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Prediction of Earth Pressures in Soil-Bentonite Cutoff Walls

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Abstract

This paper presents a review of two models (i.e., arching and lateral squeezing) developed for predicting earth pressures in soil-bentonite (SB) cutoff walls. The assumptions of these existing models are discussed, a modified lateral squeezing (MLS) model is presented, and all three models are compared based on predicted horizontal stresses for representative field conditions. Each model predicts that the stress distribution within a SB cutoff wall may be considerably lower than a geostatic distribution, particularly at depth. The arching model yields the lowest stress distribution but may underestimate the true distribution due to the assumption of rigid trench sidewalls. The MLS model (1) allows sidewall deformation and (2) accounts for the stress-dependent nature of SB backfill compressibility. The study also finds that additional model development is needed to characterize the stress state of a SB cutoff wall in three dimensions.

Introduction

Since the 1970s, soil-bentonite (SB) vertical barriers (cutoff walls) have been widely employed for *in situ* containment of ground water and subsurface contamination. Despite the wide application and common use of SB cutoff walls in practice, limited information exists regarding field performance of existing walls (National Research Council 2007). Moreover, the relationship between field performance and the *in situ* state of stress in constructed SB cutoff walls has not been extensively studied. Previous research suggests that the state of stress within SB cutoff walls is less than that predicted by a geostatic pressure distribution (e.g., see Evans et al. 1985). However, considerable uncertainty exists regarding the true distribution of vertical and horizontal stresses in SB cutoff walls, the mechanisms governing the development of these stresses at the time of construction, and the changes in stress that may occur over time. Stress is an important consideration from the standpoint of hydraulic performance, given that increasing confining pressure typically causes a decrease in hydraulic conductivity and an increase in resistance to chemical attack of soil barriers (e.g., Acar et al. 1985, McCandless and Bodosci 1988, Evans 1994, Shackelford 1994, Filz et al. 2001, Yeo et al. 2005). Laboratory hydraulic conductivity tests conducted on SB backfill specimens using confining pressures based on an assumed geostatic pressure distribution could yield unconservatively low values of hydraulic conductivity.

Mathematical models have been developed in an attempt to predict the state of stress in SB cutoff walls (Evans et al. 1995, Filz 1996). The objectives of this paper are (1) to review the existing models, (2) to assess the assumptions of these models and propose modifications where possible, and (3) to compare the models using properties and conditions representative of field SB cutoff wall installations.

Background

The uncertain nature of the state of stress in SB cutoff walls was first discussed by Evans et al. (1985). These authors hypothesized that frictional forces along the sidewalls of the trench may cause a nonlinear stress distribution in the backfill, such that the vertical stress at a given depth in the wall is lower than that predicted based on geostatics (i.e., the self weight of the SB backfill above the point of interest). Evans et al. (1985) proposed that the vertical stress distribution with depth may follow the general trend illustrated in Fig. 1.

Subsequently, there have been two mathematical models developed to predict the state of stress in SB cutoff walls. The first model, presented by Evans et al. (1995), predicts vertical stresses based on principles conventionally applied to problems involving buried pipelines, commonly termed arching (e.g., see Filz 1996). The free-body diagram used as the basis for the arching model, shown in Fig. 2, includes the overburden stress above the backfill element, the reactionary stress below the backfill element, the frictional resistance along the sidewalls of the trench, and the self-weight of the backfill element. The cutoff wall is assumed to be of unit length, and the groundwater table is assumed to be at the ground surface. Based on these conditions, the vertical effective stress can be solved by summing forces in the vertical direction and integrating with respect to depth (see Costa 1996). Assuming that the interfacial adhesion (*a*) and interfacial friction angle (δ) along the trench sidewalls are equal to the

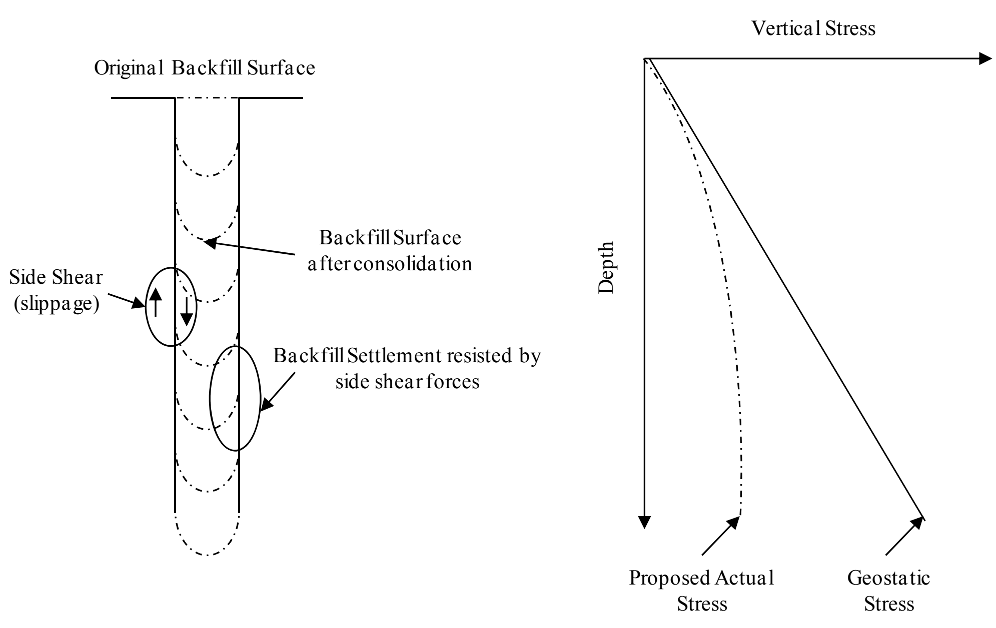
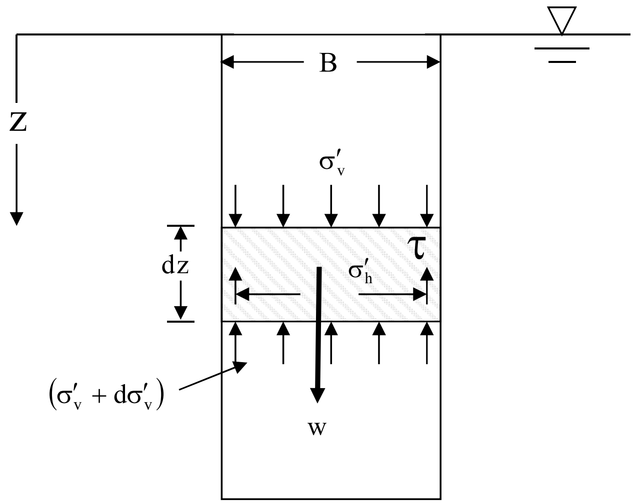


Fig. 1. Schematic illustrating the effect of trench side friction on the stress distribution with depth in a SB cutoff wall (redrawn after Evans et al. 1985).



$\tau = \left(a + k_{ob}\sigma'_v \tan\delta\right)$

$w = \gamma'_b dzB$

B = width of wall

z = depth of interest

γ'_b = unit weight of backfill

k_{ob} = at-rest earth pressure coefficient of the backfill

a = interfacial adhesion between backfill and trench sidewall

δ = interfacial friction angle between backfill and trench sidewall

Fig. 2. Free-body diagram for arching model (redrawn after Evans et al. 1995).

internal cohesion c_b and internal friction angle ϕ'_b of the backfill, respectively (i.e., $a = c_b$ and $\delta = \phi'_b$), the closed-form analytical solution for the vertical effective stress σ'_v is as follows (Evans et al. 1995, Kezdi 1975):

$$\sigma'_v = \frac{\left(\frac{B}{2}\right)\left(\gamma'_b - \frac{2c_b}{B}\right)}{k_{ob}\tan\left(\phi'_b\right)}\left\{1 - \exp\left[-2k_{ob}\left(\frac{z}{B}\right)\tan\left(\phi'_b\right)\right]\right\} \quad (1)$$

The "arching" model given by Eq. 1 predicts that, at a certain depth, side friction will negate any increase in vertical stress associated with the backfill self weight, such that σ'_v approaches a constant value (e.g., see Fig. 1). The arching model assumes that the sidewalls of the trench are perfectly rigid, i.e., no lateral movement of the sidewalls occurs during backfill consolidation. Filz (1996) indicates that this assumption is unrealistic and presents a "lateral squeezing" model that accounts for movement of the trench sidewalls after cutoff wall backfill placement. The lateral squeezing model assumes that inward lateral displacement of the sidewalls must occur in order to maintain horizontal stress equilibrium across the trench sidewalls.

According to Filz (1996), lateral displacement of the trench sidewalls can occur during three stages, as illustrated conceptually in Fig. 3. The first stage involves inward displacement of the sidewalls as the trench is excavated and filled with slurry, thereby reducing the horizontal effective stress (σ'_h) in the soil adjacent to the trench. In this initial stage, σ'_h immediately outside the trench would be lower than the at-rest stress, but higher than the active stress, as evidenced by the fact that the trench does not collapse. The second stage involves replacement of the slurry with SB backfill having a greater

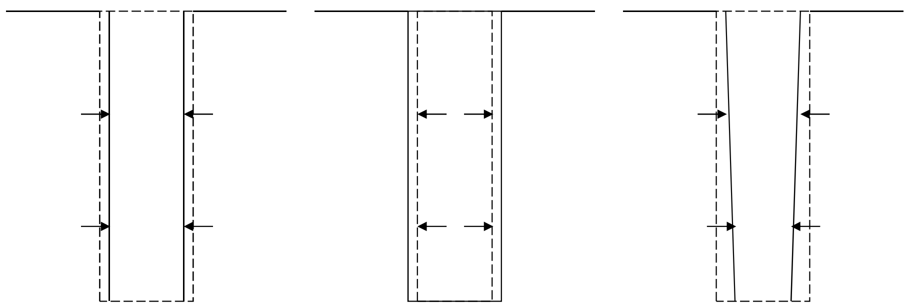


Fig. 3. Stages of sidewall movement during construction and consolidation in an SB cutoff wall (redrawn after Filz 1996).

unit weight than the slurry, potentially causing rebound (i.e., outward displacement) of the trench sidewalls. In the third and final stage, inward displacement of the sidewalls would occur as the backfill consolidates. The lateral squeezing model proposed by Filz (1996) assumes that inward displacement of the trench sidewalls during backfill consolidation begins from an at-rest stress condition in the soil adjacent to the trench.

The lateral squeezing model uses pressure equalization and stress-strain compatibility between a cohesionless native soil outside the trench and the SB backfill within the trench to predict the state of stress within the wall. The governing expression based on Filz (1996) is as follows:

$$\frac{\sigma'_{vo} B}{2D_b} = \frac{\Delta}{k_{am}} \tag{2}$$

where σ'_{vo} is the vertical effective stress outside the trench using a geostatic approach, B is the wall width prior to displacement (Fig. 2), D_b is the constrained modulus of the backfill, Δ is the deflection of the trench sidewall (one side only), and k_{am} is the mobilized active earth pressure coefficient of the soil outside the trench (i.e., $k_o > k_{am} \geq k_a$, where k_o and k_a are the at-rest and active lateral earth pressure coefficients, respectively). Filz (1996) suggests that D_b values for use in Eq. 2 can be determined from one-dimensional consolidation testing of site backfill. Since k_{am} is a function of Δ (e.g., see Clough and Duncan 1991), an iterative approach can be used to determine the appropriate value of Δ and the corresponding k_{am} such that Eq. 2 is satisfied. The horizontal effective stress σ'_h then can be determined by $\sigma'_h = \sigma'_{vo} \cdot k_{am}$

The arching model as expressed in Eq. 1 is a simple closed-form expression, whereas solution of Eq. 2 for lateral squeezing requires an iterative approach. Also, results of a recent field study by Ryan and Spaulding (2008) indicate that backfill shear strength shortly after cutoff wall construction may be approximately constant with depth. These results suggest that side friction may limit stress development with depth, as predicted by the arching model. However, the rigid sidewalls assumption may result in underestimation of the horizontal effective stress by ignoring strength gain in the backfill realized by inward movement of the trench sidewalls.

The assumption of moving sidewalls in the lateral squeezing model may more realistically represent the behavior of SB cutoff walls. For example, Filz (1996) suggests that ground surface displacements observed adjacent to SB cutoff walls may be caused by inward displacement of the sidewalls. Later, Filz et al. (1999) presented a case study in which a building adjacent to a SB cutoff wall was damaged due to ground surface displacements. The displacements described Filz et al. (1999) were attributed to inward movement of the trench sidewalls based on monitoring data from inclinometers installed adjacent to the trench. Therefore, this case study suggests that σ'_h values estimated using the lateral squeezing model may be more representative of field conditions.

The lateral squeezing model may require the determination of k_{am} values at deformations (Δ) less than those needed to achieve fully active conditions in the soil adjacent to the trench. These values of k_{am} are a function of Δ as well as the friction angle of the native soil outside the trench. In addition, the lateral squeezing model requires an input value of the constrained modulus, D_b, to represent the consolidation behavior of the backfill. Since D_b is not a constant but rather is a function of consolidation stress, D_b is expected to vary as a function of depth. These two aspects of the lateral squeezing model are addressed further in the development of a modified lateral squeezing model, presented below.

Modified Lateral Squeezing Model

One of the practical challenges associated with use of the lateral squeezing model to estimate σ'_h is determining a proper relationship between k_{am} and Δ. Estimated values of normalized deformation (i.e., Δ/H, where H = wall height) required to mobilize fully active and passive conditions for different soil types are given by Clough and Duncan (1991) and are listed in Table 1. As described by Filz (1996), the normalized deformations in Table 1 may be used as a basis for determining k_{am}. If inward displacement of the trench sidewalls is not sufficient to cause fully active conditions in the adjacent native soil, then k_{am} will be less than the at-rest earth pressure coefficient k_o but greater than the active earth pressure coefficient k_a (i.e., $k_a < k_{am} < k_o$). In such cases, both k_o and k_a must be known in order to estimate k_{am}. Thus, for the purposes of the modified lateral squeezing model presented herein, values of the effective friction angle (ϕ') have been assigned to each soil type. The assigned values of ϕ' are included in Table 1 along with corresponding values of k_o and Rankine values of k_a and k_p.

Table 1. Normalized active and passive deformations for different soil types, along with corresponding assigned friction angles and lateral earth pressure coefficients.[1]

Soil Type	Δ/H			Assigned Properties[2]			
	At Rest	Active	Passive	ϕ' (°)	k_o	k_a	k_p
Dense Sand	0	0.001	0.01	40	0.357	0.217	4.59
Med. Dense Sand	0	0.002	0.02	35	0.426	0.273	3.69
Loose Sand	0	0.004	0.04	30	0.500	0.333	3.00
Silt	0	0.002	0.02	25	0.577	0.406	2.46

[1] Soil types and Δ/H values from Clough and Duncan (1991)

[2] $k_o = 1 - \sin\phi'$; $k_a = \tan^2(45 - \phi'/2)$; $k_p = \tan^2(45 + \phi'/2)$

In Fig. 4, values of k_o and k_a are plotted against the corresponding values of Δ/H for each of the four soil types in Table 1 to illustrate the manner in which k_{am} is assumed to vary between k_o and k_a. Various s-shaped curves have been presented that define the relationship between lateral earth pressure coefficient and deformation. Although the mathematic expressions for these curves typically are not reported (e.g., see Cernica 1982, Spangler and Handy 1982, and Murthy 2003), Clough and Duncan (1991) suggest that the relationship between k_{am} and Δ/H is a logarithmic spiral. The curves shown in Fig. 4 are second-order polynomials that are close approximations of the the logarithmic spirals suggested by Clough and Duncan (1991). The second-order equations may be used to estimate k_{am} for a given value of Δ/H that is less than the limiting value of Δ/H required to mobilize fully active conditions (see Table 1). As illustrated in Fig. 4, k_{am} = k_a for deformations in excess of these limiting values of Δ/H.

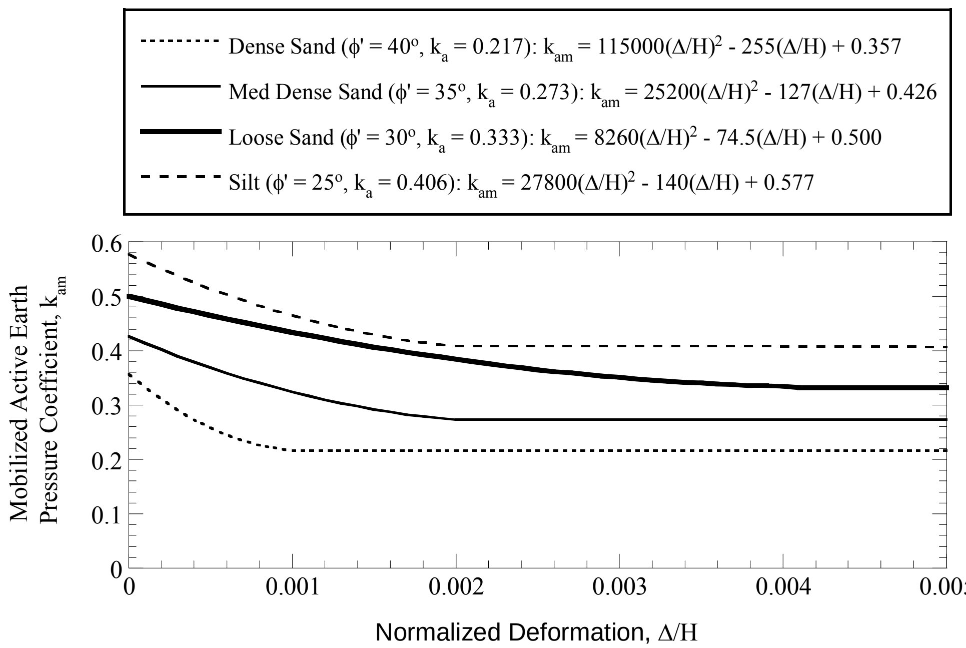


Fig. 4. Mobilized active earth pressure coefficients (k_{am}) plotted as a function of normalized deformation (Δ/H) for the four soil types in Table 1.

A primary assumption of the original lateral squeezing model that is addressed herein is the use of a single, constant value of the constrained modulus, D_b, in Eq. 2. Conventionally, D_b is defined as the inverse of the slope of the stress-strain (σ′-ε) curve from a one-dimensional consolidation test (i.e., $D_b = d\sigma'/d\varepsilon$). Filz (1996) defines D_b based on a secant connecting the origin of the σ′-ε curve (i.e., σ′ = ε = 0) to a selected point of non-zero σ′ and ε. The resulting expression (see Eq. 3 in Filz 1996) for D_b is:

$$D_b = \frac{\sigma'}{\varepsilon} \qquad (3)$$

In either case, D_b is not a constant and can vary widely as a function of σ′.

For example, consider the one-dimensional consolidation test data shown in Fig. 5 for SB backfill collected from a cutoff wall site in eastern Pennsylvania. As illustrated in Fig. 5a, the stress-strain curve obtained in this test is not linear when the data are plotted on an arithmetic scale. If D_b is determined using the secant-based approach illustrated in Fig. 5a, values of D_b ranging between 500 and 1600 kPa are possible depending upon the value of σ′ at which D_b is evaluated.

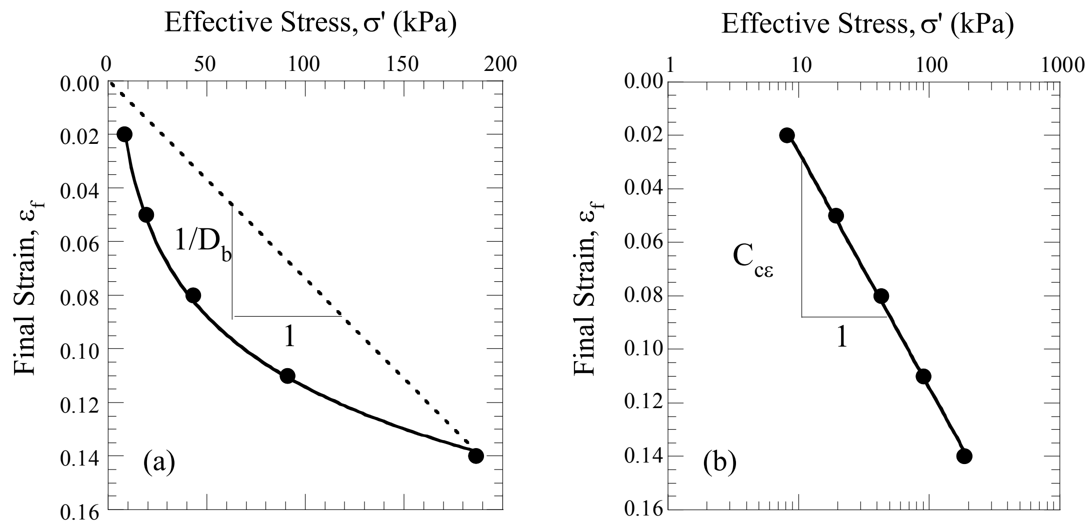


Fig. 5. One-dimensional consolidation test results for SB backfill collected from a cutoff wall site in eastern Pennsylvania.

Conversely, the slope of the σ′-ε curve is constant when the same data are plotted on a semi-logarithmic scale, as shown in Fig. 5b. In this case, the slope is termed the modified compression index, $C_{c\varepsilon}$, and is expressed as follows:

$$C_{c\varepsilon} = \frac{d\varepsilon}{d \log \sigma'} \tag{4}$$

Integration of Eq. 4 yields the following general relationship between ε and σ′:

$$\varepsilon = C_{c\varepsilon} \log \sigma' + C_1 \tag{5}$$

where the constant C_1 is the strain at a unit effective stress, (i.e., $C_1 = \varepsilon$ at $\sigma' = 1$). Best-fit values of $C_{c\varepsilon}$ and C_1 for a particular set of consolidation test data (e.g., Fig. 5b) can be obtained by least-squares regression using Eq. 5. Substitution of Eq. 5 into Eq. 3 yields the following relationship between $C_{c\varepsilon}$ and the secant-based D_b:

$$D_b = \frac{\sigma'}{C_{c\varepsilon} \log \sigma' + C_1} \tag{6}$$

For constrained lateral squeezing of a SB cutoff wall, σ' in Eq. 6 is the horizontal effective stress σ'_{ho} in the native soil immediately outside the trench. In this case, substitution of Eq. 6 into Eq. 2 yields the following expression:

$$\frac{\sigma'_{vo}B\left(C_{c\varepsilon}\log\sigma'_{ho}+C_1\right)}{2\sigma'_{ho}}=\frac{\Delta}{k_{am}} \tag{7}$$

Moreover, since $\sigma'_{ho}=k_{am}\sigma'_{vo}$, Eq. 7 may be written as follows for the case in which the ground water table is at the ground surface:

$$\sigma'_{ho}=\gamma'_o z k_{am}=10^{\left(\frac{2\Delta-BC_1}{BC_{c\varepsilon}}\right)} \tag{8}$$

where γ'_o is the buoyant unit weight of the soil outside the trench. Thus, Eq. 8 represents a modified lateral squeezing model that accounts for the stress dependency of the constrained modulus by incorporating a modulus that is defined by the modified compression index $C_{c\varepsilon}$, commonly evaluated in one-dimensional consolidation tests. This modified model is solved iteratively by determining Δ and the corresponding k_{am} (see Fig. 4) that satisfies Eq. 8 for a given depth z. The resulting values of σ'_{ho} are assumed to be equivalent to σ'_h in the backfill based on horizontal stress continuity across the trench sidewalls.

Model Comparisons

Predicted horizontal effective stress distributions in a SB cutoff wall based on at-rest geostatic conditions (i.e., $\sigma'_h=k_{ob}\gamma'_b z$), the arching model, and the original and modified lateral squeezing models are compared in Fig. 6. In all simulations, the wall thickness B = 1 m and the groundwater table is at the ground surface. The soil properties used in each model also are shown in Fig. 6. The backfill is assumed to be cohesionless and to exhibit the stress-strain characteristics shown in Fig. 5. The native soil is assumed to be a medium dense sand (see Table 1 and Fig. 4).

The results in Fig. 6 show that (1) all three models predict lower σ'_h at a given depth than the geostatic case, and (2) the differences between the geostatic σ'_h and the values of σ'_h predicted by the models increase with depth in the wall. The deviation from the geostatic σ'_h at z = 30 m is approximately 50 kPa for the modified and original lateral squeezing models and approximately 140 kPa for the arching model. As stated previously, the arching model may underestimate σ'_h for vertical barriers due to the assumption of rigid sidewalls. The modified lateral squeezing model predicts stresses within the limits of those predicted by the original lateral squeezing model for the minimum and maximum values of D_b exhibited by the data in Fig. 5. Despite the differences in the model predictions, all three models illustrate that the horizontal stress distribution with depth in a SB cutoff wall may be considerably lower than a geostatic distribution, particularly in deep walls. This factor should be considered when selecting confining pressures for laboratory hydraulic conductivity tests on SB backfill.

Parameter	Units	Value
Lateral Squeezing (LS) Model		
γ'_o	kN/m^3	11.2
D_b (min)[1]	kPa	500
D_b (max)[1]	kPa	1600
ϕ'_o	degrees	35
Arching Model and Geostatic		
γ'_b	kN/m^3	9.7
c_b	kPa	0
ϕ'_b[2]	degrees	30
k_{ob}	---	0.5
Modified Lateral Squeezing (MLS) Model		
γ'_o	kN/m^3	11.2
$C_{c\varepsilon}$[1]	---	0.10
C_1[1]	---	-0.09
ϕ'_o	degrees	35

[1] values based on data in Fig. 5.
[2] from Baxter et al. (2005)

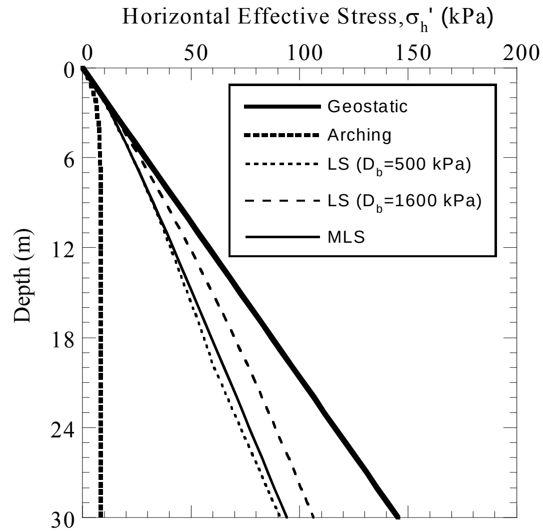


Fig. 6. Horizontal effective stress distributions in SB cutoff wall (B = 1 m) predicted by arching, lateral squeezing, and geostatic models.

Conclusions

This paper provides a review of available models (arching and lateral squeezing) to predict the state of stress in a SB cutoff wall, highlights the underlying assumptions and possible advantages of each of these models, and presents a modified lateral squeezing model that explicitly accounts for the stress-dependent nature of SB backfill compressibility. The modified lateral squeezing model improves predictions of lateral stresses in SB cutoff walls but does not directly address vertical stresses in these walls. Although lateral squeezing effects may govern the horizontal stress normal to the wall alignment, arching still may influence the vertical stress in the wall. Given the indications from limited field studies that shear strength of SB backfill may not increase substantially with depth, the mean stress may reflect some combination of lateral squeezing and arching. Another potential limitation of the original and modified lateral squeezing models is that k_o conditions are assumed in the ground adjacent to the trench prior to backfill compression. Moreover, all of the models are two-dimensional (plane-strain) models that inherently assume no lateral deformation normal to the longitudinal axis of the wall. Field studies are needed to characterize the stresses and resulting deformations in all three dimensions, and additional model development may be warranted based on the outcome of such studies.

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