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Numerical Modelling of Deformation in Preloaded Embankment over Weak Soil

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Abstract: This paper presents a numerical simulation of the deformations of an instrumented test pre-loaded embankment constructed over the soft soil underlying the Colombo-Katunayaka Expressway located in the Western Province of Sri Lanka. Finite Element Analysis is performed to simulate the preloaded embankment and sub-soils by using the Geo-Slope software, and by utilising the Modified Cam Clay (MCC) model and the linear elastic models available with the software. The loadings increments and other field operations are simulated in the numerical analysis identical to the processes in the field. Initial stresses and pore water pressure are imposed identical to the in-situ conditions, and a coupled consolidation analysis is subsequently performed by using the software to predict the settlement behaviours. The predicted settlements and lateral displacement profiles are compared with the observed field data.

The numerical results show that the settlement depends on the selection of the soil constitutive model, modulus value of the fill, and permeability of the soil. Also, the analysis indicates that the MCC model can be used for very soft soils. The displacement and stress variation pattern are identified in and around the embankment. It can be recommended that the proposed analysis method is a useful tool for engineering practice in soft soil environments of Sri Lankan and in cases of highway construction and development using the pre-loading method to strengthen soft soils.

Keywords: Preloading, Soft Soil, Finite Element Simulation and Analysis, Settlement

1. Introduction

Some major proposed infrastructure developments are planned over weak sub-soil areas in the South Asian region. The Colombo Katunayaka Expressway (CKE) is one such project, passing over weak peaty soils. In this project, test embankments were constructed with a series of stage loading planned to ensure that no failures occur. Different types of geotextiles were used to control the uneven settlement and separation of layers.

When heavy embankments are placed on soft soils, loading will tend to squeeze the water out of the soil in a vertically upward direction to a drainage blanket (sand mat) placed at the bottom of the embankment, and possibly downward to a natural drainage layer that might exist below the soft layer of low permeability that is being squeezed. The prediction by analytical means of the behaviour of the soft peaty soil in a large area is difficult due to the complex geometries and non-linear material behaviours involved. The development of finite element methods in the geo-technical engineering field has enabled engineers to handle these problems. In this work, the FEM software "Geo-slope" package is used (Geo-Slope, 2000) to simulate the long-term consolidation under different material models. The actual stage-loading sequence implemented in the field is simulated in the FE analysis here. The sub-soil here consists mainly of peat and peat mixed with very loose sand, underlain by the loose to dense sandy soil. The characteristics of peat behaviour are obtained and the relevant parameters under various conditions are identified from previous work (Ketheeswaravenayagan 2006, Kugan 2004, RDA 1999). Prediction of the lateral ground movement and the behaviour of excess pore water pressure are quite difficult unless the precise relevant material parameters are available.

Typical stress-strain behaviour of soil is not linear elastic, and depends on many factors like

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the sequence of loading and the stress history of the soil. Thus no mathematical model can completely describe the complex behaviour of real soils under all conditions (Balasubramaniam et al., 1992). Among the many attempts to formulate common features of soil based on its yield locus and plastic potentials, one successful model is the "Cam Clay" model developed by Roscoe et al. (1968). Instead of depending on a large number of empirical constants, Cam Clay model has only a few well-known soil parameters.

2. Numerical analysis

In any consolidation analysis, it is important to start with a reasonable minimum time step. The software used in this work gives guidelines for estimating the minimum time step as

$$\Delta t = \frac{\gamma_w M_v}{4 k_{(sat)}} * Element \quad Area$$

where,

γ_w - Unit weight of water

M_v - Coefficient of Volume compressibility

$k_{(sat)}$ - Saturated permeability

In the software segment Sigma/W, the embankment loading is simulated by adding elements to the Finite Element (FE) mesh. These implied loads are automatically calculated by the program. In the software segments Sigma/W and Seep/W, the characteristics of each finite element in a mesh are identified by a material number, which is defined by the soil model and respective parameters. The boundary conditions can be defined at nodes or along element edges (Geo Slope, 2003).

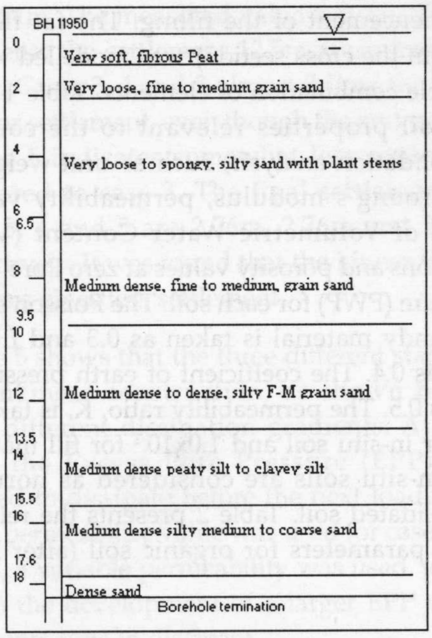


Figure 1: Profile of subsurface conditions across 11950 location

Kugan (2004) tested peaty soil underlying the CKE project. The axial stress versus strain plot for the Unconsolidated Undrained (UU) Tri-axial test done by Kugan (2004) was used to derive the Young's modulus for peat material.

The preloading segment of 11950 from the Nadurupittiya trial area was considered for analysis and simulation. The subsurface stratigraphy of this area is shown in Figure 1. The visual classification of the 2nd and 3rd layer of the strata shows that it is sand in a suspension stage, like a slurry type fluid (Ketheeswaravenayagan, 2006).

The loading and construction were simulated over 342 days but field monitoring had been started after a certain number of days from the

Table 01: Summary of Linear Elastic and other relevant parameters for the coupled consolidation analysis of 11950 PL segment (Bowels, 1996., and Poulos, 1975).

Soil Material	Unit weight of soil (kN/m³)	Young Modulus (kN/m²)	Permeability, k_x (m/s)	Volumetric water content function. (Mv's are in m²/kN)
Fibrous Peat	11	403	$8.0x10^{-7}$ (I)	$Mv=1.0x10^{-3}$, $n_o=0.68$
Fine to medium sand	16.7	708	$5.5x10^{-7}$	$Mv=5.5x10^{-3}$, $n_o=0.84$
Silty sand with plant debris	14	708	$8.0x10^{-7}$ (I)	$Mv=2.5x10^{-3}$, $n_o=0.87$
Fine to medium sand	16.7	8500	$5.5x10^{-7}$	$Mv=8.0x10^{-5}$, $n_o=0.35$
Silty F-M grain sand	16.7	13000	$5.0x10^{-7}$	$Mv=6.0x10^{-5}$, $n_o=0.3$
Peaty silt to Clayey silt	16.7	9750	$5.0x10^{-7}$	$Mv=8.0x10^{-5}$, $n_o=0.35$
Silty medium to coarse sand	17.7	20000	$1.0x10^{-7}$	$Mv=5.0x10^{-5}$, $n_o=0.25$
Dense sand	17.7	40000	$1.0x10^{-7}$	$Mv=1.0x10^{-5}$, $n_o=0.2$
Fill material	17.4	25000	$5.0x10^{-7}$	$Mv=3.0x10^{-5}$, $n_o=0.3$

commencement of the filling. The far field at 65m (in the cross section) was modelled with a suitable combination of elements. Table 1 gives the soil properties relevant to the coupled consolidation analysis, such as unit weight of soil, Young's modulus, permeability values, slope of Volumetric Water Content (VWC) functions and porosity values at zero Pore Water Pressure (PWP) for each soil. The Poisson's ratio for sandy material is taken as 0.3 and fibrous peat as 0.4. The coefficient of earth pressure at rest is 0.5. The permeability ratio, K, is taken as 0.5 for in-situ soil and $1.0x10^{-2}$ for fill material. The in-situ soils are considered as normally consolidated soil. Table 2 presents the relevant MCC parameters for organic soil (after RDA, 1999).

Table 2: Relevant Modified Cam Clay parameters for organic soil (case 2) (after, RDA, 1999).

Parameters	Values used for fibrous peat	Values used for very loose peaty soil
λ	1.505	0.203
κ	0.096	0.011
Γ	9.411	1.711
M	1.2	1.2
ν	0.4	0.3

The settlement plates that were used as monitoring instruments in the test segment were represented by selected nodes of the mesh. The settlement plate at the centre had been installed 6 days after the start of work in the field. The fill load, applied in stages, was exactly simulated in the analysis. In this segment, the planned embankment height was 4.32m but the fill height went up to 5.69m including a surcharge fill height of 0.94m. For the numerical analysis, a maximum of 150 iterations and a minimum of 15 were chosen. By using different properties for soft soil material, several sensitivity analyses were carried out and the profile that best matched with the field behaviour was obtained. The FE mesh is shown in Figure 2.

2D plane strain FE analysis was carried out under the different situations. In each case the simulated subsurface profile and fill sequence were identical with the field. The analyses were performed under the following cases:

Case 1: The very soft to very loose soils were considered as linear elastic. The Young's modulus of peat and very

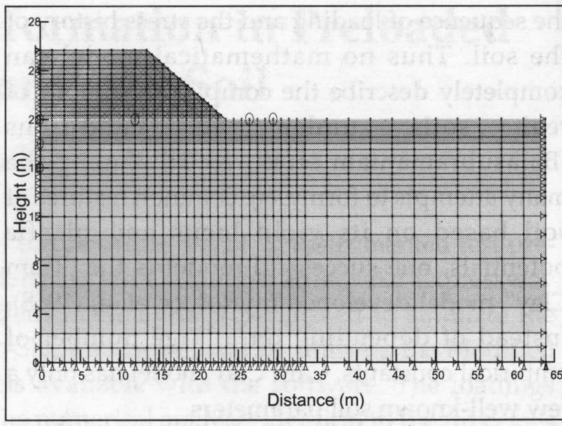


Figure 2: Finite element mesh for 11950 section

loose sand was taken as 310kPa, and that of very loose sand with plant debris was taken as 350kPa. The other relevant parameters are given in Table 1.

Case 2: The very soft to very loose soils were taken into account as being normally consolidated soils that obey stress-strain behaviour of MCC. The respective MCC parameters are shown in Table 2.

Case 3: This analysis case is the same in Case 2 but its permeability was varied during the analysis as $8.0x10^{-7}$ to $5.0x10^{-7}$ m/s for peat and $8.0x10^{-7}$ to $1.0x10^{-7}$ m/s for very loose sand.

Case 4: One geotextile (Poly Propylene (PP) mat with Young's Modulus value of 830kPa) was used for the embankment construction on the ground, while other conditions were the same as in case 3.

Case 5: Young's Modulas for the fill was selected as 50,000kPa while other conditions are the same as in case 3.

In Case 4, the geotextile was modelled using the plane strain bar elements available in the software (which was achieved by specifying a zero value for Moment of Inertia in the corresponding beam element). Young's moduli values presented in Table 3 were derived from the axial stress versus strain plots for the UU tests done by Kugan (2004). For the cell pressure of 75kPa, Young modulus was 403kPa and for a cell pressure of 100kPa it was 450kPa.

Table 3: Summary of Young's modulus from UU test (after Kugan, 2004)

Unconsolidated	Cell pressure (kPa)		
Undrained test (UU)	**50**	**75**	**100**
Middle sample (UU-2)	539.2	403.0	450.4
Bottom sample (UU-3)	329.8	310.9	462.2

Note: The Top sample results are unreliable

3. Numerical predictions and comparisons

Using the plain strain model, a preloaded test embankment segment was analysed under different situation for comparisons. The numerical results were compared with available field data, and it was seen that the comparison was good.

Results of the analyses for the five different situations (cases) are presented in Figure 3. Cases 3, 4 and 5 follow almost the same path for consolidation settlement, even though at the end, case 5 shows a slightly less settlement compared to case 3. There is no significant difference between settlement graphs for case 3 and case 4 (Figure 3); this indicates that the geotextile does not influence the settlements significantly, and thus the stresses in the geotextile are not discussed here. For cases 2, 3 and 4 the final settlement is 2.98m, but for case 5 it is 2.94m. The measured value is 3.0m. Figure 4 presents the settlement 12.5m away from the centre. Cases 3, 4 and 5 almost follow the same path for settlement, even though the end portion of case 5 indicates somewhat less settlement compared to case 3. The final settlement for cases 3, 4 and 5 are 2.76m, 2.76m and 2.71m respectively. It was found that the Linear Elastic Analysis gives less settlement.

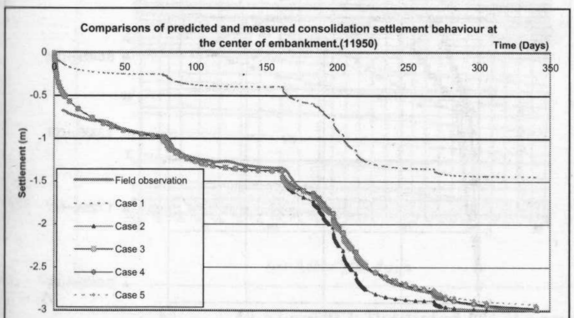


Figure 3: Comparison of the consolidation settlement at 11950 at centre of the embankment.

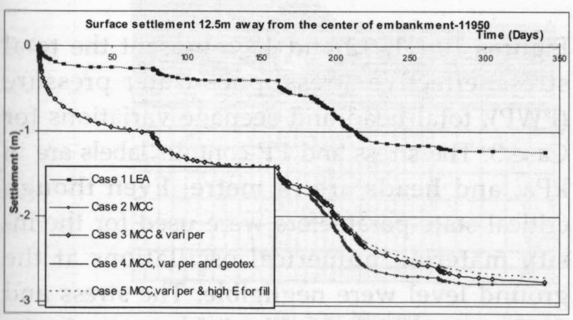


Figure 4: Predicted settlement at 11950 at 12.5m away from the centre of embankment.

Figure 5 shows that the three different stages of embankment construction, as shown by the three different dissipation gradients. At each stage the Excess Pore Pressure (EPP) was allowed to dissipate before the next loading. A lesser permeability value was used for case 2. In case 3, a variable permeability was used, giving rise to the development of a larger EPP which took more time to dissipate.

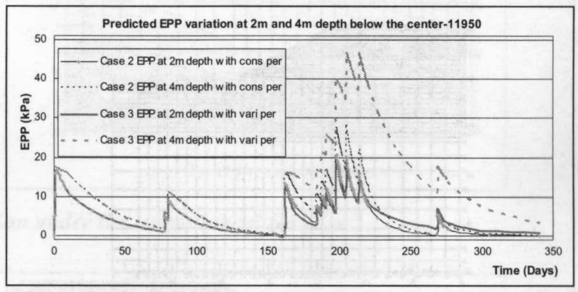


Figure 5: Predicted EPP at 2.0m and 4.0m depth at the centre of embankment

Figure 6 presents the X-Y (on the cross-section) deformation at 3m and 6m away from the embankment toe. It can be seen that the XY deformation plots under Linear Elastic Analysis show less deformation than the MCC analysis at the end portion, but they are approximately in the same order at the initial stage. Also, Figure 6 compares case 3 and case 5; it shows that parametric changes of Young moduli influences the deformations.

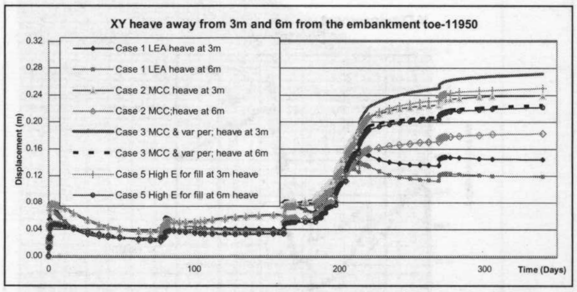
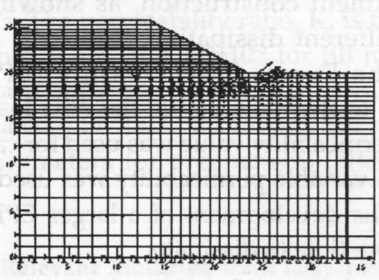
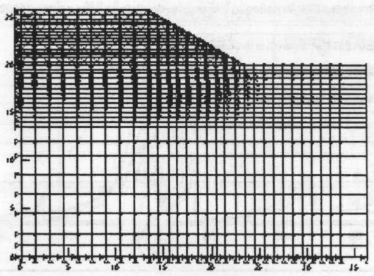


Figure 6: XY (resultant) deformation patterns away from embankment toe.

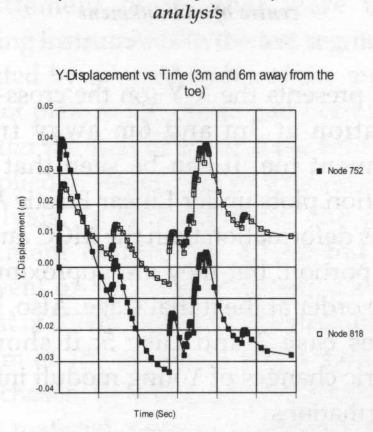
Figure 07(a) shows greater displacement close to the toe area (also see Figure 08), though the soft soil thickness is less. Thereafter the displacement vector is large under the embankment along the Y direction (see Figure 07(b)). Figure 07(c) and 07(d) show that initially the heave is high at 3m as compared to that at 6m. Also, the Y-heave is larger during the 2nd and 3rd stages. When variable permeability is used, the amounts of heave at both locations are large. Node 752 and 818 are 3m and 6m respectively from the toe.



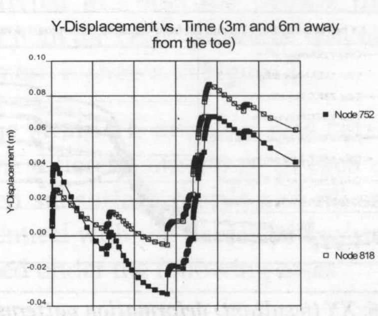
(a) Deformation pattern shown by Vectors for case 3 after 1 day



(b) Deformation pattern shown by vectors for case 3 at the end of analysis



(c) Predicted heave at 3.0m and 6.0m away from the toe for case 2



(d) Predicted heave at 3.0m and 6.0m away from the toe for case 3

Figure 7: Deformation patterns and heaving

In Figure 8 and 9 the legend shows the time in seconds; here surface settlement profile of the ground surface and displacement curves are plotted for number of days of 1, 154, 200 and at the end of analysis. In Figure 8, initially more settlement is observed at the toe area; thereafter settlements increase in the centre region. Here the distance is shown from the centre of the embankment. The Y coordinate of 20m is located at the ground level.

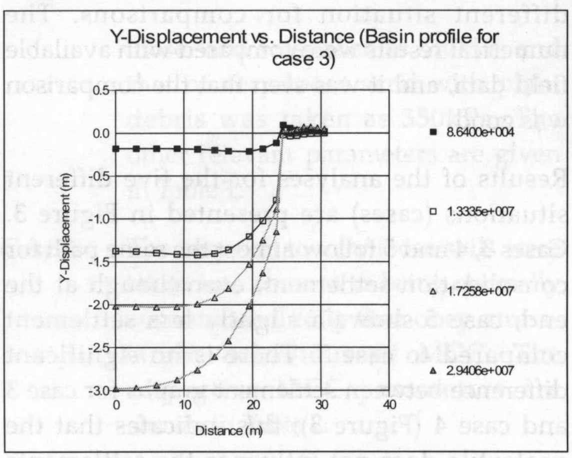


Figure 8: Basin analysis (settlement of the embankment) for case 3)

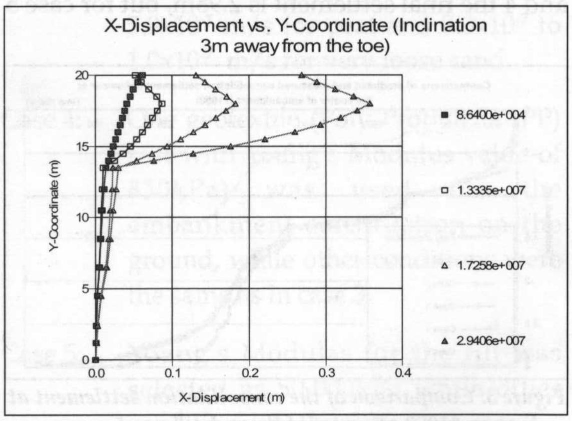


Figure 9: Prediction of horizontal displacements 3.0m away from the toe for case 3

Figures 10, 11, 12 and 13 represent the total stress, effective stress, pore water pressure (PWP), total head and seepage variations for Case 3. The stress and PP contour labels are in kPa, and heads are in metre. Even though critical state parameters were used for the in-situ material, numerical oscillations at the ground level were negligible. The stress and PWP variations during the field process in the in-situ soil and the fill are shown by the figures.

The dissipation paths of pore water are represented by vectors. Figure 13 shows that there is a vertically upward flow at the ground surface, while horizontal flow takes place in the in-situ soil.

4. Discussion and Comments

4.1 Parameter evaluation for soft peaty soil

For the soft soil material, the evaluation of elastic parameters using the conventional testing equipment produce significant errors due to difficulties in collection and preparation of small samples (Kugan et.al., 2003). Evaluation of parameters from the top region of a large scale sample tested by Kugan (2003) produces unrealistic values. One reason may be that the dissipated water, which moves to the top surface of the sand layer, descends back into the top layer due to gravity when the soil is unloaded at the end. Another reason may be that the application of geotextile between the peaty soil and sand layer develops a siltation effect (filter cake) which obstructs the flow of water.

In order to account for the sensitivity of the various parameters used for the modelling, five different cases were investigated in all.

4.2 Concluding remarks for preloading section

MCC model is normally applicable for clayey soil even though it was used for this segment containing very loose to spongy peaty soil. Volumetrically major part of this soil consists of water and decayed plant stems, and it seems that specific gravity of sand and specific gravity of the mineral part of this material (slurry) are almost the same. According to BS 1377 this soil can be classified as a sand material (considering the mineral part). It is necessary to further investigate this soil by physical and chemical means. Also, the visual soil classification method is suggested to be modified to reflect Sri Lankan organic soils (with volumetric measures), by giving more emphasis to the water content for organic soils. The soft ground improvement designer must emphasise the water content of the soft soil in the soil classification.

Figure 05 shows that the development and dissipation of excess PWP depend on the depth of the monitoring piezometer and permeability of the material. The application of variable permeability predicts higher excess PWP. The toe region is not always in positive heave, (see Figure 7(c) and (d)); under load of the embankment, the original ground surface under the embankment as well as the ground surface beyond the toe of embankment deform (settle). In the first stage more Y-displacement (heave) is predicted at 3m than at 6m from the toe. Figure 9 shows that beyond the toe, large deformations take place at shallow depth below the surface, indicating a shear-type failure. The overall settlement and displacement show that the radius of the possible failure surface increases with time, and the centre of failure circle-radius moves away from the centre of embankment with loading. Initially peak settlement occurs at the toe area, and then it shifts to the center, as shown in Figure 8. It can summarised that initially peak horizontal displacement occurs at the surface of the toe area then the most probable failure line passes 2m below original ground surface and it is 3m away from the toe while loading (see Figure 9).

Total stress increment, PP development and dissipation, seepage, and the effective stress with time and loading are illustrated in Figures 10 to 13. The soil improvement takes place gradually in the subsoil while pore water dissipation occurs. Thus, the stress under embankment increases when the dissipation of pore water takes place. Except the topsoil, more water seeps in the horizontal direction as can be seen by the total head contours (Figure 13). It can be seen that contour plots are not smooth at the ground level, since the critical state model develops numerical oscillations, but it is negligible. It can be summarised that more stresses are concentrated at the centre region of embankment after the dissipation of EPP.

5. Conclusions

Following conclusions can be reached:

Coupled analysis by using the MCC under 2D plane strain conditions yielded numerical results for settlement under the embankment which were in good agreement with observed data.

The analysis shows that the stiffness of the fill represented by its Young's modulus influences the settlement under the embankment but the application of geotextiles does not significantly influence the settlement.

The numerically predicted settlement depends on the soil characteristics, selection of soil model, Young's modulus of the fill and permeability of the soil.

The numerical results show that MCC model can be used for modeling of very loose to spongy soils.

It is recommended that the proposed method is useful procedure for engineering practice in peaty clay environments encountered in a highway construction and development works in Sri Lanka.

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