Numerical analysis of consolidation behavior of soil-bentonite backfill in a full-scale slurry trench cutoff wall test

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Abstract

The stress state of the SB backfill in a full-scale cutoff wall test during the construction and consolidation was simulated by a finite-element model. The applicability of the model was demonstrated by good agreements between modeled and field monitored earth pressures and pore pressure in the backfill. Based on the analyzed results, the backfill consolidation process can be described as a four-staged cycle of load transfer: (1) the effective stress of the backfill increases due to the dissipation of excess pore pressure caused by self-weight consolidation of the backfill; (2) the increased effective stress results in settlement of the backfill and an increase of sidewall friction between the backfill and the sidewall interface; (3) the increased sidewall friction results in the transfer of backfill weight to the sidewalls and a decrease of the consolidation stress on the backfill; and (4) the decreased consolidation stress attenuates the decrease of excess pore pressure, influencing the subsequent consolidation. The cycle from (1) to (4) continues until the consolidation is completed.

Keywords: Consolidation; Cutoff wall; In situ testing; Finite-element analysis; Soil-bentonite

1. Introduction

Soil-bentonite (SB) slurry trench cutoff walls are commonly used as vertical containment barriers for controlling groundwater flow and contaminant transport at landfills and contaminated sites, mainly due to the low permeability of SB backfill \( k_{sb} \leq 1 \times 10^{-9} \text{ m/s} \) (D’Appolonia, 1980; Yeo et al., 2005). However, the \( k_{sb} \) is highly dependent on the stress state of the SB backfill, which, in the field, can be impacted by consolidation and load transferred from the shear along the sidewalls of the trench. Many studies have demonstrated the considerable decrease in \( k_{sb} \) with increased consolidation pressure (Evans, 1994; Filz et al., 2001; Yeo et al., 2005; Ruffing and Evans, 2010; Evans and Huang, 2016). For example, the \( k_{sb} \) decreased by 1–2 orders of magnitude when the effective consolidation stress increased by 100 kPa (Evans, 1994; Yeo et al., 2005). Thus, an accurate prediction of the stress state of the backfill during the consolidation process is essential to a proper evaluation of long-term barrier performance of SB backfill cutoff walls.

The in situ stress state of a SB backfill can be lower than the geostatic stress estimate based on the effective weight of the overlying backfill (Evans et al., 1985; Khoury et al., 1992; Evans et al., 1995; Bennert et al., 2005). Thus, using the geostatic stress in the consolidation/permeation tests can result in an inaccurately low \( k_{sb} \) and an unconservative design. To obtain accurate predictions of long-term effective stresses of SB backfill, three theoretical models have

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been established during the past three decades, i.e., the arching model (Evans et al., 1995), the lateral squeezing model (Filz, 1996), and the modified lateral squeezing model (Ruffing et al., 2010), all of which successfully explained the mechanism behind the phenomenon of lower in situ effective stress relative to the geostatic stresses of SB cutoff walls. However, the calculated distributions of stress along the depth of the wall in these models still differed significantly from that of field measurements. Li et al. (2015) and Ke et al. (2018) proposed analytical models with consideration of both arching and lateral squeezing mechanisms and successfully calculated the distribution of the major principal stress in a SB cutoff wall at the Mayfield site in Australia.

The SB backfill is a semi-fluid material, typically with a slump of 100–150 mm at gravimetric water content of 20–40% (e.g., Malusis et al., 2008). After placement in the trench, the SB backfill consolidates under the combined vertical loading from the weight of the overlying backfill and the horizontal squeezing stresses from adjacent soils (Baxter, 2000; Ruffing and Evans, 2010; Ruffing et al., 2011). While reasonable correlations of the estimated shear strength among the vane, the CPT, and the dilatometer have been developed (Evans and Ryan, 2005; Evans et al., 2019; Ruffing et al., 2012), the estimations by these methods are limited compared to full-scale field measurements. As a result, several full-scale field tests have been established for assessing the holistic field behavior of SB cutoff walls in terms of stresses, deformations, and hydraulic conductivity (Baxter, 2000; Ruffing et al., 2010; Evans et al., 2017; Tong et al., 2020). Based on in situ monitoring data, Baxter (2000) developed an all-stage finite-element model for the consolidation process of a SB cutoff wall. However, the analysis was mainly focused on lateral deformations and settlements of the adjacent soils rather than the change of earth pressures and excess pore pressure of SB backfill. To date, the understanding of consolidation mechanisms (both primary and secondary) of SB backfills in slurry trench cutoff walls remains limited.

In this paper, a finite-element model with considerations of both vertical friction and lateral squeezing of the sidewalls is proposed to explain the consolidation behavior of a SB backfill slurry trench cutoff wall. The proposed model was applied to a full-scale SB backfill cutoff wall test. Applicability of the model was assessed by comparing the simulated earth pressures and pore water pressure with field measurements. Then, the model was applied to assess the changes of stress state of the cutoff wall in terms of the distribution of sidewall friction and consolidation stress, the dissipation of excess pore pressure, and the degree of consolidation of the wall.

2. The full-scale sand-bentonite cutoff wall test modeled

In July 2016, a full-scale experimental SB backfill slurry trench cutoff wall was constructed in a buffer zone between a mining area and a natural wetland near the Bucknell University campus (Lewisburg, Pennsylvania, USA). The purpose of this project was to assess field behavior of a SB cutoff wall in terms of (1) the in situ stresses and deformations in the wall during construction and over time, (2) the in situ properties in terms of hydraulic conductivity, water content and shear stress of the backfill and their variabilities with location and time, and (3) to investigate the feasibility of electrical resistivity (ER) imaging as an effective geophysical method for detecting variations in homogeneity, including defects, within the wall (Evans and Ruffing, 2017; Malusis et al., 2017). According to field investigations (Malusis et al., 2017), the subsurface soil comprised of silty sand and gravel, underlying by sand and clay layers with varying thickness and hard materials. The groundwater table was approximately 2.5 m below the ground surface at the time of the investigations.

The construction lasted for 11 days from July 11th to 21st, 2016, resulting in a wall with a length of 194 m, a width of 0.9 m, and an average depth of 7 m (as shown in Fig. 1). The bentonite slurry used for construction contained 5–6% (by weight of water) of PremiumGel® sodium bentonite (CETCO, Hoffman Estates, IL). The slurry was then replaced by SB backfill comprised of bentonite slurry and an imported base soil from another area of the mining site. The SB backfill had fines content of 44–57% and clay content of 13–20%, exhibited a liquid limit and plasticity index of 16–18% and 6–7%, respectively, and were classified as SC-SM or CL-ML in accordance with the Unified Soil Classification System [ASTM D2487 (ASTM, 2017)].

During construction, four pairs of inclinometers were installed at Stations 0 + 30, 0 + 60, 0 + 90, and 1 + 20 m near the wall for evaluating the lateral displacement of the adjacent soils. Four instrumentation cluster cages (see Fig. 2) were installed at 2, 4, and 6 m below ground surface at Station 0 + 87 m and at 5.8 m below ground surface at Station 0 + 75 m. Three vibrating-wire stress sensors and a vibrating-wire piezometer were equipped on each cage to monitor in situ total stresses in transverse (perpendicular to the wall alignment), longitudinal (parallel to the wall alignment), and vertical directions, vertical and lateral deformations, and pore water pressure of the backfill (Evans and Ruffing, 2017). More details of the research project have been reported by Evans et al. (2017), Evans and Ruffing (2017), Malusis et al. (2017), Barlow (2018), Barlow and Malusis (2019), Evans and Ruffing (2019), Evans et al. (2019), Malusis and Barlow (2020), Evans et al. (2021), Jacob et al. (2021), and Sample-Lord et al. (2021). Although more recent in situ data have been reported by the aforementioned publications, the parameters associated with field measurements during the first 150 days, in which the most significant changes of stresses and deformations occurred, were used to establish and validate the numerical model in this paper. As a result, the numerical model described herein accounts for primary consolidation but does not explicitly consider the longer-term secondary (creep) compression following primary consolidation.
3. Numerical model

3.1. Geometry

A model using geotechnical finite-element analysis software PLAXIS 2D (Plaxis B.V., 2017) was employed for the stress state analysis of the SB backfill. The geometry of the model is shown in Fig. 3. The cross-section at the Station 0 + 90 m, where nearly all types of sensors were embedded closely, was selected as the modeling plane. The dimensions of the model were 30 m long and 15 m deep. Since the wall was symmetric with respect to the centerline, only the right half of the cross-section was analyzed in the model.

The foundation soil was comprised of a 4 m thick silty sand layer, a 1 m thick soft silty sand layer, and a 10 m thick clay layer. The groundwater table ($h_w$), set at 2.5 m beneath the surface, was assumed to be unchanged during the consolidation. The initial earth pressures in soil layers before trench excavation were calculated using the unit weight of the layers with the coefficient of lateral earth pressure at rest ($K_0$).

The modeled SB backfill was 7 m deep ($H$) and 0.45 m wide ($B$), corresponding to the 7-m average depth and 0.9-m width of the research wall. To simulate the filter cake formed by the bentonite slurry during trench excavation, a thin layer was added between the backfill and the founda-
tion soil (right to the backfill). The thickness of the filter cake, \( B_{fc} \), was set to be 5.0 mm, in accordance with the range of 0.5–13 mm in a pilot-scale SB cutoff wall test (Britton, 2001) and the value of 6.35 mm used in a finite-element model (Baxter, 2000). Vertical deformation between the filter cake and backfill was achieved by adding an interface element. The sidewall friction on backfill was simulated by specifying the shear strength properties to the SB-filter cake interface (i.e., the shear plane).

As illustrated in Fig. 3, the left boundary \((x = 0)\) was assumed to have zero horizontal displacement \((d_x = 0)\) and zero water discharge \((q = 0)\). The right boundary \((x = 30 \text{ m})\) was assumed to have zero horizontal displacement and zero excess pore pressure \((u_e = 0)\). The bottom boundary \((z = 15 \text{ m})\) was assumed to have zero horizontal and vertical displacements \((d_x = d_z = 0)\) and zero water discharge \((q = 0)\).

### 3.2. Material parameters

While undrained shear strength has been measured in the field using vane shear, cone penetrometer and dilatometer tests (Evans et al., 2019; Evans et al., 2021), effective stress strength properties for the SB backfill have not been reported as of the time of these analyses. Based on the grain-size distribution and bentonite content (\( \approx 1.4\% \)) of the backfill, the Modified Cam-Clay model and parameters of the Cam-Clay compression index \((c)\), the Cam-Clay swelling index \((j)\), the Poisson’s ratio \((\mu')\), and the tangent of critical state line \((M)\) obtained from consolidation and triaxial tests for a similar backfill SB 1 from Baxter (2000) were adopted in this paper (see Table 1).

The relationship between hydraulic conductivity \((k_{sb})\) and void ratio \((e)\) of the SB backfill under the effective consolidation stress \(\sigma'\) was considered by the following expression (Plaxis B.V., 2017):

\[
\log \left( \frac{k_{sb}}{k_{sb,0}} \right) = e - e_0 \frac{c_k}{c_k}
\]

where \(k_{sb,0}\) and \(e_0\) are the hydraulic conductivity and void ratio of the backfill under the initial effective stress \(p_0\), and \(c_k\) is a dimensionless parameter. Based on one dimensional nonlinear consolidation theories (e.g., Raymond, 1969), the relationship between \(e\) and \(\sigma'\) can be expressed as:

\[
e = e_0 - \lambda \ln \frac{\sigma'}{p_0}
\]

Then, the relationship between \(k_{sb}\) and \(\sigma'\) can be obtained by substituting Eq. (2) in Eq. (1), that is,

\[
\log \left( \frac{k_{sb,2}}{k_{sb,1}} \right) = -\frac{\lambda}{c_k} \ln \left( \frac{\sigma'}{\sigma_{sb,1}} \right)
\]

where \(k_{sb,1}\) and \(k_{sb,2}\) are the hydraulic conductivities of the SB backfill under \(\sigma_{sb,1}\) and \(\sigma_{sb,2}\). Based on Eq. (2), the \(\lambda = 0.07\) for the undisturbed samples near Station 0 + 90 m was

### Table 1

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Value</th>
<th>Parameter</th>
<th>Unit</th>
<th>Value</th>
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<td>(\epsilon_{inter}^\prime)</td>
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<td>(c_k)</td>
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<td>0.12</td>
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<td>(M)</td>
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<td>(k_{sb,0})</td>
<td>m/s</td>
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<tr>
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<td>10.8</td>
<td>(k_{ic})</td>
<td>m/s</td>
<td>1.7 \times 10^{-11}</td>
</tr>
</tbody>
</table>

Note: \(\gamma\) = unit weight; \(sb\) = soil-bentonite backfill; unsat = unsaturated; sat = saturated; \(\sigma\) = bentonite slurry; \(\epsilon\) = effective cohesion; \(\psi\) = effective internal friction angle; \(\psi'\) = effective dilation angle; inter = interface between the backfill and the filter cake; \(e_0\) = initial void ratio of SB backfill; \(c_k\) = dimensionless parameter in the relationship between \(k\) and \(e\); \(k_{sb,0}\) = initial hydraulic conductivity of SB backfill; \(k_{ic}\) = hydraulic conductivity of filter cake.
used in this paper, same as that adopted by Baxter (2000). The $k_{\text{sb}}$ of in situ collected backfill samples were $1.4 \times 10^{-9}$ and $7.0 \times 10^{-10}$ m/s under $p'$ of 20.7 and 34.5 kPa, respectively (Malusis et al., 2017). Thus, $c_2$ was calculated to be 0.12 based on Eq. (3), slightly lower but adequately close to the values of 0.16 and 0.17 measured by Sivapullaiah et al. (2000) and Xu et al. (2016), respectively.

The $p'_0$ of the backfill was set to be 4 kPa in the model, based on the average value of in situ monitored transverse, longitudinal and vertical effective stresses at depths of 2, 4 and 6 m at the beginning of the consolidation (i.e., $t = 5.02$ d). Based on Eq. (3), the estimated $k_{\text{sb},0}$ at $p'_0 = 4$ kPa was $1.3 \times 10^{-8}$ m/s. The calculation was validated by an estimated $k_0$ of $3.2 \times 10^{-9}$ m/s at $p' = 11$ kPa that were consistent with the measured value from the in situ slug test (Barlow, 2018). The $e_0$ of the backfill was assumed to be 0.8 at $p'_0 = 4$ kPa, slightly larger than the maximum $e$ of 0.75 of the undisturbed specimens at $p' = 6.9$ kPa (the lowest applied stress) in isotropic consolidation (Barlow, 2018).

During the construction, the measured unit weights of the in-trench bentonite slurry ($\gamma_{\text{al}}$) and SB backfill ($\gamma_{\text{sb},\text{sat}}$) were 10.8 and 18.3 kN/m$^3$, respectively (Evans et al., 2017). The hydraulic conductivity of the filter cake ($k_{\text{fc}}$) was set to be $1.7 \times 10^{-11}$ m/s, consistent with the range reported by Chung and Daniel (2008). Thus, the shear strength of the interface between the backfill and the filter cake, $\tau_{\text{inter}}$, was assumed to follow the Mohr-Coulomb criterion, that is,

$$\tau_{\text{inter}} = c'_{\text{inter}} + \sigma'_x \tan \phi'_{\text{inter}}$$

where $c'_{\text{inter}}$ and $\phi'_{\text{inter}}$ are the effective cohesion and internal friction angle of the interface; and $\sigma'_x$ is the transverse effective stress at the interface. $\phi'_{\text{inter}} = 21–38^\circ$ were reported for filter cakes by Scott (1978) and Day et al. (1981). Values of $c'_{\text{inter}}$ and $\phi'_{\text{inter}}$ selected for this model were identical to those adopted in an arching model (Evans et al., 2017) that yielded a good agreement with field measured $p'$ of the backfill.

The in situ properties of foundation soils remained mostly unknown except for the results from standard penetration tests ($N_{\text{SPT}}$) (Malusis et al., 2017). At the closest boring location to Station 0 + 90 m (i.e., Station 1 + 00 m), $N_{\text{SPT}}$ for silty sand layer, soft silty sand layer, and the topsoil in clay layer were 6–20, 4 and 6, respectively. Isotropic linear elastic/Mohr-Coulomb plastic model was applied to the soil layers in the model. The relationships between the Young’s modulus ($E$) and $N_{\text{SPT}}$ of sand have been proposed by many researchers. For example, Khoiri and Ou (2013) proposed an relationship between $E$ and depth of Kaohsiung sand (mostly silty sand) based on inverse analysis, as follows:

$$E = (10.2 - 15.4) \cdot z^{0.5}$$

where $z$ is the depth (m). Based on Eq. (5), the $E$ of foundation soil at $z = 0$–4 m ranged from 0 to 30.8 MPa (II), essentially consistent with the range of $E = 4$–62 MPa (I) of silty sand layer at $N_{\text{SPT}} = 6$–20 (Khoiri and Ou, 2013). Similarly, Ng and Yan (1998) and Gourvenec and Powrie (1999) proposed linear relationships between $E$ and the depth for clay in diaphragm walls under construction. Since the soil stiffness is influenced by confining pressure (or overburden pressure), a reasonable assumption of increased $E$ with depth has been adopted in the model. In summary, the stiffness parameters applied to the three soil layers were: (1) $E = 10 + 4z$ MPa for the silty sand layer, i.e., $E = 10$–26 MPa at $z = 0$–4 m (essentially consistent with range II); (2) $E = 6$ MPa for the soft silty sand layer, since the layer was relatively thin (1 m thick) and the $N_{\text{SPT}}$ of the layer was slightly less than the minimum $N_{\text{SPT}}$ of silty sand layer; and (3) $E = 10 + 40z$ MPa for the clay layer, since the $N_{\text{SPT}}$ of the topsoil in clay layer was identical to the minimum $N_{\text{SPT}}$ of silty sand layer and the clay soil was described as “very stiff”. In addition, field investigations indicated that there could be shaley limestone laying below 10-m depth and this material was encountered at least one location during trench excavation (Malusis et al., 2017). Thus, $E$ of the topsoil in the clay layer was assumed to be 10 MPa, then increased by 40 MPa per meter depth (Ng and Yan, 1998).

Values of unit weight ($\gamma$), Poisson’s ratio ($\nu$), cohesion ($c'$), and internal friction angle ($\phi'$) of typical types of sand and clay were adopted for the three soil layers, as summarized in Table 2. Lower values of $c'$ and $\phi'$ were adopted for the soft silty sand layer, in accordance with the low shear strength of the layer. The dilation angle ($\psi'$) was assumed to be $\psi' \approx \phi' - 30^\circ$ for sand (Bolton, 1986) and $\psi' = 0$ for clay.

Field measurements indicated that, within the backfill the direction of total displacement after backfilling was inward, and, as expected, the adjacent soils moved outward from the trench upon backfill placement. Thus, the foundation soil was assumed to be over-consolidated before the construction. Gourvenec and Powrie (1999) assumed $K_0$ of Liasclay to be 2.5–1.1 at depth of 0–40 m in a diaphragm wall. Note that diaphragm walls are constructed in panels giving rise to three dimensional effects that do not present in SB slurry wall construction. Also, an excessively large value of $K_0$ might result in excessively overstimated displacement of adjacent soils after excavation. Thus, $K_0$ of the sand layers and the clay layer were assumed to be 1.1 and 0.8, respectively. Hydraulic conductivity ($k_c$) was assumed to be isotropic in each layer since flow is predominantly horizontal. The material parameters of foundation soil layers are summarized in Table 2.

### 3.3. Simulating procedures

The state of stress of the SB backfill during construction and consolidation were simulated by the following procedures:
(1) Trench excavation with bentonite slurry support

The excavation of the trench was simulated by applying hydrostatic pressure of bentonite slurry on the sidewall and bottom of the trench. The hydrostatic pressure was calculated based on $\gamma_{\text{sl}}$ with consideration of height difference between the slurry surface and the ground surface ($h_{\text{sl}} = 0.28$ m, Evans et al., 2017), as shown in Fig. 4(a). For the reported 5-day duration between the excavation and backfilling, consideration of lateral displacements of adjacent soils into the trench with limit of that needed to mobilize active earth pressures during this period was included in the model.

(2) Trench backfilling

Malusis et al. (2017) and Tong et al. (2020) both reported that the distribution of initial stress state in the SB backfill cutoff walls were nearly isotropic, and that the initial total stresses and initial pore pressure after backfilling were close to the geostatic stress caused by self-weight of the backfill. Thus, the initial stress state of the wall was established via two steps:

(i) The $\sigma'_{\text{inter}} = 0$ was applied to the interface between the wall and the foundation soil to avoid reduction of the self-weight loading on backfill caused by sidewall friction. In addition, pressures applied to the sidewall and bottom of the trench were updated to simulate the forces induced by the semi-fluid backfill. The saturated unit weight of the backfill was used to calculate the updated pressure, as shown in Fig. 4(b). Then, the backfilling process was simulated by applying 10% of the self-weight pressure of the backfill to the excavated area.

(ii) The pressure in the excavated area from the backfill (see Fig. 4(b)) was replaced by 100% of self-weight pressure of the backfill to be consistent with the construction process. This step simulates excess pore pressure in the backfill induced by self-weight pressure.

![Fig. 4. Pressures applied on the sidewalls and bottom of the trench during excavation and backfilling processes.](image-url)
After the two steps, the estimated initial effective stresses of the backfill ranged from 0 to 5 kPa, similar to the average field measurement of 4 kPa, while the remaining total stress of the SB backfill was taken by the pore water and resulted in relatively high excess pore pressure. The total stress of the backfill was slightly less than the geostatic stress due to the existence of low sidewall friction (<1 kPa) at this early stage of primary consolidation.

(3) Consolidation

Prior to consolidation, sidewall friction was simulated by applying \( k_{fc} \) of \( 1.7 \times 10^{-11} \text{ m/s} \) and \( \varphi_{\text{inter}} \) of 30° to the backfill/filter cake interface. After backfilling, the backfill consolidates under the impact of both self-weight and the squeezing from the foundation soil. In the finite element model, the lateral deformation of the backfill, the filter cake and the foundation soil were consistent, and the consolidation was allowed to continue until excess pore pressure of the backfill reduced to <1 kPa. Note that any phases prior to the consolidation were assumed to be instantaneous in the model.

4. Results

4.1. Model verification

The field monitoring at Station 0 + 90 m started instantaneously when the excavation began on July 15th, 2016. Since it took a few days for the backfilling to be completed, the starting time of consolidation in the model was set to be the time when backfilling was completed (i.e., maximum earth pressures and pore water pressure were achieved) at 12:03, July 21st, 2016. Thus, a 5.02-day period referring to the pre-consolidation stage has been added to the model to ensure a consistent timeline with that of the field testing.

To evaluate the applicability and accuracy of the model, simulated earth pressures in three directions and pore water pressures of the SB backfill at \( z = 4 \) and 6 m were compared with field measurements during the first 150-day consolidation process, as shown in Fig. 5. During the consolidation, three “blips” with rapid increase and decrease in measured earth pressures and pore water pressure were captured by field monitoring data, likely are attributed to the disturbance of topsoil cover, instrument placement, an unusually heavy rainstorm, and non-uniform movement of the earth pressure cells on their frame (Malusis et al., 2017). The “blips” had insignificant impact on the long-term measurements, thus were neglected in the modeling. The modeled earth pressures and pore water pressures at 4-m and 6-m depth gradually decreased with time, which are consistent with field measurements.

For \( z = 6 \) m, the modeled transverse earth pressure (\( \sigma_x \)) was 5% lower than the measured \( \sigma_x \) at the beginning of consolidation and remained nearly constant thereafter. The modeled longitudinal earth pressure (\( \sigma_z \)) was 10% lower than the measured \( \sigma_z \) at the beginning of consolidation and maintained an 8–10% difference. The modeled vertical earth pressure (\( \sigma_y \)) was 1 kPa higher than the measured \( \sigma_y \) at the beginning of consolidation, and the difference increased up to 5 kPa (12%) as the consolidation developed. The modeled pore water pressure (\( u \)) started nearly identical to the measured data, and the difference between the two remained below 3%. For \( z = 4 \) m, the modeled earth pressures were in good agreements with the measured data, while the modeled \( u \) was slightly lower than the measured \( u \). Note that the simulated earth pressures in three directions at the same depth were close to each other, similar to the measured data. While the differences are small, the transverse, vertical and longitudinal pressures typically are the major, minor and intermediate principal stresses. The isotropy of earth pressures can be explained by the low effective stresses of the SB backfill.
during the 150-day consolidation, and the earth pressures were primarily carried by pore water with the same \( u \).

Profiles of \( \sigma_z \) and \( u \) with depth at different consolidation time are shown in Fig. 6. At the beginning of consolidation, the modeled \( \sigma_z \) and \( u \) were slightly lower than the measured data at \( z = 2 \) and \( 4 \) m, but close to the measured data at \( z = 6 \) m. As consolidation continued, the modeled \( \sigma_z \) decreased more slowly than the measured \( \sigma_z \). For example, the modeled \( \sigma_z \) at \( z = 2 \) m was 5.6 kPa higher than the measured \( \sigma_z \) after 20 days of consolidation, and 2.6 kPa higher than the measured \( \sigma_z \) after 150 days of consolidation. The modeled \( u \) at \( z = 2 \) m decreased more slowly than the measured \( u \) and was 8.5 kPa higher than the measured \( u \) after 20 days, which can be attributed to groundwater seepage and fluctuations of groundwater table. Similar to \( \sigma_z \), the difference between the modeled and measured \( u \) decreased with time, i.e., reduced to 1.4 kPa after 150 days. The results at \( z = 4 \) and \( 6 \) m shown consistency with those in Fig. 5.

Overall, the modeled earth pressures and pore water pressure with time and depth are in excellent agreements with those measured in situ, implying a good applicability of the model in simulating the state of stresses of the SB cutoff wall during backfilling and consolidation.

4.2. Analysis of consolidation mechanism of SB backfill cutoff wall

In this section, the consolidation mechanism of the SB backfill cutoff wall is analyzed in terms of sidewall friction, consolidation stress, and degree of consolidation using the results obtained from the numerical model.

4.2.1. Sidewall friction

The modeled sidewall friction (i.e., shear stress at the interface), \( \tau \), with depth at different consolidation times is shown in Fig. 7. According to the measured \( \sigma_z \) at \( z = 2 \), 4 and 6 m, the average sidewall friction of the depth sections (\( \tau \)) of 0–2, 2–4 and 4–6 m can be calculated using the following equation with consideration of the vertical force equilibrium of the SB backfill in the corresponding depth section:

\[
\tau = \frac{\gamma_B \Delta h + \sigma_{z1} - \sigma_{z2}}{2} \frac{B}{2} - \frac{\sigma_{z1} - \sigma_{z2}}{2} \Delta h
\]

where \( \Delta h \) is the thickness of the section (i.e., \( \Delta h = 2.0 \) m); \( \sigma_{z1} \) and \( \sigma_{z2} \) are the vertical earth pressures on the upper and lower surfaces of the depth section. The \( \tau \) calculated from the measured data were generally consistent with the range of the modeled \( \tau \) at the corresponding depth section, as shown in Fig. 7 (the calculated \( \tau \) is represented by the dash lines with the colors matching the corresponding consolidation time). Note that the values of \( \tau \) are less than the interface shear strength determined by Eq. (4), due to small relative displacement between two sides of the interface.
The modeled $\tau$ at $z < 5$ m increased rapidly during the first 20 days of consolidation. For example, $\tau$ at $z = 2.5$ m increased to 4.5 kPa, corresponding to 70% of the long-term stabilized value. Meanwhile, $\tau$ at $z = 5$ m reached only 22% of the long-term stabilized value. From 20 to 60 days, $\tau$ of shallower region of the wall increases, whereas that of deeper region changed insignificantly. From 60 to 150 days, $\tau$ at $z > 5$ m increases significantly. After 150 days of consolidation, $\tau$ continuously increased but the rate of increase gradually reduced.

The sidewall friction causes relatively low effective stresses in SB cutoff walls. Since the measured values of hydraulic conductivity are stress dependent, these lower than geostatic stresses must be applied during design and quality control permeability testing in order to avoid an unrealistically low measured value of hydraulic conductivity.

After 60 days, the distribution of modeled $\tau$ had a “W” shape with depth. For example, at 400 days, $\tau$ appeared to reach peak values at 6.5 and 5.8 kPa at $z = 2.5$ and 4.7 m, respectively, but remained at 3.3 kPa at $z = 4$ m (about half of the $\tau$ at $z = 2.5$ m). The W-shape distribution of $\tau$ with $z$ could be mainly attributed to the existence of the soft silty sand layer at $z = 4$–5 m. In a complementary analysis for a similar model with a homogeneous soil layer, $\tau$ increased to a peak value at a shallow position and gradually decreased to 0 with increasing $z$ (not shown in this paper).

### 4.2.2. Consolidation stress and excess pore pressure

The profiles of the vertical total consolidation stress ($p_z$) with depth at the centerline of the trench are shown in Fig. 8. The rate of decrease of $p_z$ was relatively large at the beginning of consolidation (e.g., 0–20 days) and gradually reduced with time. The change of $p_z$ with time was consistent with that of $\tau$ due to the soil arching effect, since $p_z$ at any given depth is affected by the sum of $\tau$ for the overlying soil (Handy, 1985; Evans et al., 1995). The increase of $\tau$ leads to the decrease of $p_z$. For example, $p_z$ at $z = 5$ m was 61.3 kPa after backfilling and reduced to 31.1 kPa (by approximately 50%) after 20 days of consolidation. The rapid decrease in $p_z$ in the first 20 days can be attributed to the rapid increase of $\tau$ induced by the overlying backfill, as shown in Fig. 7. From 20 to 150 days, $p_z$ decreased to 12.8 kPa (by approximately 80% of the initial value). From 150 to 400 days, $p_z$ decreased by 3.7 kPa and became 15% of the initial value. After 400 days, $p_z$ was <20 kPa along the depth except for a narrow region near the bottom of the trench (i.e., $z \geq 5.5$ m), where the $\tau$ was relatively low (see Fig. 7) and cannot carry the increment in self-weight of backfill with depth.

The profiles of excess pore pressure ($u_e$) with depth at the centerline of the backfill are also shown in Fig. 8. It is noted that the decrease of $u_e$ ($\Delta u_e$) is caused by two factors, i.e., the decrease of consolidation stress $p_z$ ($\Delta u_{e,p}$) and the dissipation due to backfill consolidation ($\Delta u_{e,d}$). The decrease of $p_z$ accelerated the decrease of $u_e$, and the decrease of $u_e$ caused by the dissipation ($\Delta u_{e,d}$) was converted into the effective stress $\sigma'_e$, which corresponds to the horizontal value between the $p_z$ and $u_e$ isochrones in Fig. 8. At the beginning of the consolidation (i.e., after backfilling was completed), the effective stress in the backfill was very low and the consolidation stress from the self-weight of the backfill was mostly carried by pore water. The significant decrease of $u_e$ in the first 20 days was primarily contributed by the rapid increase in $\tau$ (see Fig. 7) instead of the dissipation caused by backfill consolidation, since the increase of $\sigma'_e$ was much slower than the decrease of $p_z$ during this period. After 400 days, $u_e$ was <1 kPa in the backfill, indicating the primary consolidation in the SB backfill being close to completion.

The contributions of $\Delta u_{e,p}$ or $\Delta u_{e,d}$ to $\Delta u_e$ were calculated by $(\Delta u_{e,p} / \Delta u_e \times 100\%)$ or $(\Delta u_{e,d} / \Delta u_e \times 100\%)$, and are summarized in Table 3 in terms of depth and consolidation time. Overall, $\Delta u_{e,p}$ contributed to 63–92% of the $\Delta u_e$ whereas $\Delta u_{e,d}$ only contributed to 8–37%, indicating that the decrease of consolidation stress had a greater impact on the decrease of excess pore pressure, relative to the dissipation. The proportion of $\Delta u_{e,p}$ increased during the early consolidation period (the first 60 days) and gradually decreased after 60 days, but remained dominant to the contribution of $\Delta u_e$ relative to $\Delta u_{e,d}$. By the time when modeled consolidation was completed (at 400 days), the proportions of $\Delta u_{e,d}$ were still smaller than that of $\Delta u_{e,p}$, i.e., 37%, 22% and 14% at $z = 2$, 4 and 6 m, respectively.

At any given consolidation time, the contributions of $\Delta u_{e,p}$ to $\Delta u_e$ is smaller than that of $\Delta u_{e,d}$, i.e., 72%, 86% and 92% at $z = 2$, 4, and 6 m, respectively. Since the sidewall friction accumulates with depth, the decrease of consolidation stress increases with depth.

The transverse distribution of $u_e$ in the backfill and filter cake at $z = 4$ m is shown Fig. 9. $u_e$ dropped rapidly in the
the value in bracket is the proportion of the decrease of $u_e$ due to the decrease of consolidation stress $p_z$; $\Delta u_{e,d}$ is the decrease of $u_e$ due to the dissipation of backfill consolidation; and the value in bracket is the proportion of the decrease of $u_e$ due to the corresponding factor.

Table 3
Contributions of decrease of consolidation stress and dissipation of backfill consolidation on the decrease of excess pore pressure.

<table>
<thead>
<tr>
<th>Depth</th>
<th>20 days</th>
<th>60 days</th>
<th>150 days</th>
<th>400 days</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\Delta u_{e,p}$</td>
<td>$\Delta u_{e,d}$</td>
<td>$\Delta u_{e,p}$</td>
<td>$\Delta u_{e,d}$</td>
</tr>
<tr>
<td>2 m</td>
<td>7.7 kPa</td>
<td>4.1 kPa</td>
<td>14.9 kPa</td>
<td>5.8 kPa</td>
</tr>
<tr>
<td></td>
<td>(65%)</td>
<td>(35%)</td>
<td>(72%)</td>
<td>(28%)</td>
</tr>
<tr>
<td>4 m</td>
<td>23.2 kPa</td>
<td>5.5 kPa</td>
<td>35.2 kPa</td>
<td>6.0 kPa</td>
</tr>
<tr>
<td></td>
<td>(81%)</td>
<td>(19%)</td>
<td>(86%)</td>
<td>(14%)</td>
</tr>
<tr>
<td>6 m</td>
<td>33.2 kPa</td>
<td>3.2 kPa</td>
<td>45.6 kPa</td>
<td>4.1 kPa</td>
</tr>
<tr>
<td></td>
<td>(91%)</td>
<td>(9%)</td>
<td>(92%)</td>
<td>(8%)</td>
</tr>
</tbody>
</table>

Note: $\Delta u_{e,p}$ is the decrease of $u_e$ due to the decrease of consolidation stress $p_z$; $\Delta u_{e,d}$ is the decrease of $u_e$ due to the dissipation of backfill consolidation; and the value in bracket is the proportion of the decrease of $u_e$ due to the corresponding factor.

4.2.3. Degree of consolidation

As previously discussed, the decrease of $u_e$ was caused by two factors, i.e., the decrease of consolidation stress and the dissipation due to backfill consolidation. So, the degree of consolidation can be expressed by two corresponding forms, that are:

$$U_p(t) = \frac{\int_0^t u_e(z, 0) - p_z(z, t) \, dz}{\int_0^t u_e(z, 0) \, dz}$$

(7)

$$U_d(t) = \frac{\int_0^t p_z(z, t) - u_e(z, t) \, dz}{\int_0^t u_e(z, 0) \, dz}$$

(8)

where $u_e(z, 0)$ is the initial excess pore pressure at the depth of $z$ and equals to the initial vertical consolidation stress $p_z(z, 0)$; $p_z(z, t)$ is the vertical consolidation stress at the depth of $z$ and time of $t$; and $u_e(z, t)$ is the excess pore pressure at the depth of $z$ and time of $t$. The term $p_z(z, t) - u_e(z, t)$ in Eq. (7) represents the decrement of vertical consolidation stress on backfill due to sidewall friction, while the term $p_z(z, t) - u_e(z, t)$ in Eq. (8) represents the vertical effective stress in backfill. So, $U_p(t)$ and $U_d(t)$ represent the components of decreased $u_e$ attributed by the decrease of consolidation stress and the dissipation due to backfill consolidation, respectively. The total degree
of consolidation of the backfill in the cutoff wall in terms of excess pore pressure, \( U(t) \), is the sum of \( U_p(t) \) and \( U_d(t) \), that is,

\[
U(t) = U_p(t) + U_d(t) = 1 - \frac{\int_0^t u_e(z, t) \, dz}{\int_0^t p_e(z, 0) \, dz} \tag{9}
\]

The relationships between the degree of consolidation and time are shown in Fig. 11. During the entire consolidation process, \( U_p \) was much greater than \( U_d \), which indicates that the decrease of consolidation stress contributes more to the decrease of \( u_e \) than dissipation due to backfill consolidation. When the consolidation was completed (i.e., 400 days), \( U_p \) and \( U_d \) were 77% and 23%, respectively. \( U_p \) increased rapidly in the first 90 days whereas \( U_d \) only started to increase after 40 days. 50% of \( U \) was quickly achieved in 20 days. However, 90% of \( U \) required for a much longer duration, as predicted to be 180 days.

5. Conclusions

A finite-element model has been proposed to simulate the stress state of a full-scale SB backfill slurry trench cutoff wall during the construction and primary consolidation processes. The changes of earth pressures and pore water pressure in the backfill with time and depth were simulated and show excellent agreement with field measurements. The consolidation mechanism in the SB backfill was analyzed in terms of sidewall friction, consolidation stress, degree of consolidation using the results obtained from the numerical model. The following conclusions can be drawn:

(1) At the beginning of consolidation (immediately after backfilling completed), the consolidation stresses and excess pore pressure in the backfill are equivalent to the geostatic stress calculated with buoyant unit weight of backfill. During the consolidation, excess pore pressure decreases as the effective stresses increase due to the pore pressure dissipation; the increase of effective stress and relative settlement at the sidewall interface leads to the increase of sidewall friction, which in turn results in the decrease of the consolidation stress on the backfill and the decrease of excess pore pressure. The cycle of load transfer continues until the consolidation is completed.

(2) The decrease of excess pore pressure in the backfill can be attributed to two factors, that are, the decrease of consolidation stress and dissipation due to backfill consolidation. The former dominates the change in the short-term consolidation, while the contribution of the later becomes more significant to the change in the long-term consolidation. The decrease of excess pore pressure induced by dissipation results in an increased effective stress in backfill. According to the simulation, the decrease of consolidation stress resulted in 63–92% decrease of excess pore pressure.

(3) The backfill/filter cake interface locates adjacent to the transverse outflow boundary. This results in the rapid development of the effective stress and sidewall friction at the interface, which in turn results in a rapid decrease in consolidation stress in the early consolidation period.

(4) The durations for 60% and 90% of total degree of consolidation achieved are approximately 30 and 180 days, respectively, based on the simulation in this case.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.
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