVATIONS ON 3D LEVEE UNDERSEEPAGE

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Effect of Landslide Excavations on 3D Levee Underseepage

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ABSTRACT: This summary is based on the technical paper, Three-Dimensional Levee Underseepage, authors N.H. Jafari, T.D. Stark, A.L. Leopold, and S.M. Merry, in review in the Canadian Geotechnical Journal.

INTRODUCTION

Two-dimensional (2D) seepage models can under predict landslide hydraulic gradients (Money 2006; Cobos-Roa and Bea 2008; Ahmed and Bazaraa 2009). For example, Money (2006) reports computed three-dimensional (3D) vertical hydraulic gradients that are ~45% and ~120% greater than 2D computed hydraulic gradients for infinite and finite landslide excavations, respectively. Ahmed and Bazaraa (2009) show that neglecting seepage flow through excavation sidewalls can lead to errors in computing the uplift pressures and exit gradients.

The state-of-practice for examining levee and floodwall seepage involves 2D finite element analyses (FEA) and/or analytical solutions proposed in the U.S. Army Corps of Engineers (USACE) design manuals EM 1110-2-1901 and EM 1110-2-1913 (USACE 1993, 2000). The 2D FEA calculates uplift pressures and flow assuming levee geometry, soil stratigraphy, boundary conditions, and excavations are infinitely wide. However, landside excavations are often present and of limited extent near the landside toe. Examples of landside excavations include borrow pits,

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building foundations, agricultural storage silos and tanks, residential swimming pools, burrowing animals, trees, utilities, conduits, pipelines, drainage canals, and culverts. This paper uses the Sherman Island levees to field calibrate SLIDE and RS³ models and then perform a 3D parametric analysis to investigate the effects of landslide excavations on landside hydraulic gradients.

**SHERMAN ISLAND**

The Sacramento-San Joaquin Delta (referred to herein as Delta) is located at the confluence of the Sacramento and San Joaquin Rivers in Northern California. The Delta is important to California’s economy and infrastructure, including a source of water supply for about 25 million Californians, a source of irrigation for over 7 million acres of agricultural land, and an extensive infrastructure of state and local roads, railroads, pipelines, and shipping ports (CALFED 2000). A levee network protects the many islands in the Delta and directs water to San Francisco Bay. Combined with subsiding interiors and high flood levels, both levees and their foundations are vulnerable to seepage and seepage-induced failures. In particular, subsidence is a major concern on Sherman Island because a lower landside elevation increases the total head difference between riverside to landside. From 1930 to the early 1980’s, over 50 Delta islands or tracts flooded due primarily to levee foundation instability (Prokopovitch 1985). Significant consequences occur after a levee breach, such as in 2004 when the Lower and Upper Jones Tract flooded resulting in economic impacts of greater than $100 million (California DWR 2005). Therefore, assessing the probability of seepage-induced levee failures on levee infrastructure is important to public health, commercial activities, and environmental safety of Delta islands.
Sherman Island lies at the western limit of the Delta where the Sacramento and San Joaquin rivers converge and is bordered to the northeast by Three-Mile Slough (see Fig. 1a). The island is located northeast of the city of Antioch, California, and is within the jurisdiction of Sacramento County. Sherman Island is currently protected by approximately 29 km of perimeter levees (Hanson 2009). The levees were originally constructed in the 1860’s over organic soils and have been enlarged periodically as the foundation soils subsided. Approximately 15 km of Sherman Island levees are constructed to federal standards and supervised by the U.S. Army Corps of Engineers (USACE), while the remaining 14 km of levees are non-project levees (maintained by the local levee district). Subsidence and substandard levee protection has resulted in major levee breaches that inundated Sherman Island in 1904, 1906, 1909, and 1969. In 1969, the levee segment on the San Joaquin River between levee Stations 520 and 525 (Fig. 1b) failed and the high velocity flow from the levee breach eroded the island interior and created a scour hole about 6.5 m deep (see scour lake in Fig. 1b). Since 1969, seepage and stability problems have plagued the southern levees (south of Antioch Bridge to Station 545). Numerous piezometers, inclinometers, and settlement plates are being used to monitor levee performance between Stations 520 and 545 (see rectangle in Fig. 1a) to help prevent another breach (Hanson 2009). Borings from previous studies were utilized to develop a subsurface profile at cross-section A-A’ in Fig. 1b and a nearby piezometer was used to calibrate the 2D and 3D seepage models developed herein. These seepage models permitted an investigation of landside excavations on underseepage leading to landside vertical gradients and uplift pressures.
Fig. 1.  Aerial photographs of: (a) Sherman Island and white box shows location of aerial photograph in (b); and (b) close-up showing location of cross-section A-A’ and nearby instrumentation (USDA photos from 2012 [http://datagateway.nrcs.usda.gov/])

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Levee Profile and Soil Properties

The subsurface profile shown in Fig. 2 is developed using Borings B-1, B-2, and B-3 from Hanson (2009). The boring locations shown in Fig. 1b were drilled as part of the levee improvements along landside of Mayberry Slough from approximately Stations 520 to 545. Borings B-1 and B-2 are located in the free field, i.e., past the landside levee toe, while B-3 is located at the landside toe. (The aerial view in Fig. 1b was photographed in 2012 after construction began on a landside stability berm. Borings B-1 and B-2 were drilled at the landside toe, and boring B-3 was drilled through the levee crest; however, they were drilled prior to the 2009 construction.) The levee foundation is comprised of a range of coarse-grained sediments, including gravels and loose clean sands, and silty sands. Thus, the profile in Fig. 2 starts at depth with a fine sand stratum below -15 m NAVD88 (North American Vertical Datum, 1988). Above the sand is a layer of silty clay, locally known as Bay Mud, deposited as the sea level rose following the last ice age. The clay stratum is about 3.1 m thick and overlain by organic soils that extend to the ground surface. Shelmon and Begg (1975) report sea level rise in the past 7,000 years created tule marshes that covered most of the Delta. The repeated burial of the tules and other vegetation growing in the marshes formed approximately 8 m of highly organic soils at cross-section A-A’. The highly organic soils are not classified as peat because the organic contents (ASTM D2974) and classification defined in ASTM D4427 (2013) are not available to confirm sufficient organics for peat classification.

The Sherman Island levee embankment is comprised of dredged loose to medium sand and silt. Weight of the levee embankment caused settlement of the organic soil layer and hence a decrease in horizontal hydraulic conductivity. In addition, the levee appears to be located directly over natural levees of the San Joaquin River, which are represented by a layer of silty clay between...
the organic soil and levee fill. This natural levee, known locally as overbank deposits, is found to be of limited lateral extent, grading into the thick organic soil stratum beneath the levee berms.

Water levels are maintained 0.6 to 1.5 m below land surface by an extensive network of drainage ditches. Foot et al. (1992) report an artesian condition in the sand substratum causing upward seepage through the clays and organic soils, where seepage is typically drained off, collected, and pumped out of the island via a series of levee toe drainage ditches flowing to a pumping station (see toe ditch in Fig. 2).

![Fig. 2. Levee cross-section A-A’ of Sherman Island at Station 532](image)

Table 1 summarizes index properties and engineering parameters used in the seepage analyses. Due to the weight of levee fill, the natural water content ($w_o$) of the organic soils under the levee embankment ranges from 115 to 265% while the organic soils not under the levee (landside organic soils) have natural water contents from 225% to 410% (Weber 1969; Foot et al. 1992). The resulting organic soil saturated unit weight (11.6 kN/m³) is greater than the landside organic soils (10.5 kN/m³) and is in agreement with unit weights reported in Mesri and Aljouni (2007). Available hydraulic conductivity tests are limited for the levee embankment, silty clay, and sand present in Fig. 2. As a result, estimates of horizontal hydraulic conductivity ($k_h$) for the
levee fill, silty clay, and sand were made using the Delta Risk Management Strategy (DRMS) levee vulnerability technical memorandum (URS 2013). The $k_h$ of the organic soils was evaluated using Weber (1969); Mesri and Aljouni (2007); and Mesri et al. (1997) and the appropriate average effective vertical stress ($\sigma'_v$). Weber (1969) utilized piezometer data to estimate the horizontal hydraulic conductivity of organic soils in the Delta using an inverse analysis and found the horizontal hydraulic conductivity ranges from $1 \times 10^{-7}$ to $1 \times 10^{-4}$ cm/s for $\sigma'_v \leq 550$ kPa. The $\sigma'_v$ of organic soils under the levee and landside is about 190 kPa and 45 kPa, respectively, based on the cross-section in Fig. 2. This $\sigma'_v$ correlates to $k_h$ of about $3 \times 10^{-5}$ cm/s and $3 \times 10^{-4}$ cm/s for organic soils under the levee and landside, respectively. The anisotropy ratio, i.e., ratio of horizontal to vertical hydraulic conductivity ($k_h/k_v$), for buried Middleton peat deposit is estimated to be 10 and 3 to 5 for surficial peats (Mesri and Aljouni. 2007). Thus, the values of $k_h/k_v$ chosen for Sherman Island organic soils are 10 and 3 for organic soils under the levee and landside, respectively. The anisotropy ratio was assumed four (4) for levee fill and ten (10) for sand and natural silty clays (URS 2013). Because this study is focused on underseepage during steady-state conditions, unsaturated soil properties are not modeled for the levee embankment fill and landside organic clay.

<table>
<thead>
<tr>
<th>Soil Type and Classification</th>
<th>$\gamma_{sat}$ (kN/m$^3$)</th>
<th>$w_o$ (%)</th>
<th>$k_h$ (cm/s)</th>
<th>$k_h/k_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Levee Fill</td>
<td>17.7</td>
<td>8-13</td>
<td>$1 \times 10^3$</td>
<td>4</td>
</tr>
<tr>
<td>Organic Soil Under Levee</td>
<td>11.6</td>
<td>116-265</td>
<td>$3 \times 10^5$</td>
<td>10</td>
</tr>
<tr>
<td>Landside Organic Soil</td>
<td>10.5</td>
<td>224-408</td>
<td>$3 \times 10^4$</td>
<td>3</td>
</tr>
<tr>
<td>Silty Clay</td>
<td>16.7</td>
<td>49-78</td>
<td>$1 \times 10^6$</td>
<td>10</td>
</tr>
<tr>
<td>Sand</td>
<td>19.5</td>
<td>25-35</td>
<td>$1 \times 10^2$</td>
<td>10</td>
</tr>
</tbody>
</table>
Calibration of 2D Seepage Model

The 2D SLIDE finite element program (Rocscience 2014) is used to determine the phreatic surface through the levee fill and calibrate pore-water pressures in the substratum using piezometer data. SLIDE is a general seepage analysis program formulated to model saturated and unsaturated transient flow through soil. Piezometer PZ-533 (Fig. 1b) was installed in 2008 at Station 533 to monitor pore-water pressures during landside stability improvements (Hanson 2009). The initial pore-water pressures prior to construction of the stability berm were used to calibrate the SLIDE model.

The initial riverside steady-state boundary condition is a total head ($h_t$) of +1 m NAVD88 and is consistent with canal water levels measured at the Antioch gage station. The “seepage exit face” option was selected for the ground surface from the levee centerline to landside levee toe (40 m from levee centerline in Fig. 2) in SLIDE because the phreatic surface on the landside levee slope is unknown. The boundary condition from landside levee toe to the right-hand side (RHS) of the finite element mesh is assumed to be 0.75 m below the ground surface (-4.25 m NAVD88). The left-hand side (LHS) vertical boundary is specified as the steady-state river stage (+1 m NAVD88) because the sand stratum is hydraulically connected to the San Joaquin River. The bottom boundary condition is defined as an impervious boundary to represent the low hydraulic conductivity clay underlying the sand stratum. The boundary conditions and soil profile are illustrated in Fig. 2.

The cross-section in Fig. 2 was calibrated using data from PZ-533, which is located 43 m landside from the levee centerline and at ground surface elevation of -3.5 m NAVD88. Three piezometers shown in Fig. 2 are installed at elevations of -9.6 m (PZ-533-1), -14.2 m (PZ-533-2), and -18.8 m (PZ-533-3) in PZ-533 (Hanson 2009). The total heads in Table 2 were recorded prior
to construction of the improvement activities. The total heads of -3 m and -2.7 m measured by PZ-533-2 and PZ-533-3, respectively, indicate artesian conditions exist in the sand layer and confirm the hydraulic connection between the San Joaquin River and sand stratum reported in Foott et al. (1992). The comparison of PZ-533 response and SLIDE at steady-state conditions is shown in Table 2. The SLIDE model slightly over estimates pore-water pressures but is in close agreement with measured values. The phreatic surface computed in SLIDE is shown in Fig. 2. Based on the initial pore-water conditions and phreatic surface, the 2D calibrated model was used to perform 3D simulations of landside excavations and levee bends.

**Table 2.** Calibration of Sherman Island seepage model at steady-state conditions

<table>
<thead>
<tr>
<th>Piezometer</th>
<th>Depth (m)</th>
<th>Soil Layer</th>
<th>Measured $h_t$ (m)</th>
<th>SLIDE $h_t$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-9.6</td>
<td>Organic Soil</td>
<td>-4.0</td>
<td>-4.2</td>
</tr>
<tr>
<td>2</td>
<td>-14.2</td>
<td>Silty Clay</td>
<td>-3.0</td>
<td>-3.1</td>
</tr>
<tr>
<td>3</td>
<td>-18.8</td>
<td>Sand</td>
<td>-2.7</td>
<td>-2.8</td>
</tr>
</tbody>
</table>

**EFFECT OF LANDSIDE EXCAVATIONS**

The effect of floodside and landside excavations came to the forefront in the final litigation chapter of the IHNC (Inner Harbor Navigation Canal) (Stark and Jafari 2014) and was the impetus for this 3D study. The USACE contracted for the demolition and environmental clean-up of the East Bank Industrial Area (EBIA) to allow expansion of the IHNC. The EBIA had a long history of industrial use and contamination (WGI 2005), as well as the presence of buried structures and foundations that could interfere with construction of a bypass channel in the expanded IHNC. Thus, the removal of contaminated soil and buried structures resulted in many floodside excavations near

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the IHNC floodwalls (see Fig. 3). If these excavations were backfilled with high hydraulic conductivity soils and penetrated to depths below the floodwall sheet pile tip, floodside underseepage could have been facilitated as illustrated in Fig. 3 (see blue arrows in Fig. 3).

![Schematic of hypothetical underseepage induced by IHNC floodside excavations](after Stark and Jafari 2014)

**Fig. 3.** Schematic of hypothetical underseepage induced by IHNC floodside excavations (after Stark and Jafari 2014)

Stark (2012) performed 3D seepage analyses to understand the development of the IHNC floodwall breaches and found the maximum 3D landside vertical gradients correspond to the 2D cross-section that cuts through the floodside excavation. In contrast, the 3D seepage analyses showed that landside excavations, e.g., the Jourdan Avenue Box Culvert in Fig. 3, cause 3D gradients to increase because of inward seepage from the sidewalls. Therefore, landside excavations are investigated herein to illustrate 3D flow and provide recommendations on when a 3D analysis is warranted.
**Formation of 3D Model**

The 3D Sherman Island seepage model in Fig. 4 is developed in the software package RS³ (Rocscience 2014) to compare the effect of finite landside excavations to the calibrated 2D SLIDE seepage model in Fig. 2. The soil profile, hydraulic properties, and levee geometry are identical between the two models (SLIDE and RS³). The 3D model geometry is constructed by extruding “into the page” the 2D profile in Fig. 2. The 3D model uses three extruded slices to model the excavation, with the center slice modified to include an open excavation. The outer two slices duplicate Fig. 2 and are 50 m wide to minimize end effects (see Fig. 4). The excavation width is varied by widening the center slice. The landside toe ditch does not affect vertical gradients in the 2D model, so it is removed for these parametric analyses. The boundary conditions shown in Fig. 4 include a seepage exit face applied to the inside surfaces of the excavation. The excavation is modeled as a seepage face because water can seep into the excavation during flood conditions. This boundary condition is applied in open excavations, e.g., during construction of buildings, swimming pools, and underground utilities. The riverside and LHS vertical boundary condition are assumed a maximum river stage of +1.8 m NAVD88. The boundary conditions from the landside levee toe to the RHS of the finite element mesh is zero pressure head (\(h_p\)). The RHS vertical boundary is modeled as a total head boundary (\(h_t = -3.5\ m\) or at the ground surface) to represent the landside groundwater conditions. The boundary condition along the bottom of the seepage model remains as a no flow boundary. Figure 5 shows the 2D flow field and total head contours at a river stage of +1.8 m. The total head contours decrease from +1.8 m at the riverside to -6 m at the bottom of excavation. The contours in the sand stratum are horizontal, indicating minimal head loss in this stratum. The total head contour of -6 m at the bottom of excavation results in hydraulic gradients close to 0.4.
Fig. 4.  RS$^3$ model showing 3D soil stratigraphy, boundary conditions, and landside excavation

Fig. 5.  2D flow field and total head contours through landside excavation

**Landside Excavation Parametric Results**

The multiple excavation widths used to define the vertical hydraulic gradients at the center and edge of a landside excavation are shown in Fig. 6. Points 1 and 2 in Fig. 6 are situated along the centerline of the landside excavation and Points 3 and 4 are located at the excavation edge.
At widths of 2 m, high gradients observed in the center of the excavation are attributed to inward seepage from the sidewalls. For excavations less than 3 m, the gradients measured at Points 1 and 3 as well as 2 and 4 are equal because of the short distance between the edge and center of the excavation. The lower gradients at Points 2 and 4 compared to 1 and 3, respectively, are attributed to increased seepage length to the middle of the excavation. As the excavation width increases, gradients converge to 2D results (performed through the excavation) and the effect of inward seepage reduces on Points 1 and 2.

For this analysis, Points 1 and 2 approach 2D equivalent gradients when the excavation width is at least 15 m. This width corresponds to an excavation aspect ratio (length to width; L:W) of 1L:1.5W and is considered the threshold when vertical gradients in the excavation center are not influenced by 3D seepage. Vertical gradients are always higher at Points 3 and 4 compared to the excavation center because of converging flow. For example, Points 3 and 4 approach a constant hydraulic gradient of 0.66 and 0.44, respectively, at 10 m width. For a width of 20 m, the vertical gradients at Points 3 and 4 are 150% and 190% greater than Points 1 and 2, respectively. As a result, seepage-induced failures (heave or sands boils) can develop near the excavation sidewalls, as well as in the center for narrow excavations, having an aspect ratio of 1L:1.5W.
Fig. 6. Changes in vertical hydraulic gradient with increasing landside excavation width are compared to 2D seepage analyses through the excavation.

Figure 7 shows total head contours at +1.8 steady-state flood condition with increasing excavation widths. The total head contours in Fig. 7a are about -6.5 m at the excavation floor and -3.5 m on the landside surface. The concentrated total head contours for the 5 m excavation in Fig. 7b indicate greater change in total head and higher vertical exit gradients within the excavation. As the excavation width is increased in Figs. 7b – 7e, the total head contours in front of the excavation straighten and the contours are spaced farther apart at the excavation sidewalls, thus arriving at 2D conditions. Total head contours in Fig. 7 corroborate the results in Fig. 6 by showing 3D seepage effects are limited for excavation widths greater than 15 m and the excavation sidewall is the critical zone to evaluate vertical hydraulic gradients and uplift pressures.
Fig. 7. 3D total head contours for (a) model and landside excavation widths of (b) 5 m, (c) 10 m, (d) 20 m, and (e) 30 m

The analyses in Figs. 6 and Fig. 7 correspond to open excavations, which may be present for a temporary period. Closed excavations consist of an impermeable structure or lining (buried culverts, building foundations, and swimming pools) and can be prone to heave or sand boils around the excavation if excessive uplift pressures develop. Increases in uplift pressure can be facilitated by the low high conductivity structure not allowing migration of the hydraulic pressures and acting as a dam. Thus, it is necessary to compare the average uplift pressure in the excavation and the applied overburden stress. The uplift pressures are computed by applying a no flow...
boundary condition in the excavation bottom and sidewalls. This boundary condition yields an average 3D uplift pressure of 31 kPa. By estimating an overburden pressure due to the building load, the factor of safety against uplift for the building foundation can be computed.

SUMMARY

The state-of-practice to examine levee and floodwall seepage performance includes 2D plane strain FEA and analytical equations proposed in the USACE design manuals EM 1110-2-1901 and EM 1110-2-1913 (USACE 1993, 2000). This study shows that 2D analyses under predict hydraulic gradients for landside excavations, e.g., drainage ditches, building foundations, residential swimming pools, utilities, and culverts. The Sherman Island levee system is used to develop a calibrated seepage model and evaluate the effect of finite landside excavations.

The parametric analyses indicate that a 3D analysis should be used for a finite landside excavation with an aspect ratio less than 1L:1.5W. Because this aspect ratio is site specific and can depend on material hydraulic conductivity, soil profile, and excavation geometry and location from landside toe, this example is used to illustrate the importance of 3D flow in landside excavations. Narrow excavations, e.g., trenches, pipelines, conduits, animal burrows, and tree trunks, are likely to impact floodwall or levee performance because vertical gradients rapidly increase below widths of 5 m. For cases such as drainage canals and toe ditches that run parallel to the levee and are greater than 20 m wide, the gradients at the center of the excavation are approximately equal to 2D vertical gradients so a 2D analysis can be performed. Money (2006) reports that 3D vertical gradients are about 145% greater for infinitely long landside excavations, e.g., toe drainage ditches, than 2D models. These results are in disagreement with Fig. 6 that illustrates long

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excavations, i.e., widths > 20 m, approach 2D vertical gradients. The reason for this over prediction is not clear but the authors recreated the 2D model and found that increasing the mesh density from 1.5 m to 0.25 m increased 2D hydraulic gradients closer to the 3D values.

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REFERENCES


Calfed (Calfed) Bay Delta Program. 2000. Levee system integrity program plan, Final Programmatic EIS/EIR Technical Appendix.


Hanson, J.C. (Hanson) Consulting Civil Engineer. 2009. Reclamation district 341 Sherman Island five-year plan, SH 08-3.0. California DWR, Sacramento, CA.


URS. 2013. Delta risk management strategy (DRMS), Phase 1 Final Risk Analysis Report, Sacramento, CA.


USACE. 2000. Engineering and design—design and construction of levees. EM 1110-2-1913, Department of the Army, Washington, D.C.
