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Numerical modeling of Mechanically Stabilized Earth Walls

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Abstract

This paper describes the numerical modeling details of an example full-scale physical steel reinforced soil wall taken from a series of structures constructed at the FHWA Reinforced Soil Project site at Algonquin, Illinois. In this research, numerical analyses were performed with the finite difference based computer program FLAC (Fast Lagrangian Analysis of Continua). Details of the numerical model and constitutive modeling of the component materials are described. The modeling results are presented and compared to the field measurements from case histories to assess the accuracy of the numerical approach. Example parametric analyses are carried out using the verified numerical code to investigate the influence of internal stability design factors on Earth Pressure Coefficient of a theoretical wall of 7.6 m height. The lessons learned here are of value to modelers who wish to: (a) explore the mechanical behavior of these systems; (b) generate data to fill in the gaps in performance data from the limited number of monitored structures reported in the literature; and (or), (c) carry out parametric analyses.

Keywords: reinforced soil, numerical analyses, Earth Pressure Coefficient, FLAC

1. Introduction

A recent study by Allen et al.[1] and Bathurst et al. [2] of the design, analysis, and performance of instrumented reinforced soil walls constructed in the field has demonstrated that limit equilibrium-based analysis methods (AASHTO 2002) over-estimate reinforcement loads under operational conditions (on average) by a factor of 2 to 3. As the emphasis shifts to prediction of reinforced soil walls under operational (serviceability) conditions the demand for improved and more accurate design models for these systems increases. Furthermore, the need to develop calibrated serviceability limit states design models for design engineers requires data that are difficult to obtain due to the limited number of monitored field structures.
Useful reviews of geosynthetic reinforced soil wall numerical modeling efforts can be found in the papers by Bathurst and Hatami [3], and Hatami and Bathurst[4]. This paper describes the numerical modeling details of an example full-scale physical metallic reinforced soil wall from a series of structures constructed by FHWA at Algonquin, Illinois.
The numerical model is first validated against measured results from instruments. The validated model is then used to investigate the influence of internal stability design factors on Earth Pressure Coefficient of this type of walls. Four different backfill unit weight, six different thickness for facing panels, six different value for soil friction angle, four value for vertical spacing of reinforcement, five different wall face batter and six different Reinforcement length to wall height ratio were included in the study to examine their respective influences on the predicted Earth Pressure Coefficient of the wall.

2. Physical model

1.2. Wall configurations

The Federal Highway Administration (FHWA) built a series of full-scale instrumented test walls 6 m high at Algonquin, Illinois. These test walls were constructed as part of a FHWA study to investigate the behavior of mechanically stabilized earth (MSE) walls [5].
Five of the walls utilized the same precast concrete facing panels. One of these sections (Wall 1) used Reinforced Earth Company (RECO) steel strips. The control structure (Wall 1) was constructed with 8 layers of reinforcement at a 0.76 m vertical spacing and extending 4.1m into the backfill soil. The performance of

wall was measured using inclinometers and surface optical surveys for deflections, and strain gages and extensometers for internal strain distributions.

The results of numerical models described in this paper are compared to measurements taken from the control (reference) physical model test wall (Wall 1). Figure (1a) shows the wall geometry, material properties, and instrumentation details for the control wall (Wall 1).

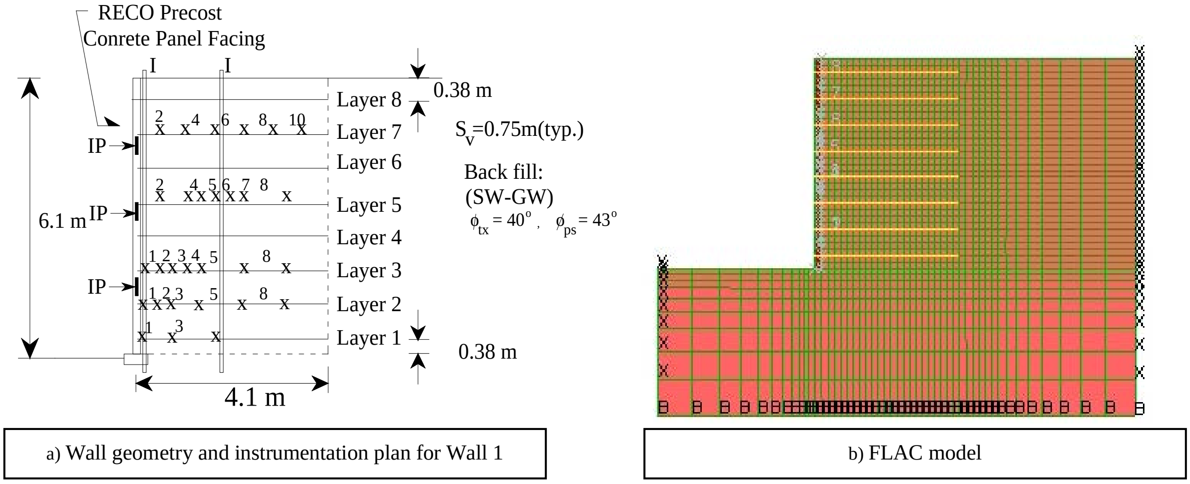


Figure 1. Physical and numerical models

2.2. Materials

The gravelly sand backfill used was well graded gravelly sand with a maximum particle size of 50 mm and a *d50* size of 4 mm. Soil shear strength was determined through triaxial testing in this case. A peak soil friction angle was provided for the sand. The unit weight of the soil was measured in-situ with a nuclear densometer during wall construction after compaction. Foundation conditions beneath the wall consisted of 5 m of dense gravelly sand underlain by very dense sandy silt. The tensile strength (Fu = 520 MPa) and modulus (200,000 MPa) of the steel were based on minimum ASTM specification requirements for the steel used.

3. Numerical Modeling

1.3. Numerical Approach

The finite difference-based program FLAC 2D (Itasca 2001) was used to carry out the numerical simulations. An initial modeling effort for this Wall using FLAC has been reported by Lee [6]. Figure 1b shows the FLAC numerical grid used in this study. FLAC is suitable for modeling problems that involve large deformations and plastic behavior. In addition, complex user-defined constitutive models for component materials can be programmed and included in the analysis, as needed.

2.3. Model Validation

Four kinds of material elements include, elastic material elements , Mohr-Coulomb material elements, structure (cable) elements, and interface elements, were used to create models of MSE retaining structures. Elastic material elements were used to represent the material with higher strength and a linear stress-strain behavior under working stresses (facing materials such as precast concrete panels and modular blocks); Mohr-Coulomb material elements were used to represent the soil, including the backfill and foundations of MSE retaining structures. Cable elements were used to represent the steel strip reinforcement and Interface elements were used to describe the interaction between different materials or the discontinuities between the same materials such as interfaces between backfill soil and structural facing, and interfaces between structural facing units. Table 1 lists the input properties of this model.

3.3. Backfill

The compacted backfill soil was modeled as a homogenous, isotropic, nonlinear elastic-perfectly plastic material with Mohr-Coulomb failure criterion and dilation angle. The sand friction angle and dilation angle values were determined from triaxial, plane strain and direct shear tests [5]. The measured value for the sand peak friction angle from plane strain tests was consistent with values estimated from direct shear and triaxial test results using empirical relationships proposed Lade and Lee [7].
The backfill soil nonlinear elastic response prior to peak strength was simulated using the nonlinear stress-dependent hyperbolic model proposed by Duncan et al [8] and implemented in the numerical model for reinforced soil walls as described by Hatami and Bathurst [9].

4.3. Reinforcement

The Geometry of Steel Strip reinforcement in Wall 1 was 50 x 4 (ribbed steel strip). The reinforcement layers in the FLAC simulations were modeled using Two - noded cable elements that they were attached to the nodes of the material elements with constant tensile stiffness, J= 54690kN/m and yield strength, Tyield= 67.9kN/m, that they calculated per meter of wall based on the coverage ratio (the width of the reinforcement divided by the center to center horizontal spacing).

Table 1- Input properties of Model

Input property	Soil Friction Angle	Soil Dilation Angle	Soil Cohesion	Soil Moduli	Ribbed Steel Strip
Model	43 deg	15 deg	0 kPa	K=1100 Rf=0.73 n=0.5	E = 1.2E7 kN/m2 Yield = 1E6 kN, Ycomp = 1E6 kN

5.3. Boundary Conditions

A fixed boundary condition in the horizontal direction was assumed at the numerical grid points on the backfill far-end boundary to allow for free settlement of the soil along that boundary. A fixed boundary condition in both vertical and horizontal directions was used at the bottom boundary of the foundation simulating bedrock.
The interface between dissimilar materials was modeled as a linear spring-slider system with interface shear strength defined by the Mohr-Coulomb failure criterion and properties as given in Table 2.

Table 2- Interface Properties used in the Wall Models

Interface between soil and facing units					
soil type	Normal Stiffness, k_n (kN/m)	Shear Stiffness, k_s (kN/m)	Friction Angle, δ (o)	Cohesion, c (kPa)	Dilation Angle, ψ (o)
SW	373000	187000	2/3x43	0	0

6.3. Wall Construction

The wall models were constructed in 0.19 m backfill lifts. To reduce the iteration time, we select an aspect ratio of 0.8 for before and bottom of reinforced zone and 1.2 for after it. The soil and reinforcement elements were constructed in layers. Computations were carried out in large-strain mode to ensure sufficient accuracy in the event of large wall deformations or reinforcement strains and to accommodate the moving local datum as each row of facing units and soil layer was placed during construction simulation. In this research, a stress ratio less than 0.01 was used as the equilibrium criteria for intermediate stages (during wall construction) and stress ratio less than 0.001 was used as the equilibrium criteria for final stages (after construction). MSE wall construction procedures can be modeled as follows:
1. Place backfill soil (Mohr-Coulomb) and facing (elastic) elements of first layer.
2. Insert the interface between the soil and facing elements.
3. Place reinforcement (cable) elements.

4. Place backfill soil (Mohr-Coulomb) and facing (elastic) elements of next layer.
5. Insert the interface between the soil and facing elements, including interfaces between facing elements of different layers, if any.
6. Solve the model to intermediate equilibrium state (intermediate equilibrium criteria: srat=0.01) , and
7. Repeat steps 3 to 6 until the wall is completed.

7.3. Result of the Validation Analysis

1.7.3 Wall deflection

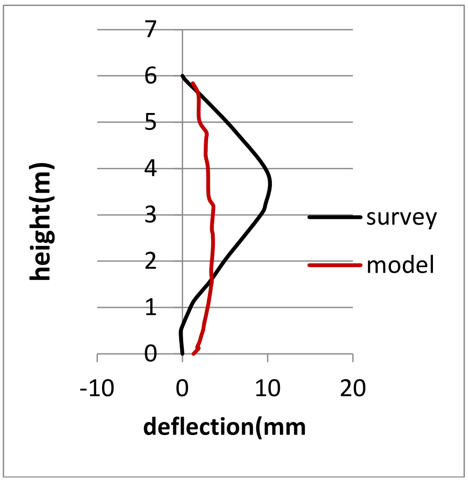
Figure (2a) shows a comparison of the measured and predicted facing lateral deformations for FHWA Wall 1. The Model was able to predict both the shape and maximum deflection of the wall within reasonable agreement. Because of the high stiffness of the reinforcement of Wall 1, the magnitudes of face deflections and the reinforcement strains are very small. Thus the potential error in the measurements due to the accuracy of the optical survey could be very close to the magnitude of the measurements themselves. As shown in Figure (2a), Models predicted very small survey face deflections, and the difference between predicted and measured maximum deflection is only about 5 mm. Therefore, the level of agreement between the predicted and observed wall deformation results observed in Figure (2a) is considered as satisfactory and within the accuracy of experimental measurements.

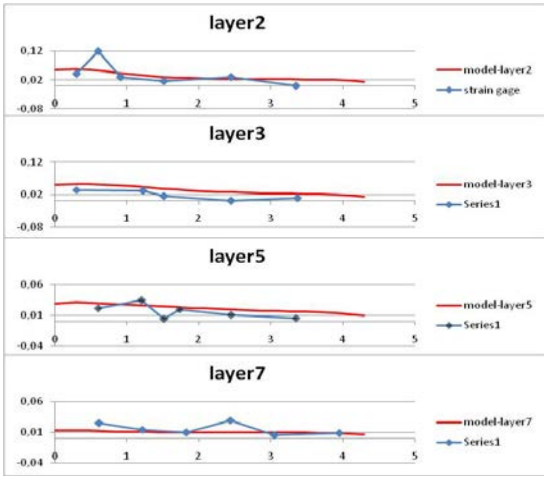
2.7.3 Reinforcement Strain Distribution

Results of Model showed a good agreement to the measured reinforcement strain distributions (Figure (2b)). Because of the high stiffness of the reinforcement of Walls 1 and 3, the magnitudes of the reinforcement strains are very small. Model predicted very small reinforcement strains, and the difference between predicted and measured maximum strain is less than 0.03% except where the strain gage measurements appear to have been defective.

3.7.3 Reinforcement Tension Distribution

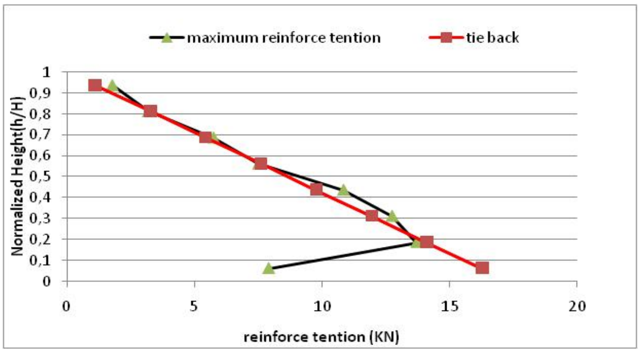
Distributions of reinforcement tensions of each layer obtained from the developed numerical models were found to be different from the distribution assumed in present design procedures. For example, the tie-back wedge method assumes that the reinforcement tension is highest at the bottom of the wall and decreases with wall height to the smallest values at top of the wall. Modeling results shown in this section indicated that the reinforcement tensions increased from top of the wall as the depth increased, and reached their maximum value at elevations between 0.5H to 0.2H, where H is the height of the wall. After reaching their maximum values, the reinforcement tensions started to decrease and had very small values at the bottom of the walls.
As shown in Figure (2c), Modeling showed that the tie-back method was able to predict the tensions of steel reinforcements quite well, especially in the upper half of the wall.





a) Predicted and measured wall face deflection of Wall 1

b) Predicted and measured reinforcement strain distributions of Wall 1



c) Distributions of reinforcement tensions of Wall 1

Figure 1. Modeling Results

4.7.3 Discussion and Conclusions

1. The developed model was able to reproduce the field measurements of deflections of Wall1 within reasonable ranges. The material property determination procedures and modeling techniques developed in this research were found to be suitable for predicting the external performance of various MSE walls.
2. The developed models were able to reproduce reasonably well in the reinforcement strain distributions of MSE walls, so accurate material properties were found to be the key to reproducing the internal performance data of MSE walls.
3. Reinforcement tensions calculated using the tie-back wedge method appeared to be much higher, especially at the lower half of the walls, than those predicted by the numerical models that were able to reproduce both external and internal performance of GRS walls. This observation verifies that the conventional design method tends to over-design the reinforcement tensions in the lower part of the wall. Possible reasons are that the conventional method uses the lateral earth pressure distribution without modifications for soil-reinforcement interactions and toe restraint.

4. The parametric analysis

1.4. Lateral Reinforced Earth Pressure

An important internal design consideration of MSE retaining structures is the lateral earth pressure distribution behind the face of the wall. However, none of the lateral earth pressure distributions used in present MSE wall design procedures have clearly taken the reinforcing effects contributed by reinforcement into account.
Direct measurements of the lateral earth pressures behind the face of the MSE walls are not available today because accurate field measurements of earth pressures are virtually impossible to obtain.
It is also difficult to determine K_{comp} (It's defined as the composite lateral earth pressure coefficient of the reinforced soil) because the state of the backfill soil (K_{soil}), whether active, at rest, or passive, is hard to determine. However, reliable prediction of the MSE composite lateral earth pressures can be obtained from well-developed numerical models that are capable of reproducing the internal strain measurements within a MSE wall using Equation 1. In Equation 1, modeling results of horizontal stresses of soil elements behind the wall faces were used as the horizontal reinforced earth pressure, σ_h, in the equation.

$$k_{comp} = \frac{\sigma_h}{\gamma . z} \tag{1}$$

Where σ_h = horizontal reinforced earth pressure obtained from numerical model

Extensive parametric studies were performed to investigate the influence of internal design factors such as layer vertical spacing, soil strength properties (such as soil friction angle and soil unit weight), facing batter, the length of reinforcement to the height of wall and the thickness of facing panels on Earth Pressure Coefficient of this type of walls. In this parametric study, numerical models of FHWA Algonquin concrete panel test wall (Wall 1) were used as the fundamental models of the parametric study but for better investigation the height of wall increased to 7.6m.These analyses were performed by varying only one design factor in each group while the other factors were fixed.

Table 3- Input material properties used in parametric study

Input material properties	Values
Soil unit weight	16, 18, 20 and 22kPa
The thickness of facing panels	10, 15, 20, 25, 30 and 35cm
Soil friction angle	30, 35, 40, 45, 50, and 55 degrees
Reinforcement vertical spacing	0.38, 0.57, 0.76 and 0.95m
Wall face batter from the vertical	0, 5, 10, 15 and 20 degrees
Reinforcement length to wall height ratio(L/H)	0.4, 0.5, 0.6, 0.7, 0.8 and 0.9

5. Acknowledgments

1. Four vertical spacing (Sv), 0.38m, 0.56m, 0.76m, and 0.95m, were used in the parametric analysis models. K/Ka was found to be affected by vertical spacing of the reinforcements. Especially at the upper half of the wall, the magnitude of K/Ka increased as the reinforcement vertical spacing decreased but at the lower half of the wall it's approximately constant.(Figure 3a)
2. Six values for L/H, 0.4, 0.5, 0.6, 0.7, 0.8 and 0.9m, were used in the parametric analysis models. As shown in the Figure 3b, at the lower half of the wall, the magnitude of K/Ka is approximately constant as facing panel thickness increased, but at the upper half of the wall, the variation of that, don't follows from any specific low and it alternatively increases and decreases.

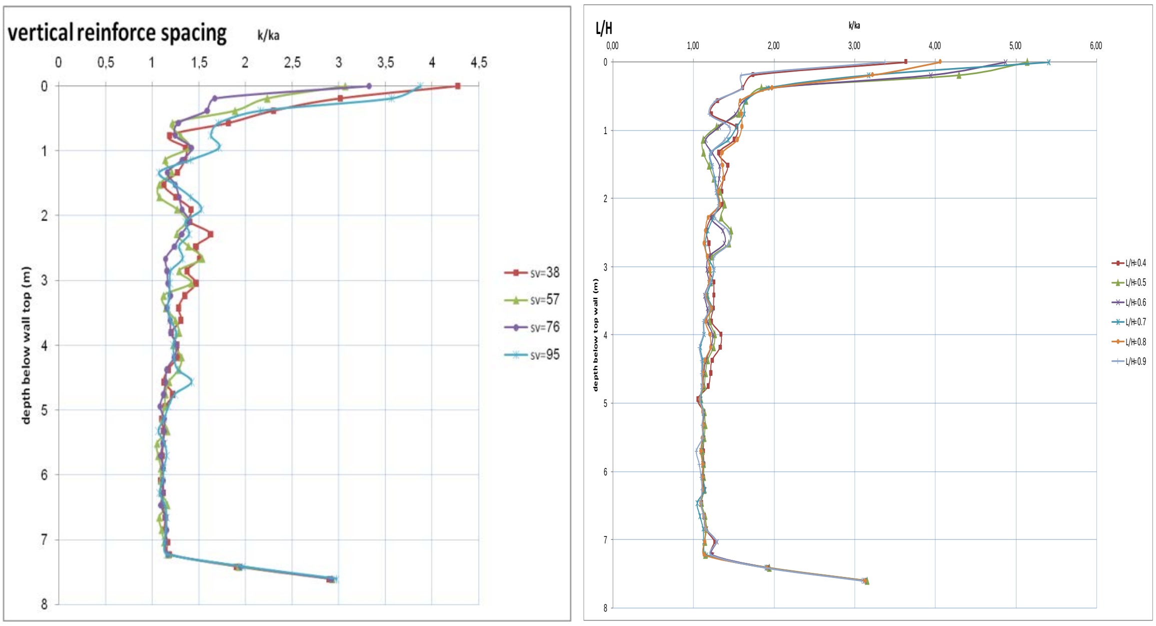


Figure 2. K/Ka profiles versus depth bellow wall top (a): while vertical reinforce spacing varied.(b): while L/H varied

3. Four types of granular soils with different unit weight were chosen as the backfill materials for the parametric analysis. The magnitude of K/Ka (composite lateral earth pressure coefficient to active earth pressure) was not affected by this factor. As shown in the Figure 4a, different values of unit weight created similar shape for K/Ka and available difference between them is inconsiderable.

4. Six values for thickness of facing panels, 10cm, 15cm, 20cm, 25cm, 30cm and 35cm, were used in the parametric analysis models. Lateral earth pressure coefficient profiles shown in Figure 4b indicate that, K/Ka was not affected by the thickness of facing panels.

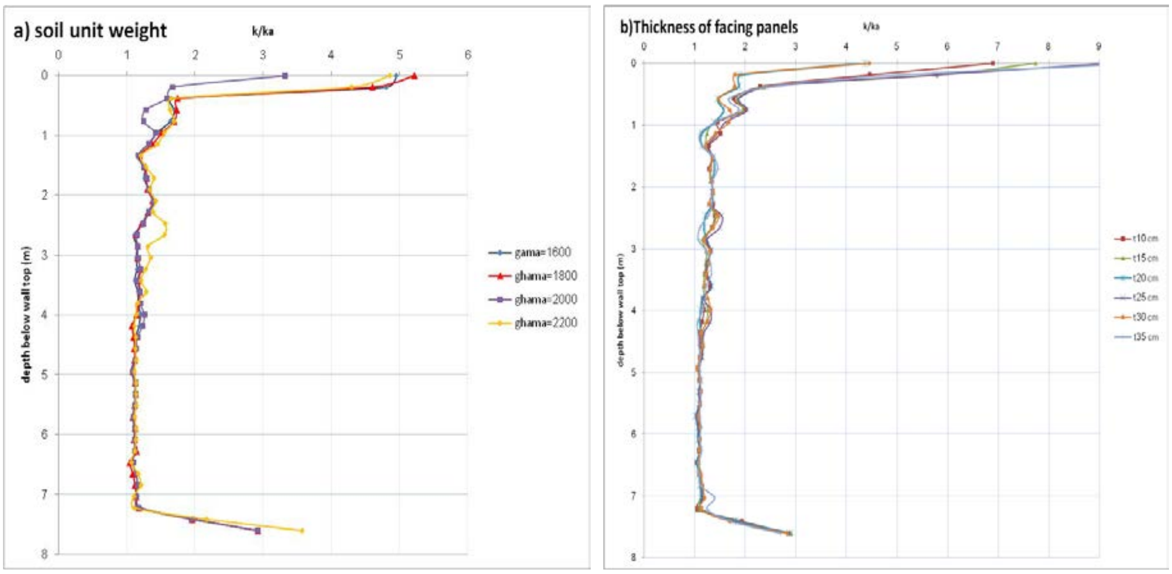


Figure 3. K/Ka profiles versus depth bellow wall top (a): while soil unit weight varied. (b): while thickness of facing panels varied

5. Six types of granular soils with different friction angle were chosen as the backfill materials for the parametric analysis. K/Ka was found to be affected by soil friction angle. As shown in the Figure 5a, the value of K/Ka increased as the soil friction angle increased. It has to be noted that the active earth pressure coefficient is affected by soil friction angle too. So for better understanding from the influence of soil friction angle on earth pressure coefficient, we can plot (K) versus H instead of K/Ka. Lateral earth pressure coefficient profiles shown in Figure 5b indicate that the K value decreased as friction angle increased.

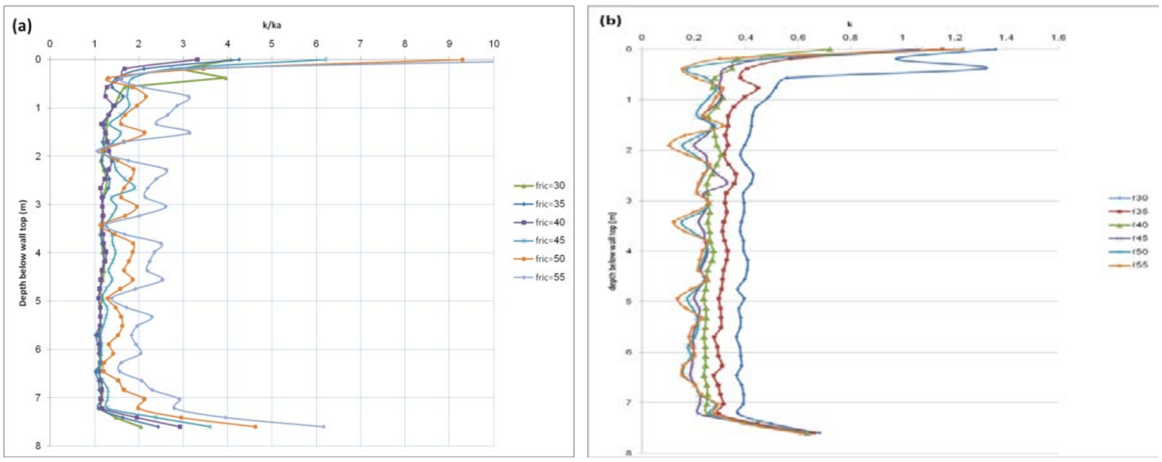


Figure 4. (a) K/Ka profiles versus depth bellow wall top while soil friction angle varied (b): K profiles versus depth bellow wall top while soil friction angle varied

6. Five values for wall face batter from the vertical, 0, 5, 10, 15 and 20 degree, were used in the parametric analysis models. K/Ka was found to be affected by wall face batter. As shown in the Figure 6a, the value of K/Ka increased rapidly as the soil friction angle increased and maximum values occurred in reinforcement locations. It has to be noted that the active earth pressure

coefficient is affected by wall face batter too. So for better understanding from the influence of wall face batter on earth pressure coefficient, we can plot (K) versus H instead of K/Ka. Lateral earth pressure coefficient profiles shown in Figure 6b indicate that the K value for walls with face batter less than 10 degree is approximately constant and for walls with face batter10 degree or more increased as wall face batter increased.

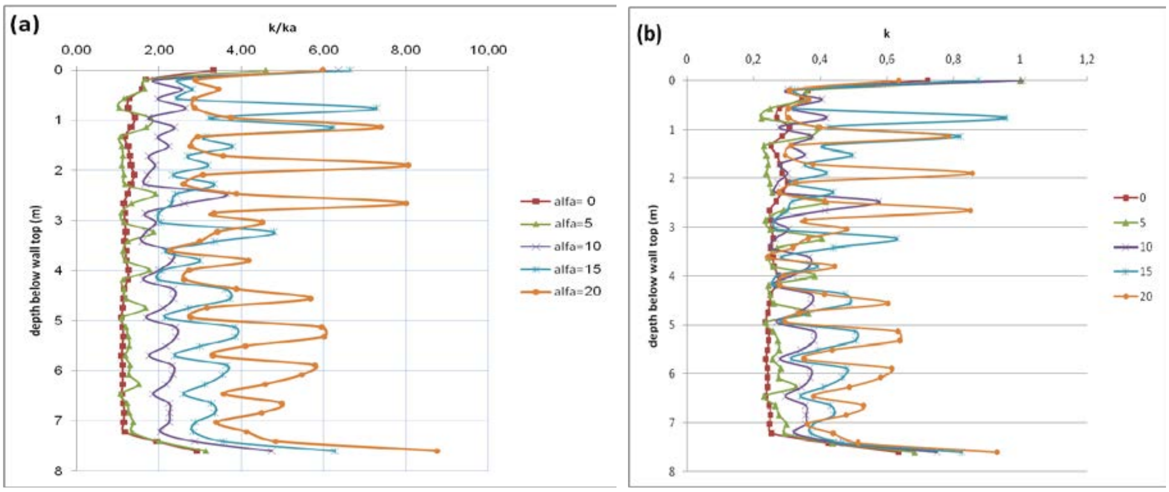


Figure 5. (a) K/Ka profiles versus depth bellow wall top while wall face batter varied (b): K profiles versus depth bellow wall top while wall face batter varied

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