Modified Unit Cell Approach for Modelling Geosynthetic-Reinforced Column-Supported Embankments

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Abstract: Geosynthetic-reinforced and column-supported (GRCS) embankments have proven to be an effective construction technique for fills on soft foundations. The paper introduces a modified unit cell approach to model GRCS embankments supported by deep mixed column walls. The modified unit cells include linear elastic springs at one or both vertical boundaries to simulate lateral displacements of the embankment fill and foundation soil. Program FLAC is used to compare numerical outcomes using the modified unit cells with those using the typical unit cell arrangement with lateral rigid side boundaries. Numerical results demonstrate good agreement between simulations using small-strain and large-strain modes in some cases and large differences in other cases. Lateral displacements of the embankment fill and foundation soil using the modified unit cells are shown to have large influence on reinforcement loads. Finally the paper demonstrates that calculated reinforcement loads are sensitive to choice of small-strain or large-strain mode when using program FLAC.

Key Words: Column-supported embankments, numerical modelling, unit cell, deep mixing, geosynthetic reinforcement, FLAC

1. Introduction

Embankments over soft foundations must be designed to avoid bearing capacity failure of the foundation, unacceptable lateral spreading of the embankment fill, and damage to adjacent structures due to large differential settlements. An effective technique to overcome these challenges is to use geosynthetic-reinforced and column-supported (GRCS) embankments (**Figure 1**). Common support types for GRCS embankments are cement-soil deep mixing (DM) columns (e.g., **Bergado et al. 1999; Borges and Marques 2011; Bruce et al. 2013; Chai et al. 2015; Forsman et al. 1999; Han et al. 2007; Huang and Han 2009, 2010; Huang et al. 2009; Lai et al. 2006; Liu and Rowe 2015; Yapage et al. 2014**) and geosynthetic-encased stone columns (e.g., **Hosseinpour et al. 2015; Khabbazian et al. 2015; Yoo 2010**). Concrete and timber piles with and without pile caps and basal reinforcement have also been used to increase the construction rate and to improve load transfer from the soft soil to the stiffer piles (e.g., **Briançon and Simon 2012; Liu et al. 2007; Nunez et al. 2013; Rowe and Liu 2015; Zhang et al. 2013**).

GRCS embankments can be designed using close-form solutions that take advantage of soil arching and tensioned membrane load transfer mechanisms within the GRCS embankment system (e.g., **Hewlett and Randolph 1988; BS8006 2010; EBGEO 2011; Kempfert et al. 2004; Van Eekelen et al. 2011**). Advanced numerical models using the finite element method (FEM) and finite difference method (FDM) are becoming more common as a research tool to improve understanding of the behaviour of GRCS embankments (e.g., **Liu and Rowe 2015; Han et al. 2007**). The advantage of using a full-width numerical model of a GRCS embankment is that lateral deformations that vary across the width and depth of the embankment fill and foundation are predicted. Of course, the accuracy of numerical predictions will depend on mesh refinement and the complexity of the constitutive models used for the component materials and their interfaces. However, parametric analyses at the design stage using a full-width model can be very time consuming and may not adequately capture local load transfer mechanisms

particularly if a coarse numerical mesh is used in the simulations. A strategy to overcome this shortcoming is the unit cell approach used by many researchers (e.g., **Han and Gabr 2002; Smith and Filz 2007; Khabbazzian et al. 2015**). The location of an example unit cell in a GRCS embankment is shown in **Figure 1**. However, there are also limitations associated with typical unit cells including the use of fixed lateral boundaries (**Figure 2a**). Using both a unit cell and a full-width numerical model for a GRCS embankment, **Khabbazzian et al. (2015)** showed that the tensile loads in the geosynthetic reinforcement using the full-width model were much greater than those using the unit cell approach for the same structure. They attributed this discrepancy to lateral spreading of the embankment fill and foundation soil in the full-width model that was not captured by the unit cell. Regardless of which approach is used to model GRCS embankment performance, numerical results can also depend on how geometric nonlinearity of the soil and reinforcement is modelled using the small-strain and large-strain options in FDM program FLAC (**Itasca 2011**) or with and without mesh updating in FEM software programs.

The objectives of this paper are to demonstrate a new modelling technique using a modified unit cell approach to simulate the lateral spreading of the embankment fill and foundation soil, and to examine the influence of large-strain and small-strain model options in program FLAC on numerical outcomes (i.e., with and without mesh updating during calculation steps). Numerical results using (conventional) unit cells with lateral rigid boundaries and units cells with one or both vertical boundaries supported by horizontal linear elastic springs are presented and compared. The effect of lateral spring stiffness values on lateral spreading of the embankment fill and foundation soil, and reinforcement loads are demonstrated.

2 Small- and Large-Strain Mode in FLAC

Numerical analyses using FLAC models (**Itasca 2011**) can be executed in either large-strain mode (based on the Lagrangian formulation) or small-strain mode (based on the Eulerian formulation). For the Lagrangian formulation, the numerical grid coordinates at the end of each calculation step (or specified steps) are updated by adding the grid incremental displacements to grid coordinates before the next step. Hence, stresses and displacements at the current calculation step are calculated based on the updated grid representing the deformed material zones. However

for the Eulerian formulation, the grid is fixed to the original geometry and material zones. The calculation of stresses and displacements is based on the fixed grid even though the material zones move and deform during subsequent calculation steps. The reader is directed to the FLAC manual (Itasca 2011) for details regarding small- and large-strain options in the program.

3 Problem Definition and Parameter Values

Figure 1 shows a GRCS embankment where the soft foundation soil is improved by the cement-soil DM column walls. The numerical simulations carried out in this paper are for two-dimensional cases because of the plane-strain condition associated with **Figure 1**. However, the general approach presented in this paper can be extended to model three-dimensional GRCS embankment cases. This paper uses the example of GRCS embankments with 10-m thick soft foundation soil and 1- to 5-m thick embankment fills. Above the soft foundation soil is a working platform fill with a geosynthetic layer placed 0.3 m above the foundation soil surface. The spacing of 0.7-m thick column walls was 2.8 m (e.g., Forsman et al. 1999; Han et al. 2007; Huang and Han 2010). The column walls are founded on bedrock.

The location of an example unit cell in this study is shown in **Figure 1**. **Figure 2a** shows a unit cell (Case 1) with typical boundary conditions (e.g., fixed y -direction at bottom of the cell, and fixed x -displacement at both left and right sides of the cell). The width of the unit cell is 2.8 m with half of the DM column wall (i.e., 0.35-m thick) on both right and left sides of the unit cell. To allow for lateral spreading of the embankment fill and foundation soil, the Case 1 unit cell is first modified using a column of horizontal springs (with axial stiffness k) at the right side boundary of both the embankment fill and foundation soil (Case 2 - **Figures 2b**) and then both left and right side boundaries (Case 3 - **Figure 2c**). The spring stiffness values can be different on left and right sides of the unit cell and different for the embankment fill and soft foundation soil. These cases are also examined in the paper.

For a 5-m high embankment fill, the unit cell was modelled using 8400 zones and 112 cable elements for the geosynthetic reinforcement. A linear elastic model with a Mohr-Coulomb failure criterion was selected to model both embankment fill and soft foundation soil. More

advanced constitutive models for the soils and reinforcement were not used in this study in order to focus on the modified unit cell concept. Parameter values are summarized in **Table 1** and were selected based on ranges reported in the literature. The material properties for the working platform fill were taken to be the same as those for the embankment fill. A depth-dependent Young's modulus for the foundation soil was used as follows: $E_{fs,1} = 0.5$ MPa for foundation soil depth between 0 and 2 m, $E_{fs,2} = 1$ MPa between 2 and 4 m, $E_{fs,3} = 1.5$ MPa between 4 and 6 m, $E_{fs,4} = 2$ MPa between 6 and 8 m, and $E_{fs,5} = 2.5$ MPa between 8 and 10 m. A linear elastic model was used for the DM column walls.

The load transfer between the dissimilar materials within the unit cell was modelled using interfaces with the normal stiffness, shear stiffness, friction angle, dilation angle and cohesion values shown in **Table 2** together with a strength reduction factor of $R_i = 2/3$. The method to calculate the interface parameter values based on the surrounding soil properties is described by **Yu et al. (2015)**.

The lateral springs shown in **Figure 2b** and **2c** were modelled using special interfaces with only normal stiffness values (the shear stiffness values were set to zero) using the 'glued' condition in FLAC. Thus for these interfaces, friction angle, dilation angle, and cohesion values were not required. The concept of using horizontal springs to simulate the soil lateral response in this paper is similar to that used in soil-structure interaction design of supported excavation walls and analyses for laterally loaded piles (e.g., **Canadian Geotechnical Society 2006**).

The cable nodes for the geosynthetic reinforcement on the left and right side of the unit cell were defined using grid numbers and the remainder was defined using x - and y -coordinates. The geosynthetic reinforcement was assigned an axial stiffness of $J = 2000$ kN/m. This value falls within measured ranges reported by **Forsman et al. (1999)** and is similar to the value used in numerical models by **Han et al. (2007)** and **Huang and Han (2010)**. The cable elements have a cross-section area of $A_g = 0.002$ m² and perimeter of $P_g = 2$ m (with an out-of-plane width of 1 m). Thus the Young's modulus of each cable element is calculated as $E_g = J/A_g = 2000/0.002 = 1000$ MPa. For the cable grout properties, the shear stiffness is $K_{s,g} = P_g \times k_{s,pf} = 2 \times 10 = 20$

MN/m/m, the adhesion is $C_g = P_g \times c_{i,pf} = 2 \times 0.67 = 1.33$ kPa, and the friction angle is $\phi_g = \phi_{i,pf} = 25^\circ$. **Table 3** summarizes all properties related to the cable elements.

The modelling started by setting the initial ground stresses using $K_0 = 1 - \sin(\phi_{fs}) = 0.741$ for the soft foundation soil without the DM column walls (e.g., by assuming that the DM column walls have the same material properties as the soft foundation soil). To simulate the DM column walls, the true wall material properties were then applied and the model was solved to equilibrium. The influence of installing the DM column wall on the stress distribution within the soft foundation soil was not considered in this paper. The numerical grid displacements and velocities were then set to zero. Small-strain mode was used in all simulations prior to placement of the fill. Thereafter, during embankment construction, small-strain or large-strain mode was selected to investigate the influence of strain mode on numerical outcomes. To simulate the embankment construction, 0.2-m thick embankment fill lifts were activated in sequence and the model was solved to equilibrium for each embankment fill lift until reaching the final embankment height of 5 m. The activation of the second embankment fill lift was accompanied by adding the cable elements to the model. It should be noted that only idealized conditions for the foundation soil were examined in this study; hence the generation and dissipation of pore-water pressures that may occur in field cases was not considered.

4 Results

4.1 Unit Cell for Case 1

Figure 3a shows the calculated maximum vertical stresses at the foundation surface using a unit cell with fixed x -displacement at both right and left sides of the unit cell (Case 1; **Figure 2a**). The numerical results show that increasing the embankment fill height increased the maximum vertical stress at the foundation surface. For example, the maximum foundation surface vertical stress was about 14 kPa for 1-m high embankment fill and increased to about 22 kPa for 5-m high embankment fill when using the large-strain mode. The differences in calculated maximum vertical stresses at the foundation surface using both large- and small-strain modes are negligible. However, for the embankment fill higher than 0.5 m, the calculated maximum vertical stresses at

the foundation surface were much less than those calculated using only soil self-weight ($\gamma_{\text{eff}}h$) because of the soil arching effect. The greater vertical stress at the foundation surface with increasing embankment height resulted in larger vertical settlement as shown in **Figure 3b**. The maximum settlement at the foundation surface was about 44 mm at 1-m high embankment fill and increased to about 85 mm at 5-m high embankment fill when using large-strain mode. The differences in vertical settlements at the foundation surface using large- and small-strain modes were less than 3 mm for the Case 1 unit cell examined in this paper.

The load transfer from the embankment fill to the DM column walls and soft foundation soil was modelled using interfaces. **Figure 4a** shows the interface normal stresses acting on the DM column wall top and foundation surface at 5-m high embankment fill for the Case 1 unit cell (**Figure 2a**). The vertical stresses on the soft foundation surface (i.e., on the center part of the unit cell) were much lower than those at the top of the DM column walls (i.e., left and right sides of the unit cell) because the soil self-weight load above the soft foundation is transferred to the DM column walls due to the soil arching effect. For example, the vertical stresses at the centers of the DM column walls (i.e., left and right edges of the unit cell) were about 332 kPa and increased to about 388 kPa at the intersections between the DM column walls and foundation soil when using large-strain mode and are much greater than those computed based on soil self-weight alone (i.e., 100 kPa). The vertical stresses on the foundation surface were generally less than 24 kPa which are much lower than stresses due to soil self-weight. The choice of large- and small-strain mode had negligible influence on the vertical stresses at the foundation surface and the top of the DM column walls using the Case 1 unit cell.

The soil arching effect can also be appreciated from the vertical stresses at the center of the unit cell within the 5-m high embankment fill shown in **Figure 4b**. The calculated vertical stresses within the embankment fill were equal to those from soil self-weight over the first 2 m below the embankment surface. Thereafter, the calculated vertical stress plots depart from the soil self-weight line because of the stress redistribution within the embankment fill due to soil arching.

Figure 5 shows the reinforcement tensile loads for the 5-m high embankment fill and Case 1 unit cell (**Figure 2a**) using both small- and large-strain modes in FLAC. The use of large-strain mode

resulted in reinforcement tensile loads of about 8 kN/m at $x = 0$ and 2.8 m, maximum values of 11 kN/m at $x = 0.35$ and 2.45 m, and minimum value of 1.6 kN/m at $x = 1.4$ m. However, when using the small-strain mode the reinforcement tensile loads were much lower (e.g., the maximum reinforcement load was about 3 kN/m at $x = 0$ and 2.8 m for the small-strain mode versus 11 kN/m at $x = 0.35$ and 2.45 m noted above for the large-strain mode). The influence of choice of small- and large-strain mode in FLAC on the magnitude of reinforcement tensile loads is judged to be significant for the Case 1 unit cell conditions. The differences are because the reinforcement tensile loads using the large-strain mode (i.e., with mesh updating) were generated from both the horizontal and vertical differential displacements between the soil and the reinforcement, while those using small-strain mode (i.e., without mesh updating) were generated only from the horizontal differential displacements between the soil and the reinforcement.

4.2 Unit Cell for Case 2

The results presented thus far are for the Case 1 unit cell (**Figure 2a**) with typical boundary conditions which do not account for lateral spreading of embankment fill and soft foundation soil. This section examines the first of two modified unit cells proposed in this paper where the left side of the cell is fixed in x -direction and the right side of the unit cell is supported by a series of horizontal springs (Case 2 in **Figure 2b**). To simplify the numerical analysis, only linear elastic springs are considered.

Figure 6a shows the maximum vertical stresses at the foundation surface for the Case 2 unit cell with a spring stiffness value of $k = 1$ MPa/m using both large- and small-strain modes. For this example the spring stiffness values for the embankment fill and the soft foundation soil were the same. The modelling results show that the maximum vertical stresses at the foundation surface were higher than those for the Case 1 unit cell example in **Figure 3a** when the embankment fill is higher than 2 m and other conditions being equal. For example, the use of large-strain mode for Case 2 unit cell resulted in the maximum vertical stress at the foundation surface of about 14 kPa at 1-m high embankment fill (same as 14 kPa in **Figure 3a**), and about 32 kPa at 5-m high embankment fill (compared to 22 kPa in **Figure 3a**). The maximum difference between the large- and small-strain modes for the maximum vertical stresses at the foundation surface with

Case 2 unit cell was about 4 kPa at 5-m high embankment fill compared to less than 1 kPa with Case 1 unit cell. **Figure 6b** shows the maximum settlements at the foundation surface for Case 2 unit cell with a spring stiffness value of $k = 1$ MPa/m using both large- and small-strain modes. The results show that the use of horizontal springs at right side of the unit cell (**Figure 2b**) resulted in larger maximum settlements at the foundation surface compared to those in **Figure 3b** for embankment fills higher than 1 m and other conditions being equal. The maximum difference between the large- and small-strain modes for the vertical settlement at 5-m high embankment fill was larger for Case 2 unit cell (about 23 mm) than for Case 1 unit cell (about 3 mm).

The normal stresses on the DM column wall top and foundation surface for Case 2 unit cell with spring stiffness of $k = 1$ MPa/m are shown in **Figure 7a**. When using the large-strain mode, the normal stress for Case 2 unit cell was about 309 kPa at $x = 0$ (compared to 332 kPa for Case 1 unit cell in **Figure 4a**), about 373 kPa at $x = 0.35$ m (compared to 388 kPa in **Figure 4a**), less than 33 kPa between $x = 0.35$ and 2.45 m (compared to less than 24 kPa in **Figure 4a**), about 480 kPa at $x = 2.45$ m (compared to 388 kPa in **Figure 4a**), and about 164 kPa at $x = 2.8$ m (compared to 332 kPa for Case 1 unit cell in **Figure 4a**). **Figure 7b** indicates that lateral spreading of embankment fill and foundation soil can also influence the vertical stress distribution within the embankment fill. This can be seen by comparing **Figure 7b** for Case 2 unit cell with **Figure 4b** for Case 1 unit cell.

Figure 8 shows the reinforcement tensile loads for Case 2 unit cell with spring stiffness $k = 1$ MPa/m. The results show that lateral spreading of the embankment fill and foundation soil can have significant influence on the reinforcement loads. For example, the use of Case 2 unit cell with spring stiffness $k = 1$ MPa/m and large-strain mode resulted in the reinforcement load of about 31 kN/m at $x = 0$ (compared to 8 kN/m for Case 1 unit cell in **Figure 5**), about 40 kN/m at $x = 0.35$ m (compared to 11 kN/m in **Figure 5**), about 15 kN/m at $x = 1.4$ m (compared to 1.6 kN/m in **Figure 5**), about 38 kN/m at $x = 2.45$ m (compared to 11 kN/m in **Figure 5**), and about 17 kN/m at $x = 2.8$ m (compared to 8 kN/m in **Figure 5**). Using the small-strain mode, the maximum reinforcement load was 31 kN/m at $x = 2.45$ m in **Figure 8** compared to 3 kN/m at $x = 0$ and 2.8 m for Case 1 in **Figure 5**. It is the lateral spreading of embankment fill and foundation soil that occurs using the Case 2 unit cell that results in greater axial extension of the

reinforcement and thus larger reinforcement loads for the Case 2 unit cell compared to the Case 1 unit cell.

4.3 Influence of Spring Stiffness on Reinforcement Loads for Case 2

The choice of magnitude of the spring stiffness can affect the lateral displacements generated within the embankment fill and foundation soil. These displacements will also depend on where the Case 2 unit cell is taken along the full-width of the GRCS embankment (e.g., near the center line or toe of the embankment due to different fill depths). The influence of different spring stiffness values on the reinforcement loads with large-strain mode and 5-m high embankment fill is shown in **Figure 9** (both the embankment fill and the soft foundation soil were assumed to have the same spring stiffness in this example). The numerical results show that the reinforcement loads decreased with increasing soil spring stiffness because of less lateral spreading of embankment fill and foundation soil associated with higher spring stiffness. For example, the maximum reinforcement load for Case 2 unit cell was about 40 kN/m when using spring stiffness $k = 1$ MPa/m, but decreased to about 19, 11 and 11 kN/m when increased to $k = 10$ MPa, 100 MPa and infinite stiffness, respectively.

The spring stiffness of the soft foundation soil can be different from that of the embankment fill. The influence of different foundation soil spring stiffness values on reinforcement loads below a 5-m high embankment fill using large-strain mode is shown in **Figure 10**. The results show that increasing the spring stiffness value of the foundation soil reduced the reinforcement loads because less lateral spreading of the foundation soil resulted in less differential displacement between the reinforcement and platform fill. For example, the maximum reinforcement load was about 44 kN/m for the foundation spring stiffness of $k_f = 0.1$ MPa/m and decreased to about 40 and 35 kN/m for $k_f = 1$ and 10 MPa/m, respectively.

4.4 Unit Cell for Case 3

In this section, the lateral spreading of the embankment fill and foundation soil on both sides of the cell is considered using springs on both sides of the unit cell (Case 3 unit cell in **Figure 2c**).

Figure 11 shows the maximum vertical stresses and settlements at the foundation surface for Case 3 unit cell with spring stiffness values of $k_l = k_r = 1$ MPa/m using both small- and large-strain modes. The results show that the use of lateral springs on both sides of the unit cell resulted in maximum vertical stresses of 42 and 36 kPa (compared to 36 and 32 kPa for Case 2 unit cell) and maximum settlements of 192 and 151 mm (compared to 149 and 126 mm for Case 2 unit cell) for the 5-m high embankment fill simulations using the small- and large-strain modes, respectively.

Figure 12 shows the interface normal stresses at the foundation surface and vertical stresses at the center of the unit cell for the 5-m high embankment fill and Case 3 unit cell with spring stiffness values of $k_l = k_r = 1$ MPa/m. The maximum interface normal stress was about 462 and 538 kPa at the intersection between the DM column wall and soft foundation soil for simulations in small- and large-strain mode, respectively (**Figure 12a**). These values are greater than those for Case 2 unit cell (i.e., 421 and 480 kPa in **Figure 7a** for the small- and large-strain modes, respectively). The differences in vertical stresses at the center of the unit cell between Case 3 and Case 2 unit cells can also be observed by comparing results from **Figure 12b** with those from **Figure 7b**.

Figure 13 shows the reinforcement loads for Case 3 unit cell at 5-m high embankment fill using both small- and large-strain modes and spring stiffness values of $k_l = k_r = 1$ MPa/m. The maximum reinforcement loads for Case 3 unit cell were about 34 and 44 kN/m using small- and large-strain modes, respectively. The maximum difference in reinforcement load using large- and small-strain modes was about 13 kN/m and occurred at the center of the unit cell.

4.5 Influence of Spring Stiffness on Reinforcement Loads for Case 3

The influence of different spring stiffness values at left side and right of the unit cell (Case 3) on the reinforcement loads is shown in **Figure 14** using large-strain mode. In this example the embankment fill is 5 m high and the spring stiffness at the right side of the unit cell is fixed at $k_r = 1$ MPa/m. Increasing the spring stiffness at left side of the unit cell increased the reinforcement load at $x = 0$ but had negligible effect on the reinforcement load at $x = 2.8$ m. Using spring

stiffness greater than $k_1 = 10$ MPa/m at left side of the unit cell had minor influence on the reinforcement loads between $x = 0.35$ and 2.8 m.

5 Conclusions

An effective technique to improve the performance of embankments over soft foundations is to use geosynthetic-reinforced and column-supported (GRCS) embankments. Both full-width model and unit cell approaches have been used by researchers to study the behaviour of these structures. However, the typical unit cell with fixed horizontal displacements at side boundaries is unable to capture the lateral spreading of the embankment fill and foundation soil. **Khabbazzian et al. (2015)** showed that this can lead to large differences in numerical outcomes between a full-width model and the simulation of the same structures using the typical unit cell approach. This paper presents a modified unit cell approach to simulate the lateral spreading of the embankment fill and foundation soil using horizontal linear elastic springs at the vertical boundaries of the unit cell. Program FLAC is used to examine the influence of large- and small-strain mode on conventional and modified unit cell behaviour. Based on the numerical results using the three different unit cells examined in this paper, the following conclusions can be summarized:

- For a typical unit cell with fixed lateral horizontal displacements at the side boundaries (Case 1 unit cell in **Figure 2a**), the use of large- and small-strain mode has minor influence on the maximum vertical stresses and settlements, and interface normal stresses at the foundation surface. However, when the numerical model was run in small-strain mode, the calculated reinforcement loads were much lower compared to large-strain mode.
- For the modified unit cells with lateral springs (Case 2 unit cell in **Figure 2b** and Case 3 unit cell in **Figure 2c**), the lateral spreading of the embankment fill and foundation soil increased the maximum vertical stresses and settlements at the foundation surface, and the reinforcement loads when compared to numerical results using fixed side boundaries (Case 1 unit cell) and other conditions being equal.

- The magnitude of calculated reinforcement loads for Case 2 and 3 unit cell examples depended on the choice of lateral spring stiffness value and whether or not the same stiffness value was assigned to the embankment fill and foundation soil.

This paper has demonstrated that lateral spreading of the embankment fill and foundation soil can be modelled using a modified unit cell with a series of lateral springs supporting one or both side boundaries. However, numerical outcomes depended on the choice of small-strain or large-strain mode when using program FLAC. The computed reinforcement loads were particularly sensitive to this choice.

The influence of equivalent large-strain mode in FEM programs (i.e., mesh updating option) was not investigated in this study. However, similar sensitivity to choice of fixed or updated mesh generation on reinforcement loads using these codes is expected.

Acknowledgement

The work reported in this paper was supported by grants from the Natural Sciences and Engineering Research Council of Canada (NSERC).

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Table 1. Material property values in FLAC.

Properties	Value
Embankment and working platform fill	
Unit weight, γ_{ef} (kN/m ³)	20
Young's modulus, E_{ef} (MPa)	40
Poisson's ratio, ν_{ef} (-)	0.3
Friction angle, ϕ_{ef} (degree)	35
Dilation angle, ψ_{ef} (degree)	5
Cohesion, c_{ef} (kPa)	1
Soft foundation soil	
Unit weight, γ_{fs} (kN/m ³)	16
Young's modulus of soft clay 1, $E_{fs,1}$ (MPa)	0.5
Young's modulus of soft clay 2, $E_{fs,2}$ (MPa)	1.0
Young's modulus of soft clay 3, $E_{fs,3}$ (MPa)	1.5
Young's modulus of soft clay 4, $E_{fs,4}$ (MPa)	2.0
Young's modulus of soft clay 5, $E_{fs,5}$ (MPa)	2.5
Poisson's ratio, ν_{fs} (-)	0.25
Friction angle, ϕ_{fs} (degree)	15
Dilation angle, ψ_{fs} (degree)	0
Cohesion, c_{fs} (kPa)	5
DM column	
Unit weight, γ_c (kN/m ³)	20
Young's modulus, E_c (MPa)	150
Poisson's ratio, ν_c (-)	0.3

Table 2. Interfaces and corresponding parameter values in FLAC.

Interface and parameters	Value
Soft clay-DM column and soft clay-platform fill	
Friction angle, $\phi_{i,fs}$ (degree)	10
Dilation angle, $\psi_{i,fs}$ (degree)	0
Adhesion, $c_{i,fs}$ (kPa)	3.3
Normal stiffness, $k_{n,fs}$ (MPa/m)	10
Shear stiffness, $k_{s,fs}$ (MPa/m)	1
Platform fill-DM column	
Friction angle, $\phi_{i,pf}$ (degree)	25
Dilation angle, $\psi_{i,pf}$ (degree)	0
Cohesion, $c_{i,pf}$ (kPa)	0.67
Normal stiffness, $k_{n,pf}$ (MPa/m)	100
Shear stiffness, $k_{s,pf}$ (MPa/m)	10

Table 3. Parameter values of cable elements in FLAC.

Cable element parameters ^a	Value
Young's modulus, E_g (MPa)	1000
Cross-sectional area, A_g (m ²)	2×10^{-3}
Exposed perimeter, P_g (m)	2
Grout stiffness, $K_{s,g}$ (MN/m/m)	20
Grout cohesion, C_g (kN/m)	1.33
Grout frictional resistance, ϕ_g (degree)	25

Figure 1. Schematic showing a full-width model of the GRCS embankment (with DM column walls).

Figure 2. Unit cell (with half of DM column wall on each side of the cell) with: (a) fixed x -displacement boundary condition on both left and right sides of the cell (Case 1), (b) fixed x -displacement boundary condition on left side and springs on right side of the cell (Case 2), and (c) springs on both left and right sides of the cell (Case 3).

Figure 3. Influence of large- and small-strain mode for Case 1 unit cell on (a) the maximum vertical stresses at the foundation surface and (b) the maximum settlements at the foundation surface.

Figure 4. Influence of large- and small-strain mode for Case 1 unit cell with 5-m high embankment fill on (a) the interface normal stresses at the foundation surface elevation, and (b) the vertical stress at the center of the unit cell.

Figure 5. Influence of large- and small-strain mode for Case 1 unit cell on the reinforcement tensile loads for 5-m high embankment fill.

Figure 6. Influence of large- and small-strain mode for Case 2 unit cell with spring stiffness $k = 1 \text{ MPa/m}$ on (a) the maximum vertical stresses at the foundation surface, and (b) the maximum settlements at the foundation surface.

Figure 7. Influence of large- and small-strain mode for Case 2 unit cell with spring stiffness $k = 1 \text{ MPa/m}$ on (a) the interface normal stresses at the foundation surface elevation and 5-m high embankment fill, and (b) the vertical stress at the center of unit cell within the embankment fill.

Figure 8. Influence of large- and small-strain mode for Case 2 unit cell with spring stiffness $k = 1 \text{ MPa/m}$ on the reinforcement tensile loads for 5-m high embankment fill.

Figure 9. Influence of soil spring stiffness values for Case 2 unit cell on the reinforcement tensile loads with large-strain mode at 5-m high embankment fill using the same spring stiffness values for the embankment fill and foundation soil.

Figure 10. Influence of spring stiffness values of the foundation soil for Case 2 unit cell on the reinforcement tensile loads with large-strain mode at 5-m high embankment fill.

Figure 11. Influence of large- and small-strain mode for Case 3 unit cell with spring stiffness values $k_l = k_r = 1$ MPa/m on (a) the maximum vertical stresses at the foundation surface, and (b) the maximum settlements at the foundation surface.

Figure 12. Influence of large- and small-strain mode for Case 3 unit cell with spring stiffness values $k_l = k_r = 1$ MPa/m on (a) the interface normal stresses at the foundation surface elevation and 5-m high embankment fill, and (b) the vertical stress at the center of unit cell within the embankment fill.

Figure 13. Influence of large- and small-strain mode for Case 3 unit cell with spring stiffness values $k_l = k_r = 1$ MPa/m on the reinforcement tensile loads for 5-m high embankment fill.

Figure 14. Influence of spring stiffness values at left side of Case 3 unit cell on the reinforcement tensile loads with large-strain mode at 5-m high embankment fill using the same spring stiffness values for the embankment fill and foundation soil.