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Comparison of Different Ground Improvement Techniques for the Road Construction Over Kuttanadu Clay Strata

Abhijeet Kumar[1][0000-0002-5506-6299] and Parvathi Geetha Sreekantan[2][0000-0002-0454-3139]

[1] M. Tech Student, Department of Civil Engineering, National Institute of Technology Karnataka, Surathkal-575025.
[2] Senior Scientist, Geotechnical Engineering Division, CSIR - Central Road Research Institute, New Delhi- 110025.
abhijeetkumar.202gt001@nitk.edu.in

ABSTRACT: Kuttanadu clay is infamous for its very low shear strength and permeability. As a result, the construction of embankment over this stratum invites a lot of trouble including bearing capacity, settlement, and stability issues. To arrest these issues, proper ground improvement technique needs to be selected, which in turn is also governed by the construction cost, time and practical feasibility. One such case study is the upgradation of Alappuzha-Changanassery Road, where the height of the road level is proposed to be increased to tackle the issue of complete submergence during monsoon. In the present study, an attempt has been made to compare different ground improvement techniques in terms of stability, total settlement, and construction time. These techniques include prefabricated vertical drains (PVD) and lightweight filling using Geofoam. The 1-D consolidation and finite element analysis of these techniques were carried out using Midas Soilworks software and the results were compared with the embankment construction without any ground improvement technique. The study inferred that the lightweight filling using Geofoam blocks is the most efficient technique in terms of construction time, settlement, and stability.

Keywords: Kuttanadu clay; prefabricated vertical drain; EPS Geofoam; Finite element analysis, Consolidation analysis; Settlement.

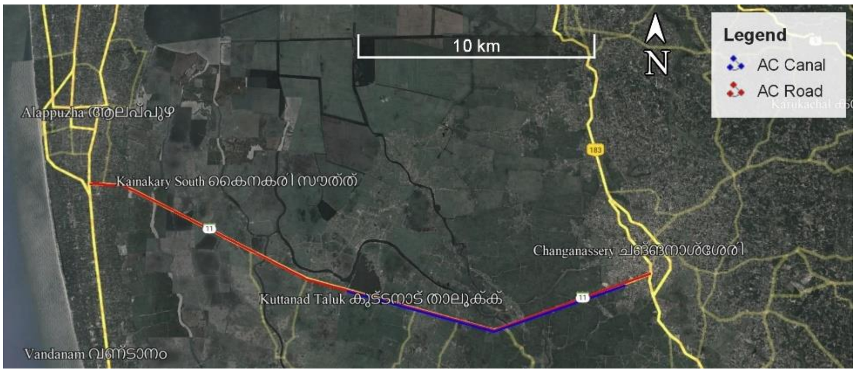
1 INTRODUCTION

State highway 11, which is popularly known as AC road (Alappuzha-Changanassery) road that starts from Kalarcode, Alappuzha and ends in Perunna, Changanassery, Kerala, India. This is a double-laned road of length 24.3 km. The road alignment is parallel to AC (Alappuzha-Changanassery) canal. Since the road was made through the paddy fields of the Kuttanadu area, where paddy cultivation is done below sea level, the whole road is a reclaimed land made on the soil collected from the paddy fields during the construction of the canal. Thus, the road is mostly bounded by paddy fields on one side and the canal on the other side. The construction of the canal, which was aimed to divert the excess water from the Kuttanadu region to Vembanadu Lake, remained incomplete even after several decades. Frequent rainfall and incomplete canal construction resulted in the submergence of the AC road even during moderate flood seasons. To overcome this difficulty, it was decided to elevate the level of AC road, with due consideration to the present and anticipated future vehicular traffic. However, this is a difficult task owing to the very soft clayey subsoil strata beneath the road embankment.

Several studies regarding road embankment construction over soft subsoils were reported across the world. The most common technique adopted was Prefabricated vertical drains with or without applying vacuum pressure (Bergado et al., 2002; Shen et al., 2005; Wu et al., 2014; Chen et al., 2016; Bergado et al., 2002), followed by granular or piled columns (Liu et al., 2007; Almeida et al., 2007; Liu et al., 2015; Nunez et al., 2013). The popular method of analysis and design of such ground improvement techniques followed calculations based on consolidation, area replacement and stress transfer between subsoil and columns. The use of finite element analysis tools facilitated the detailed study of these consolidation and stress transfer mechanism, thereby optimizing the solutions (Huang et al., 2006; Chai et al., 2013). The present study reports the possible solutions for the same considering the stability, total settlement, and construction time for the embankment raising. To this end, different solutions such as Prefabricated vertical drains and lightweight Geofoam embankments were designed and analysed using 2D finite element modelling. The results were also compared with that of the conventional embankment raising technique.

2 SITE CHARACTERISTICS

The AC Road is located in the Kuttanadu region which is surrounded by lakes and waterways. The aerial image of the Kuttanadu region along with the elevation profile of the road is shown in Fig. 1. The profile indicates that the initial 2 km stretch of the road has a higher elevation at 6m after which it remains at an elevation of -1 to 2 m from mean sea level (MSL). The high flood level recorded during the 2018 flood level (HFL) varied from 0.521 m to 0.764 m (KPWD, 2019). This indicates that the road has many stretches which are below the high flood level.



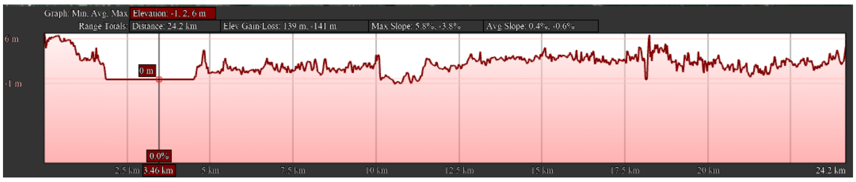


Fig. 1. Aerial image and elevation profile of AC road (Ref. Google earth, 2022)

A typical bore log data of the road stretch is shown in Fig. 2. The top layer consists of filled soil up to a maximum depth of 4.30 m. This is followed by very Loose Clay with an average N value of zero and Medium/Stiff Clay up to a depth of 50 m with an average N value of 40. The harder layers can be observed after 50 m, having Clayey Sand up to a depth of 60 m with an average N value greater than 50 and Sand layer with some silt with an average N value greater than 50. The groundwater table was found at a depth of 1.50 to 2.50 m from the ground surface, during the non-monsoon period. The liquid limit of clay layers varies from 35- 150, indicating high compressibility. The unit weights of the clay layers are also low indicating high void ratios.

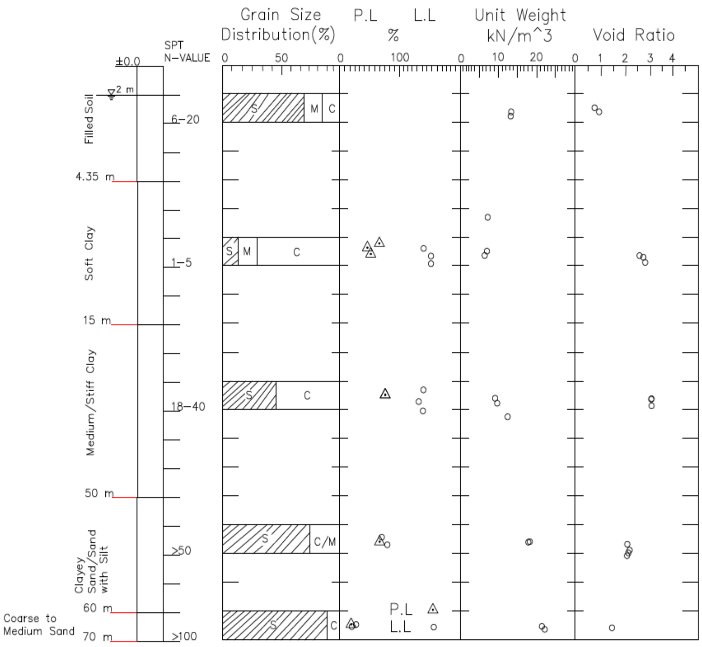


Fig. 2. Bore log data

The height increment proposed for the road ranges from 0.3 m to 1.2 m. However, the most critical section of this road is considered at a Chainage of 14/710, where the proposed embankment height is 0.805 m. This section comprises a 20 m deep soft clay layer below the existing embankment. Thus, this cross-section was used to analyse various ground improvement techniques. Fig. 3 shows the detailed cross-section profile.

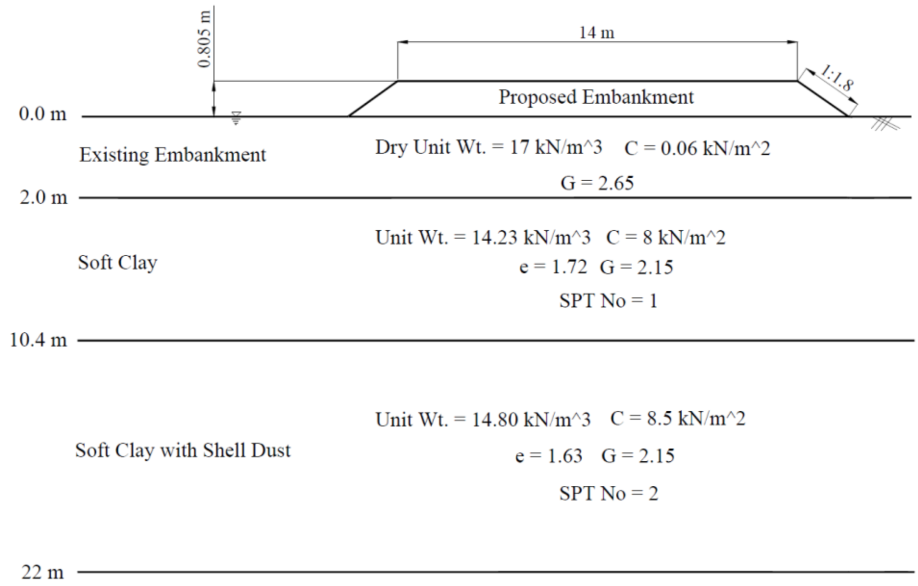


Fig. 3. Cross-Section Profile

3 MODELLING

To analyse various probable solutions for embankment construction, various options were considered as described below:

- Case a: Soil Embankment without Improvement.
- Case b: Soil Embankment with Improvement (PVD).
- Case c: Light Weight Fill Embankment (Geofoam).
- Case d: Light Weight Fill Embankment (Geofoam) with soil replacement for 0.5 m depth.

The height of new embankment construction remains 0.805 m for cases a, b, and c. However, after soil replacement, the height of the new embankment for case d increases to 1.305 m. 1-D consolidation and 2D finite element method were considered for the study. The computer program used for modelling the stability of road embankment is MIDAS soilworks, which is widely used for geotechnical modelling program. The constitutive models for soft clay are Mohr-Coulomb and Modified Cam Clay respectively for 1D consolidation and 2D-FEM analysis. For all other soil layers, Mohr-Coulomb constitutive model was adopted. However, for Geofoam, a linear elastic model was chosen (Stark et al, 2004). The input parameters for the soil embankment fill and subsoil layers are shown in Tables 1 and 2 respectively.

Different properties of proposed embankment soil, existing embankment soil, soft clay and prefabricated vertical drain and Lightweight fill used in 1-D consolidation Limit Equilibrium Method and Finite Element Method analysis for the calculation of construction time, settlement and stability are shown below.

Table 1. Soil, Existing Filled Soil and Geofoam Properties

Property	New Embankment fill	Existing Embankment	Geofoam
Bulk unit Weight (kN/m^3)	20	19.03	0.294
Saturated unit Weight (kN/m^3)	21	20.57	0.294
Cohesion, C (kPa)	0.5	0.06	59.75
Internal Friction Angle, Φ (^{o})	40	39	6
Modulus of Elasticity (kPa)	30000	29000	7400
Poisson Ratio	0.3	0.3	0.1
Earth Pressure Coeff., K_o	0.36	0.39	0.33
Permeability Coeff. (m/min)	$3*10^{-5}$	$3*10^{-5}$	$1*10^{-10}$

Table 2. Soft Clay Properties

Property	Layer 1	Layer 2
Modulus of Elasticity (kPa)	3000	3300
Saturated Unit Weight (kN/m^3)	14.23	14.80
Poisson Ratio	0.38	0.38
Earth Pressure Coeff., K_o	1	1
Cohesion, C (kPa)	8	10
Void Ratio	1.72	1.63
Horizontal Permeability (m/min)	$1.3*10^{-7}$	$1.3*10^{-7}$
Vertical Permeability (m/min)	$1.3*10^{-7}$	$1.3*10^{-7}$
Pre-consolidation Pressure, P_c (kPa)	38.06	64.06
Compression Index (C_c)	0.784	0.800
Over Consolidated Ratio (O.C.R)	1	1
Sec. Consolidated Coefficient	0.003	0.003
Strength Increase Ratio	0.26	0.26
Coeff. Of Consolidation (C_v)	$1.4*10^{-6}$ m^2/min	$1.5*10^{-6}$ m^2/min

For case b, Prefabricated Vertical drains (PVD) of width (D_w) 0.066 m having a Permeability of PVD is 0.06 m/min was considered for the analysis. For the smear zone, a permeability ratio (K_h/K_s) of 2 and a Coefficient of consolidation ratio of 0.33 were considered during modelling. To derive the optimal spacing of PVD, varying center to center distance of the PVD from 0.8 m to 1.5m was considered during consolidation analysis. To include the possible field variations in the horizontal consolidation coefficient, a varying ratio of consolidation ratio (Cv/Ch) from 0.2 to 1 was considered for the consolidation analysis.

For the 2D FEM finite element analysis, AutoCAD interface of the software was used to model the geometry. The sub-soil strata are defined as 2D plain strain elements. Six noded triangular elements of 0.5 m size were used for the surface discretization of the model. The mesh generation was followed by defining the loading conditions. Self-weight was defined initially for all cases. A pavement and traffic load of 24 kPa was applied over the top of the embankment as a line load in 2D.

2D non-linear stress analysis using finite element method was carried out for different construction stages mentioned below. The different stages of analysis for all the cases considered for analysis are given below:
Stage 1: Existing embankment with traffic load
Stage 2: Embankment height increment (with or without PVD)
Stage 3: Opening of traffic
The initial stage involves the calculation of initial stress conditions. It also includes ground support, drainage conditions, self-weight and live load (traffic). Further in the next stage, at the end of construction of the embankment after improvement, includes the embankment, and pavement load. In case d, the removal of 0.5 m of the existing embankment and filling of the same and further embankment height with lightweight fill (Geofoam) was considered. The last stage which is the opening of traffic includes the live load (traffic). The optimal spacing of PVD derived from consolidation analysis was used for FEM analysis.

4 RESULTS AND DISCUSSIONS

4.1 One-Dimensional Consolidation

One-dimensional consolidation analysis revealed that the total consolidation settlement for (case a) is 0.641 m. The improved shear strength of soft soil layers 1 and 2 are 8.4 kPa and 18.3 kPa respectively. The time required to achieve 90% degree of Consolidation is dependent on the consolidation ratio, and spacing of PVD. The variation of the same is indicated in Fig. 4 and Fig. 5. It can be observed that as the horizontal consolidation coefficient increases, the time required for achieving 90% degree of consolidation also decreases. It can be observed that as the PVD spacing increases, the time required for achieving 90% degree of consolidation increases. The results indicated that the average time for 90% consolidation is 340, 130 and 80 days for PVD spacing of 1.5 m, 1 m and 0.8 m respectively, with the constant coefficient of consolidation ratio of 0.33. However, optimal spacing of 1m requires 130 days for 90% consolidation. The results are compiled in Table 3.

Meanwhile, with improvements using Geofoam, the total consolidation significantly reduces to 0.487m and 0.171m for case c and case d respectively (Table 4). However, for case, d, there is a very significant acceleration of the consolidation time as to the other three cases. The time for 90% consolidation is derived as 225 days for case d.

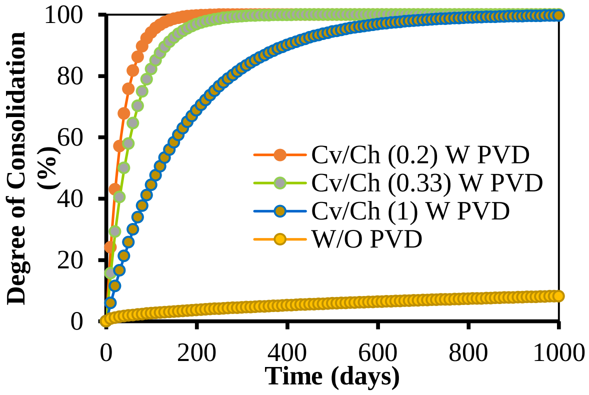


Fig. 4. Degree of consolidation for varying consolidation ratio (PVD spacing = 1 m)

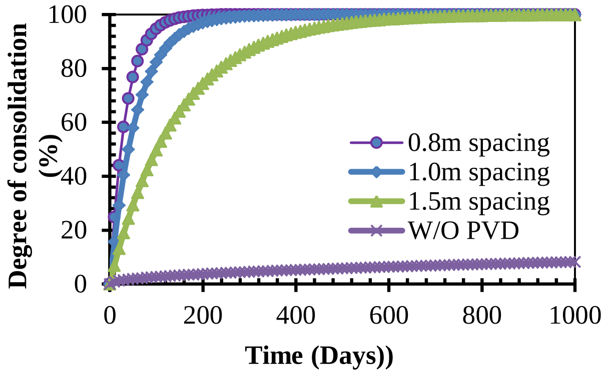


Fig. 5. Degree of consolidation for varying consolidation ratio (C_v/C_h =0.33)

Table 3. Consolidation Analysis Results

Case	Cohesion (kPa)	Cv/Ch	Total Settlement (m)	t_{90} (days)	Improved Cohesion (kPa)	
					Layer 1	Layer 2
Without Improvement		-		>10 yrs		
0.8m c/c PVD	8	0.2		50	8.4	18.3
		0.33	0.641	80		
		1		230		
1m c/c PVD		0.2		80		
		0.33		130		
		1		40		

		0
1.5m c/c PVD	0.	21
	2	0
	0.	34
	33	0
	1	10
		00

4.2 Finite element Analysis

Finite element analysis was carried out for a time period of 250 days. It was revealed that for case a and case b, the maximum settlement attained after stage 3 is only 0.12m and 0.35 respectively as shown in Fig. 6. This is different from consolidation analysis, where settlement was estimated at 0.03 m and 0.48 m respectively for case a and case b. This difference can be attributed to the immediate settlement and PVD smear effect considered in the FEM analysis. For case c and case d, the vertical settlement after stage 3 is observed as 0.18 and 0.16 m respectively. It needs to be noted that the degree of consolidation attained is maximum for case d, followed by case b, as shown in Fig. 7. This can be attributed to the fact that the increase in vertical stress is very minimal for case d, whereas accelerated consolidation using PVD is considered for case b. Thus, long-term settlement is negligible for case d and minimal for case b.

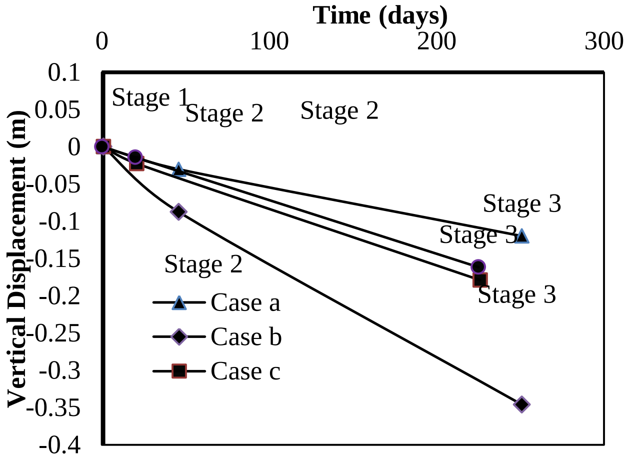


Fig. 6. Variation of vertical settlement for different stages

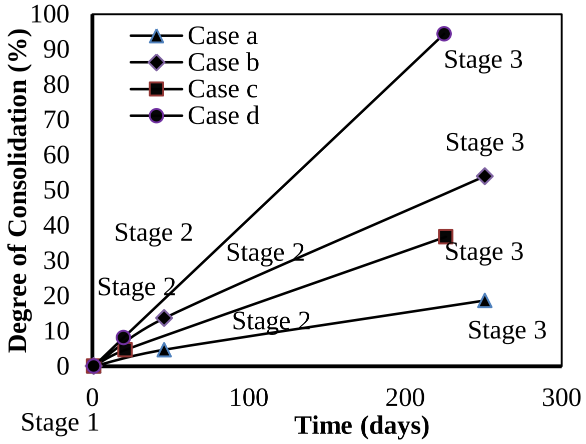


Fig. 7. Variation of degree of consolidation for different stages

As per IRC 75 (2015), The tolerable residual settlement for embankment is limited to 0.3m. However, this considers only consolidation settlement. During FEM analysis, the elastic settlement of the subsoil and the embankment fill is also compounded in the total vertical settlement. Thus, for the finite element analysis, a tolerable total settlement of 0.4m can be considered reasonable. To this end, FEM analysis was carried out till a limiting displacement of 0.4 m, to analyze the time required for attaining this settlement. The results are indicated in Table 4. It can be inferred that case a and c undergoes long-term settlement for more than 10 years. However, the rate of settlement is very slow for case a (8mm per year). This is an advantageous situation. However, the rate of settlement is the maximum for case b. For case d, the long term is negligible.

Table 4. Comparison of results

Case	Total 1D- consolidation settlement (m)	Maximum Settlement attained (m)	Consolidation Time (Years)	Degree of Consolidation (%)
Case a	0.641	0.4	50.1	62
Case b	0.641	0.4	2.6	62
Case c	0.487	0.4	10.5	89
Case d	0.171	0.16	0.61	90

The vertical settlement profile after stage 3 is indicated in Fig.8. It can be inferred that the vertical settlement is maximum at the center of the road for all the cases. However, for case b significant vertical settlement is observed in the neighbouring areas as well. This may affect the nearby structures. Except for case b, the effect of settlement extends to a depth of 5m. For case b, a deeper extent till 10m can be observed due to the lateral drainage of PVDs. The lateral displacement profile is shown in Fig. 9. The maximum lateral expansion of the left side below the existing embankment changes from 0.022 m to 0.035 m respectively for case a and case b. This indicates that after the installation of PVD due to an increase in the settlement there is a minor increase in lateral expansion as well. However, lateral expansion is observed at a depth of 2m. However, lateral displacements in the range of 0.03m are also observed at the embankment side slope. For case, c and case d, the maximum lateral displacement of the slide slope of the embankment slope is in the range of 0.03m. This lateral displacement at the embankment side slope in all the cases considered can be attributed to the rotational movement of the embankment along the slip circle. Even though this lateral displacement is very minimum and does not affect the serviceability of the structure, it is recommended to provide basal reinforcement using high-strength geogrid to arrest further movement due to the increase of traffic load in future.

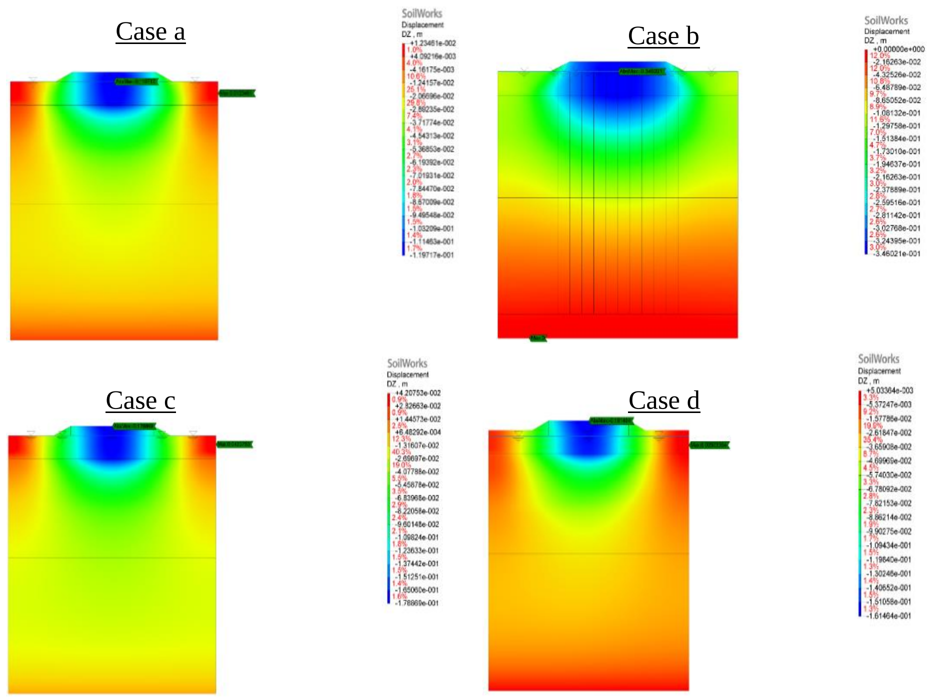
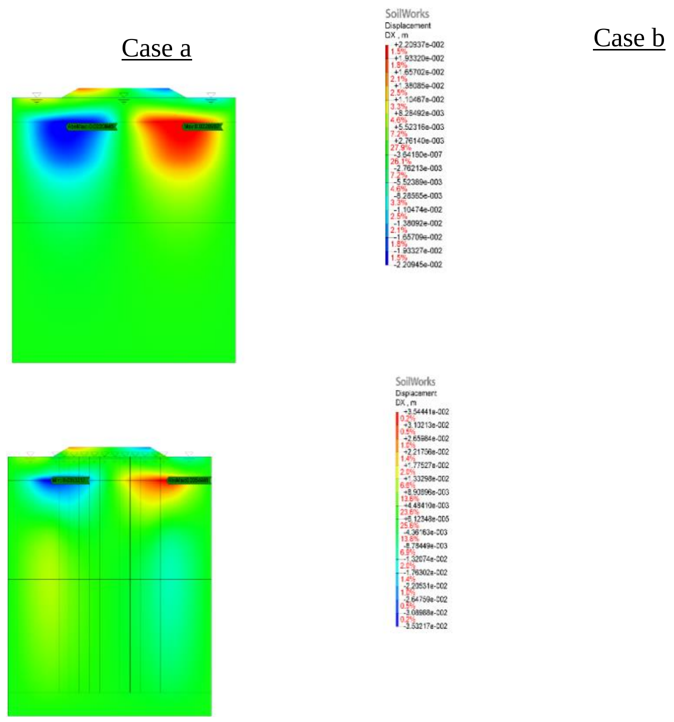


Fig. 8. Vertical displacement profile



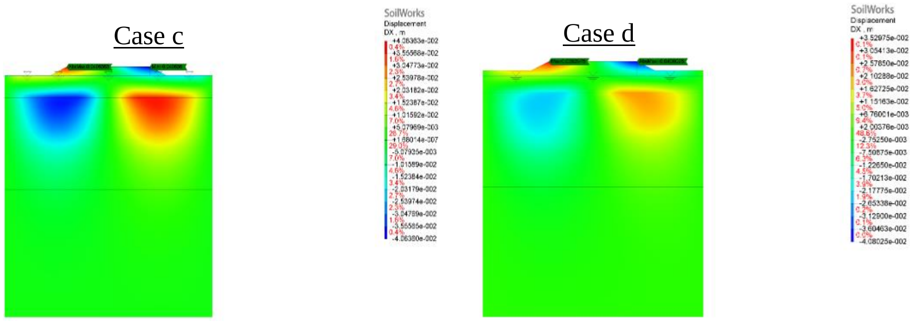


Fig. 9. Lateral displacement profile

5 CONCLUSIONS

In the present study, a comparison of different ground improvement techniques was made in terms of stability, total settlement, and construction time using 1-D consolidation and finite element analysis. The results were compared with the embankment construction without any ground improvement technique. The conclusions arrived at from the study are summarized in the following points

- The time required to achieve 90% degree of Consolidation is dependent on the consolidation ratio and spacing of PVD. The optimal spacing of PVD was derived as 1m, which requires 130 days of waiting period before laying the pavement. However, the improvement in shear strength after accelerated consolidation using PVD is very minimal for the first 10 m of soft clay.
- The degree of consolidation is maximum for the Geofoam embankment with 0.5m soil replacement, indicating faster pavement construction and negligible long-term settlement. In this case, the overall settlement is also within the tolerable settlement recommended by the IRC code. The long-term settlement is maximum when no ground improvement technique is used. However, the rate of this settlement is extremely low.
- FEM analysis indicated that the vertical settlement for the nearby areas is maximum for the embankment construction with PVD. For all other cases, settlement is limited to the area below the embankment.
- Lateral extrusion is observed for soft clay after a depth of 2m for embankment construction with conventional soil (with or without PVD). However, lateral displacement of 0.03m on the road shoulders, due to rotational slip circle movement can be anticipated for all cases.
- Considering these observations, it is recommended to adopt the embankment construction using Geofoam along with the removal of the top 0.5m of the existing embankment layer. However, since the rate of settlement is extremely low (8mm per year) conventional soil embankment without ground improvement may also be adopted provided, that lateral displacement is arrested using appropriate techniques.

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