ISSN 2229-5518

Modelling Consolidation Behavior of Embankment Using Plaxis

PK Aseeja, National Institute of Technology Calicut

Abstract— Analyzing the behavior of the soft ground under embankments is a challenging task and this paper revisits a well known case study of an embankment of Haarajoki test embankment. The behavior of the ground considering the Soft Soil model including and excluding soil creep is simulated using finite element software PLAXIS. The analyzed data are verified with field measurements and the results reveal that the influence creep should be considered while analyzing the consolidation behavior of normally consolidated clays. A parametric study is conducted to compare the effectiveness of three ground improvement options using creep model. The effect of preloading, stone columns and prefabricated vertical drain are compared in terms of settlement, lateral deformation and excess pore pressure. From the results, it is observed that PVD offer the fastest rate of consolidation and minimum pore pressure generated. The stone columns reduce the settlement considerably due to the high stiffness while preloading requires more time to achieve the same.

Index Terms — embankment, pore water pressure, preloading, settlelment, soil creep, stone column, vertical drain

1 INTRODUCTION

Due to the rapid increase in population in many countries, the construction activities have become concentrated in low-lying marshy areas and reclaimed lands, which are comprised of highly compressible weak organic and peaty soils of varying thickness. The construction of embankments for various purposes on these soft soils with a high groundwater level often leads to large lateral pressures and movement, excessive settlements as well as slope and bearing failures, which usually result in long construction delays and costly remedial works. Any site with soft soils needs to be analysed, so that the mitigating strategies/techniques to avoid problems associated with soft soil can be best identified and implemented. Consolidation and creep are significant in soft soils and hence, their long term deformation must be taken into account in engineering design and practice. Soil loading history is important as it clearly shows whether the soil is normally consolidated or over-consolidated. A soil is considered normally consolidated if the current stress in the soil is the maximum it has ever experienced, and over-consolidatedif it has been subjected to a larger stress than the current stress.

In the present research work, the behavior of embankement on soft soil is analysed using soft soil model including and excluding creep. A parametric study is done using three ground improvement options and results are compared.

2. INPUT PARAMETERS

Basic geometry and model parameters are similar for all the ground improvement options and are taken from the case study of Haarajoki Test Embankment**.** Haarajoki in the Southern Finland is the location of an instrumented embankment forming a noise barrier, which was the subject of an international competition organized by the Finnish National Road Administration (FinnRA 1997).

2.1 GEOMETRY AND GROUND CONDITIONS

The Haarajoki test embankment is 3 m high and 100 m long, 8 m wide, and the slopes have a gradient of 1:2. The embankment itself was constructed in 0.5 m thick layers and each layer was applied and compacted within 2 days. The embankment is founded on a 2 m thick dry crust layer overlying a 22.2 m thick soft clay deposit. The layers below the soft clay consists of silt and till materials can be considered as permeable.

The groundwater table is at the ground surface. The subsoil is divided into seven sub-layers with different compressibility parameters and overconsolidation ratios. The water content of the soft clay layer varies between 75 and 112% depending on the depth, and is almost the same as, or greater than, the liquid limit.

2.2 MATERIAL PARAMETERS

The soft clay deposit was modeled as lightly overconsolidated soft clay and the values of soil parameters for modeling were taken from the case studies of Neher et.al (2003), Yildiz et.al (2009), and Karstunen et.al (2012) and are given in Table 1.

The embankment, which is made of granular fill, can be modeled with a simple Mohr–Coulomb model assuming the following material parameters: $E' = 40{,}000$ kN/m^2, $\nu' = 0.35$, $\Phi' = 40°$, $\Psi' = 0°$, $c' = 2$ kN/m^2 and $\gamma = 21$ kN/m^3 (where E′ = Young's modulus; ν′=Poisson's ratio; Φ′= friction angle; Ψ′= dilatancy angle; and γ = unit weight of the embankment material). The problem is dominated by the soft clay response and is hence rather insensitive to the embankment parameters.

IJSER © 2016
http://www.ijser.org

ISSN 2229-5518

Table 1 Values for the conventional soil parameters

Layer	1	2	3	4	5	6	7
Depth(m)	0-2	2-6	6-7	7-12	12-15	15-18	18-22
γ (kN/m^3)	17.5	14.3	14.3	15.1	15.1	15.7	17.5
k_x (m/d) (10^{04})	3.46	1.04	0.864	0.864	0.864	0.864	3.46
k_y (m/d) (10^{-05})	17.3	5.18	4.32	4.32	4.32	4.32	1.73
POP (kN/m^2)	110	32	32	32	32	32	32
e_o	1.25	2.9	2.6	2.35	2.2	2.0	1.25
K_o	1.0	0.7	0.7	0.7	0.7	0.5	0.45
Φ'	36.9°	28.8°	27.7°	27°	28.8°	36.9°	36.9°
ν'	0.2	0.2	0.2	0.2	0.2	0.2	0.2
M	1.6	1.15	1.43	1.15	1.20	1.55	1.55
λ^*	0.089	0.341	0.267	0.287	0.331	0.15	0.044
$\kappa^*(10^{-02})$	0.889	1.54	1.17	1.25	8.75	1.23	0.889
$\mu^*(10^{-03})$	1.16	4.44	3.47	3.73	4.32	1.95	0.579

3. NUMERICAL MODELING OF THE BASE PROBLEM

In order to investigate the influence of creep on the behavior of an embankment on Haarajoki deposits, the construction and consolidation of Haarajoki test embankment on soft soils without any improvement were simulated with two different constitutive models , Soft Soil model(SS) and Soft Soil Creep model (SSC), using PLAXIS 2D Version 8.2 (Brinkgreve 2002). The test embankment was assumed symmetric and only half of the embankment is considered in the finite-element analyses. The plane strain condition and fifteen -noded triangular elements were used. The modeled range in vertical direction was 30m deep and horizontally 60m away from the embankment centerline. A finite-element mesh with 175 elements and 1495 nodes were generated as shown in fig 1.

3. 1. Boundary Conditions

The displacement boundary conditions were defined taking (ie. On y = 0 plane, $u_x = 0$ and $u_y = 0$ where u_x and u_y are horizontal and vertical displacements respectively) into account that the soft clay lays on a hard stratum.

The nodes on the two vertical boundaries were fixed against horizontal movement but allowed to move freely in the vertical direction. Assuming that the horizontal displacement can be defined as zero at nodes that are enough distant from the embankment, the plane of x = 60 m was considered as the lateral boundary with zero displacement in x-direction (ie. On planes x = 0 and x = 60, $u_x = 0$). The upper boundary formed by the embankment and the existing ground surface are left free to displace.

Drainage boundaries are assumed to be at the ground surface and at the bottom of the mesh (ie. the excess pore pressure at the nodes along the boundaries are set to zero), whereas the lateral boundaries are closed.

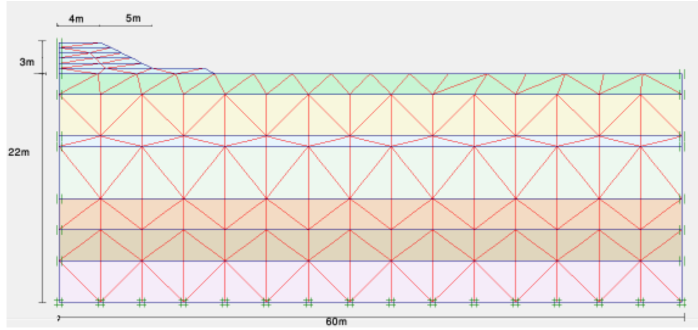


Fig. 1. Finite element analysis mesh

3.2 Initial Conditions and Calculations

An initial K_o- procedure is used to create the in-situ stress state, with a hydrostatic pore pressure generated from a groundwater table level at the ground surface. The coefficient of lateral earth pressure, K_o is based on the Jaky's formula K_o = 1-sinΦ. The embankment is deactivated while calculating the initial stresses.

The calculation consists of twelve stages. All stages are modeled as Consolidation – Staged Construction phases. The embankment construction consists of two phases: first, the embankment loading is applied under undrained conditions, assuming the embankment to be drained material and next, a consolidation phase is simulated via fully coupled consolidation analysis. The construction of Haarajoki embankment was done in 0.5 m layers, each taking 2 days, while the foundation layer was constructed in 5 days, and the real construction schedule has been simulated in the calculation. After the construction of each layer a consolidation phase is introduced to allow the excess pore pressures to dissipate. The construction of embankment was completed in 35 days. After construction of the last layer, the calculations have been taken until the excess pore pressure had dissipated to a residual value of 1 kPa to determine the final consolidation settlement.

3.3 RESULTS AND DISCUSSION

Results of the numerical predictions are compared terms of settlemet, lateral deformation and pore pressure with the actual field measurements for validation of the model.

3.3.1. Settlement

The predicted vertical settlements versus time at a node directly under the centerline of the embankment shows that the differences between the two models are relatively minor immediately after construction of the embankment, but become significant during consolidation. The soft soil model excluding creep reports a maximum settlement of 0.75 m at the end of consolidation period of 88 years. Here, primary consolidation ends after 40 years and plot becomes almost horizontal thereafter indicating the secondary consolidation stage. The soft soil creep model follow the same curve only up to the first 3 years and thereafter deviates greatly and gives a maximum settlement of 1.48m at the end of 260 years of consolidation.

To validate the above models, the time- settlement data are compared with the field value which is available only up to the first five years. The measured settlement underneath the centerline of the embankment is 0.46 m after about 5 years of consolidation. The SS model predicts a vertical displacement of about 0.35 m after 5 years of consolidation underneath the centerline of the embankment, while the SSC model predicts vertical displacements of about 0.405m (Fig. 2(a)). The final value of settlement underneath the crest, corresponding to about 5 years of consolidation, is measured to be 0.39 m. The predicted values are 0.29 m by SS model and 0.336 m by SSC model (Fig. 2 (b)).

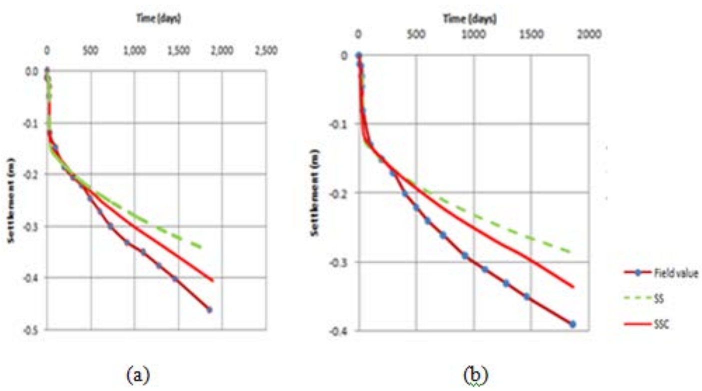


Fig. 2 Time-settlement curves: (a) center line; (b) 4 m from center line (under crest)

From the above results, it can be concluded that the Soft Soil model including creep predicts the vertical settlement more accurately than the one excluding creep. Both time-settlement curves suggest that the primary consolidation is still continuing after 2,000 days, which corresponds to the last measurement data available. The predicted surface settlements corresponding to a time immediately after construction and after 5 years of consolidation are shown in Fig.3.

Immediately after construction, measured values are in good agreement with predicted settlements. Both models predict small amounts of surface heave outside the embankment immediately after construction (Fig.3 (a)) and a maximum vertical settlement underneath the centerline of the embankment. The maximum vertical settlement measured underneath the centerline of the embankment is about 0.46 m after 5 years of consolidation. The SS model underpredicts the vertical displacement and value is about 0.35 m after 5 years of consolidation, while the SSC model predicts a value of about 0.41m (Fig. 3(b)). As moving away from the centre, both curves converge to a single one. Again it can be seen that SSC model is in good agreement with the observed surface settlements.

3. 3.2 Horizontal displacements

The horizontal displacements predicted by the SS and SSC models underneath the crest of the embankment (4 m from the centerline) and toe (9m from the centre line) after 1 and 3 years of consolidation are compared in Fig.4.

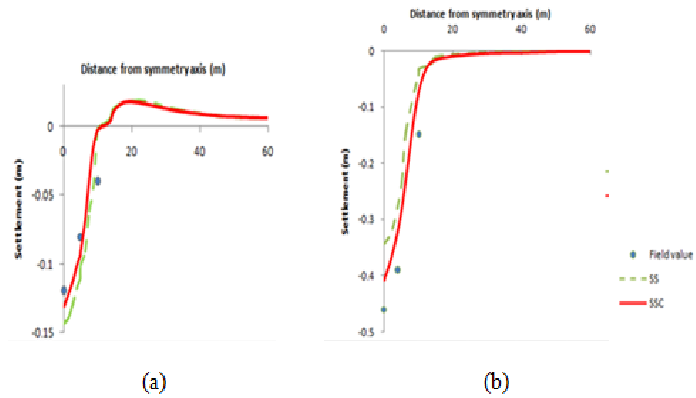


Fig. 3 Surface settlements: (a) immediately after construction; (b) after 5 years of consolidation

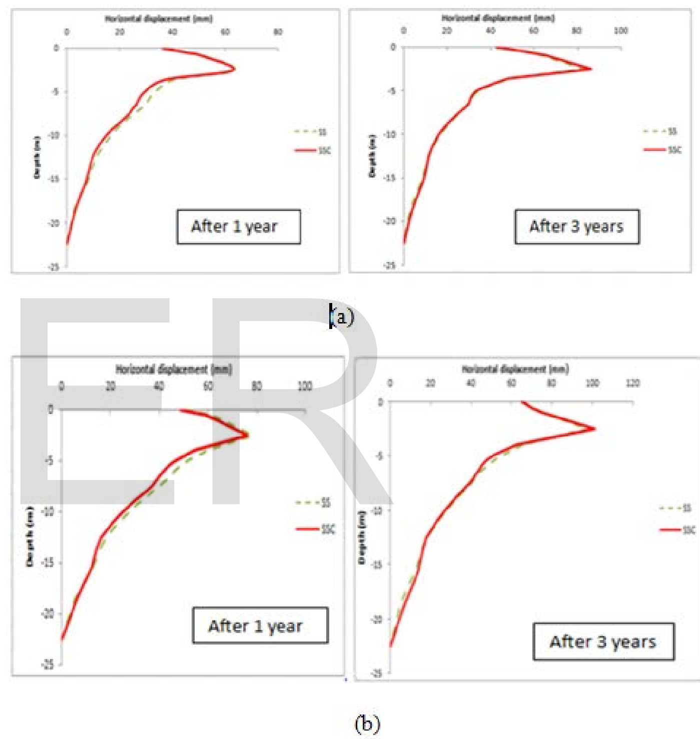


Fig. 4 Horizontal displacements underneath (a) crest, (b) toe of embankment

Underneath the crest, a maximum horizontal displacement of 0.055m is measured at a depth of 2.5m after 1 year of consolidation. Both models predict a maximum value of 0.063m. After 3 years of consolidation, the measured maximum displacement increases to 0.076m while the predicted value is about 0.082m. In all the cases, the depth of maximum horizontal deflection is predicted accurately. Both models over predicts the values up to a depth of 10m. At greater depths, lateral deflections are almost zero and results are in good agreement with measured values.

Fig. 4(b) shows the horizontal displacements predicted by the two models underneath the toe of the embankment (about 9 m from the centerline).After 1 year of consolidation, SSC model predicts lower values of displacements (even though the difference is of only about 3 cm) than SS up to a depth of 16m and then coincides. After three years, the measured maximum horizontal displacement occurs at a depth about 2.5 m and is about 0.062 m. Both the models overpredict the deflection values, but SS model gives comparatively better prediction. As expected, the lateral displacement of the ground increases with the distance from the embankment centerline (symmetry line).

3.3.3 Excess pore pressures

The predicted and measured excess pore pressure values at different depths under the centerline of the embankment are compared in Figure 5.

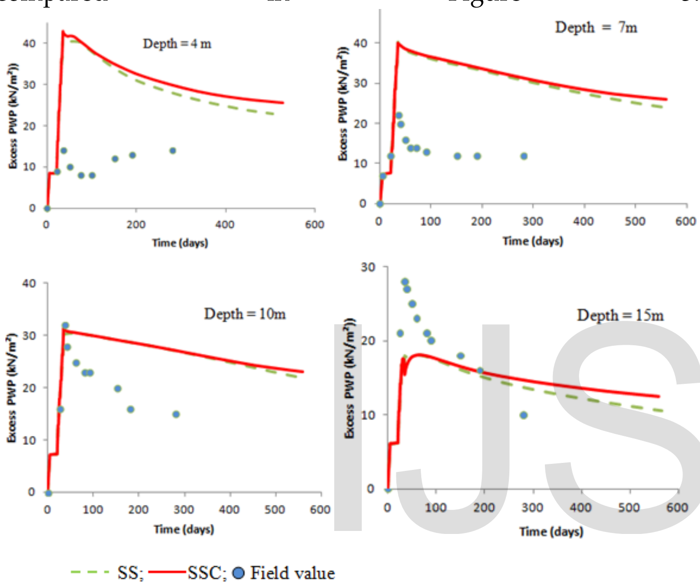


Fig. 5 Variation of excess pore pressures with time at different depths

As expected, the excess pore pressures increase during the embankment construction and then gradually dissipate with time. The predicted excess pore pressures are in general higher than the measured ones. The calculated excess pore pressure values correspond quite well with the measurements at depths of 10 and 15 m, while the predicted values at 4 and 7 m depths are greater than the measured values. This can be partly explained by the fact that the excess pore pressures are strongly influenced by the foundation soil permeability. The values of permeability used in the analyses were calculated directly from CRS oedometer test results. Laboratory tests normally underestimate the field values of permeability.

3.4. CONCLUSIONS

The influence of creep on the behavior of Haarajoki test embankment with sections constructed on unimproved soft clay deposit has been studied. With the exception of a 2 m thick dry crust, the soft soil deposit under Haarajoki embankment is normally or lightly overconsolidated, and hence very compressible. The soft clay is modeled with two different constitutive models, Soft Soil model and Soft Soil Creep model. The results of the finite element simulations, performed as large strain analyses, were compared with the field monitoring results. Based on these comparisons, it can be concluded that in case of normally consolidated or lightly overconsolidated soils, creep has to be taken into account.

4. PARAMETRIC STUDY

A parametric study is conducted to analyse and compare the effect of different ground improvement options. The clay layers are modeled with Soft Soil Creep model and embankment with the Mohr-Coulomb Model. Model parameters are similar to that used for unimproved embankment. The three ground improvement options, preloading, stone column and prefabricated vertical drains (PVD) are modelled and results are analyzed.

4.1. Model Parameters for Ground Improvement Options

Preload is provided on the ground surface in the form of sand embankment over the region where final embankment will be constructed. The embankment is trapezoidal in shape with 2:1 slope distribution and a height of 1.5 m. Unit weight of sand is taken as 22kN/m^3 and slope stability of the embankment is assured by providing higher angle of friction for material used. Preload embankment is constructed in 5 days immediately after initial phase and is maintained for 90 days resulting in further dissipation of pore-pressures.

Stone columns of diameter 0.6m are installed in a square grid pattern at 2m spacing to a depth of 10m which is the economical depth beyond which improvement is negligible. A granular bed of 0.5 m thickness and 100 times more permeable is provided on the top of stone columns for drainage purposes, as well as distribution of the applied stress coming from superstructure. The interface strength is assumed to 90% of the initial and R_{int} = 0.9 is adopted. A smear zone is introduced around the stone columns in order to take the effects of column installation on the drainage situation into account. The extension of this zone (d_s) is estimated to extend to double stone column diameter (Weber 2008) and the horizontal permeability (k_s) is reduced to half of the horizontal permeability of undisturbed soil (k_h).

The modeling parameters of preload and stone column are given in Table 2.

The effect of prefabricated vertical drains is studied using the drain option available in Plaxis. The length of the vertical drains is 10 m and they are installed in a square grid pattern underneath the embankment. The spacing is adjusted to 1 m so as to get a diameter – spacing ratio comparable to that of previous cases. The drain parameters relevant for the analysis are summarized in Table 3.

ISSN 2229-5518

Table 2. MC Parameters for Ground Improvement Options

Properties	Materials		
	Preload	SC	Sand Mat
γ_{unsat} (kN/m³)	18	16	17
γ_{sat} (kN/m³)	22	20	20
$k_h = k_v$ (m/d)	0.01	1	100
E (kN/m²)	40,000	60,000	40,000
c' (kN/m²)	2	0.2	1
Φ	40	42	35
Ψ	0	12	3
ν	0.35	0.3	0.3

Table 3 Vertical Drain Properties

Drain Pattern	Square net
Model	SOLPACK C634
Spacing (S)	1 m
Average width(w)	98.7 mm
Thickness (t)	6.83 mm
Discharge capacity, q_w	157 m³/ year

The analysis of stone column and PVD are done in plane strain by combined matching technique. The diameter of smear zone d_s for PVD is taken as $3d_m$ (Miura, 1999). The horizontal permeability of this zone is taken as one-third of undisturbed zone as proposed by many researchers.

4.2. Analysis of Results

4.2.1. Vertical settlement

The trend of vertical settlement with time for embankment on soft ground improved by the four ground improvement options are compared with that of unimproved embankment in fig. 6.

Comparing the results obtained, maximum reduction in settlement is obtained with stone column which may be due to the high stiffness of stone column material. The use of preload also decreases the post construction settlements, but the total consolidation time is not improved significantly. To decrease the consolidation period, either fill height or preloading period should be increased which may not be economical. The soft ground improved with PVD settles in much faster rate compared to the other options. This may be due to the high permeability of PVDs which is theoretically infinity.

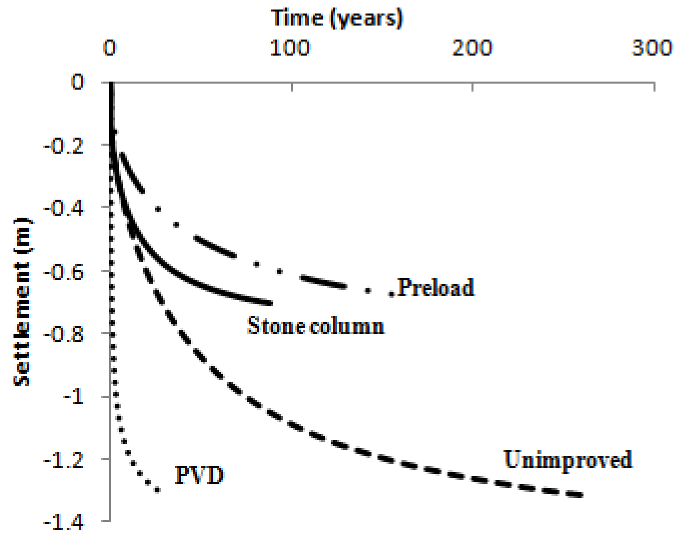


Fig. 6. Variation of settlement with time

4.2.2. Lateral Deformations

Fig. 7 shows a comparison of the lateral deformations under crest and toe of embankment.

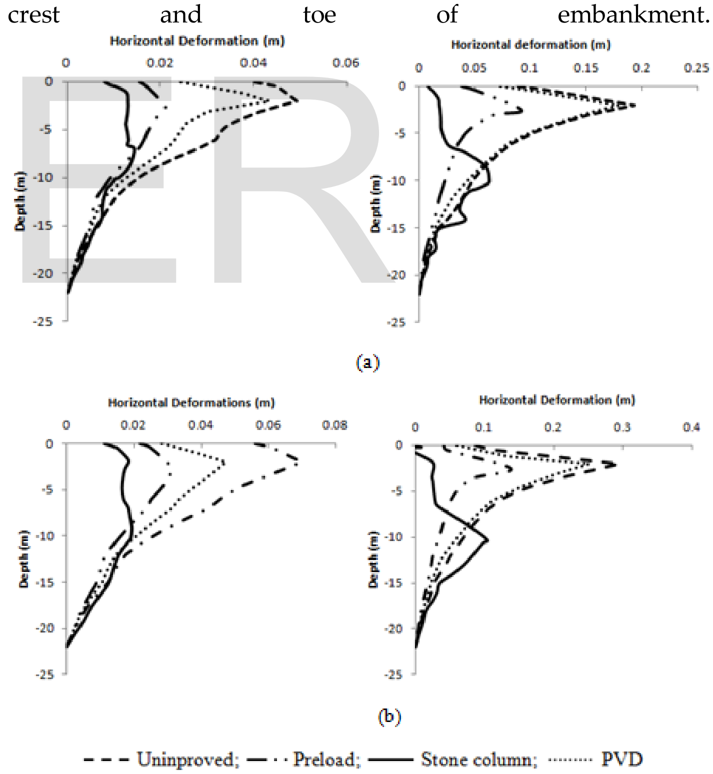


Fig. 7 Horizontal deformations; (a) under crest; (b) under toe

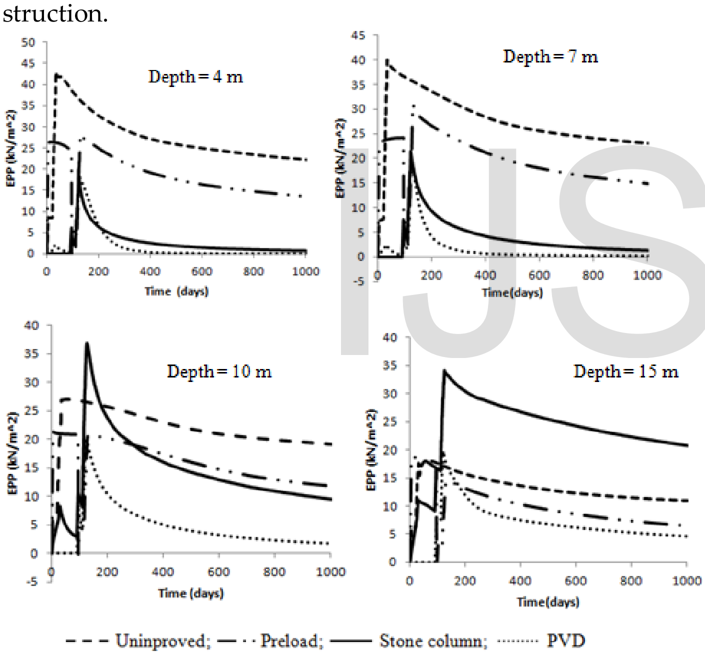
In case of embankment on soft soil without any improvement, horizontal deformations are predicted as 0.195 m and 0.294 m at a depth of 2.5m under crest and toe respectively after final consolidation. By improving the ground with different techniques, lateral deformations are reduced considerably. Minimum lateral movement is obtained with stone column due to

IJSER

the high stiffness of the material. The case with PVDs shows maximum deformations that are close to the unimproved case. In all the cases, maximum improvement is obtained on the surface and decreases with depth. Immediately after construction, all the cases show maximum deformation at a depth of 2.5 m and then decreases to zero at greater depths. After the consolidation is completed, stone column and sand column shows deviation from the general trend. In these cases, maximum deformation occur at a depth of 10m (toe of columns) and then decreases. Beyond the toe of columns, deformations are more than the unimproved case.

7.2.5 Excess Pore Pressure

Results for excess pore pressure shows that maximum excess pore water pressure occurs below the middle of the embankment crest (close to the symmetry line) and the values reduce with the distance from the centreline of the embankment. Figure7.18 shows the development of excess pore water pressures at different depths in the clay right after embankment construction.



In case of embankment on unimproved soil, maximum pore pressure predicted was 41.8 kN/m² at a depth of 4m. After the preload application, this value reduces to 28.3 kN/m² and a maximum improvement of 32% is obtained. As depth increases, percentage of improvement decreases and no effect is observed beyond 15 m. The percentage of improvement with stone column is of 53. Even though remarkable improvement is obtained up to 7m, pore pressure exceeds the unimproved case beyond 10m (toe of columns). It can be seen that the embankment load is transferred by the stone trenches to deeper soil layers and the maximum excess pore water pressure appears under the toe of the stone trenches. Compared to the other cases, excess pore pressure generated is minimum and the rate of dissipation is fast in case of PVD improved ground. Here 55% improvement is obtained at 4m depth and dissipation is almost complete within 500 days. But as the depth increases, time taken for the dissipation of pore pressure increases.

8. CONCLUSIONS AND RECOMMENDATIONS

Maximum reduction in vertical settlement is observed with stone columns due to the high stiffness of the material. Preloading also provides the same effect, but it requires more time to attain the ultimate settlement compared to stone columns. Out of the four options, PVD offer minimum time for the consolidation to be complete which outstands all other options. The time can be further reduced if length of PVD is increased to the bearing strata. This is not possible with the column techniques due to construction difficulties.

Lateral movements are maximum in PVDs due to its slender nature. Stone columns offer good improvement on the surface. But beyond the toe of stone trenches, no improvement is obtained.

The pore pressure dissipation is fast in case of PVDs due to high permeability and the generated pore pressures are low compared to the other techniques.

Based on the current study, further analytical, numerical and experimental studies associated with embankments stabilized with different techniques are suggested. The variation of permeability with radial distance from drain and the effect of compaction on the permeability are needed to be considered. More realistic analysis can be done by including well resistance also.

ACKNOWLEDGMENTS

The author thanks Dr. N. Sankar, Professor, Department of Civil Engineering, NIT Calicut for his systematic guidance, valuable advice and constant encouragement throughout this project.

REFERENCES

[1] Abdulazim Yildiz (2009), Numerical modeling of vertical drains with advanced constitutive models, *Computers and Geotechnics*, Vol. 36, pp 1072–1083.

[2] Arindam Dey and Mamidi Anvesh Reddy (2014), Compensated raft foundation on a preloaded soil improved by vertical drains, *Ground Improvement and Geosynthetics GSP*, Vol. 238, pp 515-525.

[3] Bahatin Gündüz (2008), Analysis of settlements of test embankments during 50 years - a comparison between field measurements and numerical analysis, *Master Thesis, Department of Construction Sciences, Lund University, Sweden*.

[4] R.A. Barron (1948), Consolidation of fine-grained soils by drain wells, *Transactions of the American Society of Civil Engineers*, Vol. 113, pp 718-742.