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CASE STUDY

Analysis of Railway Embankment Supported with Geosynthetic-Encased Stone Columns in Soft Clays: A Case Study

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Abstract

Construction on soft soils is a promising task, as it is necessary for engineers to overcome challenges of excess settlements and possible bearing failure. This paper deals with proposing a suitable ground improvement technique as a remedy for an already failed railway embankment and evaluating the performance of the proposed technique both in short- and long-term periods. Based on the prevailing site conditions (i.e. poor shear strength) and obligatory to complete the work within the stipulated time, it is found that geosynthetic-encased stone column (ESC) is a preferred ground improvement technique. An extensive 2D finite element analysis has been undertaken to examine the improvement in bearing capacity concurrently addressing slope stability problem for different column spacing of 1.5, 2, 2.5, 3, 3.5, and 4 m. Interpretation of results revealed that bearing capacity and stability are critical in the short-term period. The results further demonstrated that ESCs by undergoing lateral deflection and bulging, which are found to predominate in the short-term period, sustained the failure against bearing capacity and stability. The spacing of 3 m when considered bearing capacity alone and 2.5 m corresponding to lateral deflection alone is found optimal. Based on the outcome of the study, it is demonstrated that the proposed ESC is an ideal solution of ground improvement for the present case study of failed railway embankment.

Keywords Soft clays · Geosynthetic-encased stone column · Finite element analysis · Lateral deformations · Bulging · Spacing

Introduction

There is a significant increase in infrastructure development including railways, bridges, roads, and other connectivity options in India, over the last few decades. Infrastructure development is a boon for economic growth and city development [1]. Noteworthy construction activity is taking place on offshore sites, wherein underlain soil strata is predominantly of soft soil category. The expansion and growth of infrastructure on soft soils cannot be avoided. Most of the offshore construction in India is likely to pose geohazards. The infrastructure built on soft soils ought to tolerate structural weights and loads from the encompassing soil [2]. Soft soils could be labelled into a few types: soft clay, organic soil, and peat [3]. The construction of foundations on such soils is cumbersome and intricate as soils hold rich water content, exhibit high compressibility and void ratio, and low shear strength [4]. Replacement of such soil is not easy and economical.

Soft clays can only sustain self-weight, and imposition of any additional overburden will lead to excessive settlement [5] or stability failure due to lateral sliding of soil mass [6]. Such soft soils are generally identified by low undrained shear strength of less than 15 kPa and high compressibility [7, 8]. In addition, low permeability and the presence of groundwater table closer to ground surface are also seen as characteristics of soft soils. Rapid construction on such soft soils can lead to build-up of considerably high excess pore water pressure [9–11] and accelerated settlements [12]. If an embankment is built over such soft soils, it is bound to undergo high settlements and consequently, run into settlement induced slope failure, mostly a deep-seated failure due to poor bearing capacity of subsoil strata [12,

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13]. Settlements stretching over many years make it mandatory to satisfy serviceability criterion and in many cases, stability criterion as well, of structures, to be built over soft clays [14, 15].

The probable solution to such problems wherein both strength and serviceability are equally and concurrently important is improving engineering properties of soils by a suitable ground improvement technique. Different techniques have been developed by researchers to progress the construction over soft soils, such as installation of Prefabricated Vertical Drains (PVD) [12, 16–20], deep soil mixing [21, 22], deep mix column combined with PVD [23, 24], stone columns [25–27], geosynthetic-encased stone columns (ESC) [28–30] etc. Longer construction times and abnormally high settlements followed by lateral displacements over longer time period are a few associated problems put forth by researchers in the case of PVD [18]. In deep mix column, rotational failure besides bending failure governs the performance of structures built over the soft soil [31–33]. Whereas in the case of stone column technique, bulging due to shear dilatancy and voids creation due to extensive deformation are disadvantageous factors as documented because of insufficient lateral confinement offered by soft soils [34, 35]. The use of geosynthetic encasement to the stone column increases its stiffness and reduces relative deformation [8].

In most of the practical scenarios, any one of the above ground improvement techniques is implemented. Generally, the construction activity begins post-treatment of the ground. Contrarily in a few cases, necessity of ground improvement arises post-construction of a structure when the foundation soil (i.e. soft clay) fails due to inadequate bearing strength. It is relatively easier to implement a particular ground improvement technique before a structure is built. But it is extremely challenging to propose and implement a particular ground improvement technique in soils that have already experienced failure. Case studies reporting different ground improvement solutions for the already failed embankment sites are also noteworthy. Roy and Singh [12] suggested PVD accompanied by preloading technique to stabilize the berm construction while redesign of two failed embankments built over soft soils. Undeniably, PVDs accelerate the dissipation of pore pressures, but settlement occurrence exorbitantly extends over longer periods. As such, Yegian and Lasalvia [36] suggested staged construction with a berm on both sides of the embankment. However, this method is time consuming and is often integrated with the vertical drains to speed-up the consolidation process and construction schedule [37]. Relatively, PVDs installation requires a little longer time in comparison with stone columns, but the installation cost is higher for the latter technique. Stone column is an efficient approach for the ground improvement where time requirement is a great deal, as it is much lesser in comparison to many other viable solutions [38]. Alternatively, Jamaludin and Jaffar [39] proposed reconstruction of a failed road embankment using reinforced geogrids, which offered the construction of a much steeper batter of the embankment. Lau and Cowland [40] reported the combined use of geosynthetic basal reinforcement and PVDs to achieve safe embankment construction height. The use of geosynthetic reinforcement offers an increased factor of safety for the embankment during the construction and also shortens the construction time. Deep mixing method (DMM), which involves a column type approach using lime/cement, provides an increased long-term strength [41]. But there are chances of rotational failure along with bending failure in the case of deep mixed columns. Resorting to grouted columns also results in increased shear strength and stiffness within the soft clay [42, 43]. However, this method is the most complex, apart from demanding skilled manpower, and can become uneconomical when implemented in a shorter time period. Gue and Tan [35] highlighted the reconstruction of two failed embankments using stone column approach to improve the subsoil properties. It is, however, reported insufficient lateral confinement from the surrounding soft soil as a major problem.

As such, many researchers advocate for resorting to stone column or geosynthetic-encased stone column technique of ground improvement, in particular, for soft soils, wherein settlements and stability, in tandem, are prime concerns to be addressed [44–46]. Stone columns develop bearing capacity by bulging deformation and lateral confinement offered by the surrounding soil [34]. Due to the lack of adequate lateral confinement from the soil in the upper portion of stone column, it experiences large lateral deformation or bulging [47]. In such cases, adoption of ESC is preferred, which is found to be providing not only sufficient lateral confinement but also facilitates speediness and economy in the construction, besides leveraging the advantage of bearing structural loads immediately after completing the construction. Several researchers reported the importance of ESC meant for serviceability [5, 7, 48–51] and stability [52–54] in soft soils under vertical as well as lateral loading conditions [55–57]. A review of comprehensive literature also suggests the use of ESC in any type of soft soil, which needs improvement in terms of both strength and stability.

Most of the previous studies mainly focused on the design of ESC using serviceability criterion, and more importantly, ground treatment is implemented prior to the construction of a structure. Moreover, a factor of safety calculation is done at the end of the construction period condition only. As such, effect of long-term consolidation on the evolution of factor of safety is mostly not investigated. Moreover, very few studies have attempted to understand the bearing capacity of ESC coupled with embankment slopes and their ensuing stability. Furthermore, efforts devoted to comprehending the role of lateral stresses offered for stability by ESC is also quite poor. It is important to couple the combined analysis of stability and bearing capacity to reduce the chances of failure of an embankment slope when the design of ESC is done by serviceability criterion. Keeping this in mind, an attempt is made in the present study to verify the feasibility of ESC ground improvement technique for an already failed railway embankment with emphasis on stability and serviceability simultaneously. For this purpose, extensive 2D finite element simulations are conducted, and the performance of ESC is evaluated. Parameters such as degree of consolidation and factor of safety as a function of column spacing are measured in both short- and long-term periods. Finally, spacing is optimized with a preference to stability criterion than the bearing capacity. The study finds that the encased stone column technique could be a perfect ground improvement technique, in particular, of foundation soils, especially of soft clays, which have already experienced failure. From the field observations, it is ascertained that the ESC construction is fast and economical.

Site Description

The study area considered is in Kendrapara region of Odisha state, India, and the work involves construction of an embankment for a new broad gauge railway line laying from Haridaspur to Paradeep. The track formation at the investigating site location is stretched over the length of 1500 m. One end of the formation line connects the bridge abutment while the other end transits as a mainline. The geometric details of embankment formation are given in Table 1.

The foundation soil is very soft clay with undrained shear strength in the range from 15 to 25 kPa. Groundwater level at the site location is almost near the ground surface and land on either side of the embankment is being used for cultivation by farmers. The thickness of the soft clay layer is 6 m from the natural ground surface overlain by silty clay of another 6 m thick. Below the silty clay layer, there exists 8 m thick sandy clay.

Table 1 Geometric details and engineering properties of embankment and foundation soils

Property	Embankment	Foundation soil layer		
		1	2	3
Top width (m)	7	–	–	–
Bottom width (m)	45.2	–	–	–
Height/thickness (m)	8.65 (Min.) & 10.65 (Max.)	0–6	6–12	12–20
Side slopes	2H: 1 V (above the berm), 2.14H: 1 V (below the berm)	–	–	–
c_c	–	0.389	0.186	–
e_0	–	0.76	0.65	–
c_v (cm^2/min)	–	0.098	0.211	–
γ (kN/m^3)	–	13	15	19.5
ϕ (°)	–	0	0	15
c_u (kPa)	–	15	25	451

Finite Element Modelling (FEM)

Finite element analysis was performed with the help of SIGMA/W and SLOPE/W module of GEOSTUDIO-2012. The constitutive model considered for the foundation soil (top layer: clay and bottom layer: silty clay), embankment and stone column was the elastic–plastic model with pore-water pressure (PWP) change. Whereas, encasement provided with geogrid was considered as linear-elastic with PWP change and applied at the edge of the stone column in regions as an interface element. A typical cross-section of 2D model adopted for FE analysis is shown in Fig. 1a. Whereas, Fig. 1b shows the schematic of ESC used in the present study. In real life, the stone column (i.e. 3-dimensional) has a definite diameter. To idealize the same into 2-dimensional geometry, modelling was done by converting the unit length in 2-dimensional using axisymmetric model into equivalent plane strain model, as per Tan et al. [58] method. The analysis was done considering that the soft clay is loaded in the form of an embankment over a period of 2 months. The prime reason for consideration of such a short period was obligatory by the client for completing the work within the stipulated time period. Moreover, this was the only section that remained to be completed and the track shall be opened for traffic within the timeline, as has apprised by the client. The embankment was constructed over the soft clay in four levels (L1–L4), as shown in Fig. 1a. The approximate time taken for completion of each layer was 15 days. The imposition of surcharge load stages is designated as L1, L2, L3, and L4. Effectively, the overall time of construction was 2 months (60 days). A blanket layer of 0.5 m thick with sand material was placed on top of the ESC. Two layers of geogrid were embedded within the sand layer with a vertical spacing of 0.5 m. Effectively, a sand blanket is sandwiched between natural soft clay and embankment. The function of geogrid reinforcement is to distribute the embankment surcharge load uniformly on all ESCs, and the blanket layer is to provide a high permeability drainage layer for dissipating excess PWP, getting accentuated with the application of surcharge load.

The dissipation of excess PWP was allowed to account for consolidation by applying boundary conditions of drainage to the top face of the ground surface. Successively, the rate of consolidation was measured by the following Eq. (1):

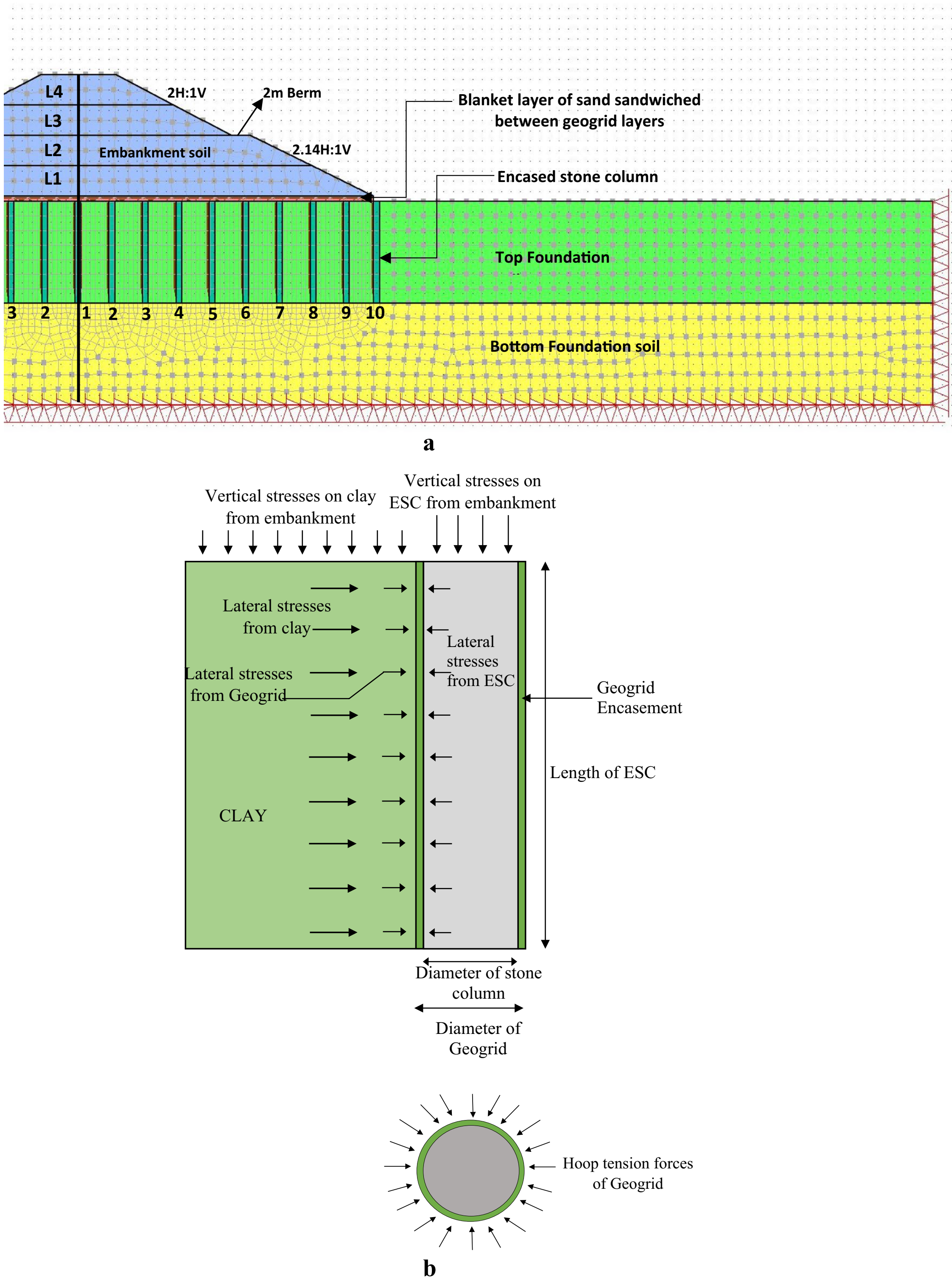


Fig. 1 **a** Typical cross section of FE model depicting geometrical details and components of earthen embankment and foundation soils. **b** Schematic of ESC depicting different stresses acting on the column

$$U\% = \frac{u_i - u_t}{u_i} \times 100, \tag{1}$$

where u_i is the initial excess pore water pressure at the end of construction and u_t is the excess pore water pressure at any time (t) from the end of construction.

With regard to the boundary conditions, fixity in both X- and Y-directions was assigned to the bottom side, while fixity in X-direction only was assigned to vertical sides on both sides of the foundation soil. The top layer was considered as the potential seepage face facilitating the dissipation of excess PWP. A global mesh size of 1.5 m was assigned to the model. However, clay soil in between the adjacent columns was discretized with a mesh size of 0.45 m. The diameter of stone columns was fixed at 0.6 m. whereas, spacing was a variable parameter in the range from 1.5 to 4 m with a triangular pattern. As per IS 15284 Part-1, spacing between stone columns can be kept at 2–3 times the diameter of stone columns. But the spacing can further be increased according to site conditions. Similarly, FHWA-NHI manual on ground improvement suggests spacing of 5 feet (1.52 m) to 12 feet (3.66 m) for triangular, square or grid pattern of installation. According to Hughes and Withers [27], the increase in spacing between stone columns above 2.5 times the diameter makes the effect of applied load on soil insignificant. In the present case study, the embankment is already failed and there is a close deadline to meet for completing the rail line. It is further noticed that the installation of stone columns with close spacing of 2–3 times column diameter found to have damaging impact on the adjacent columns. Bearing in mind the above points, relatively wider spacing is adopted in the present study.

The overall length of the stone column was 10 m, extending over the entire soft clay layer thickness. Encasement provided was having a stiffness modulus (J) of 4000 kN/m, while the tensile strength at 2% strain was 80 kN. A 0.5 m thick sand layer was provided over the clay layer to interface with the ground surface. The sand layer acts as a blanket layer to dissipate the excess PWP with time. The sand layer was effectively sandwiched between two horizontally laid geogrid reinforcement layers, which is having the same properties as the encasement for ESC. This sand layer was considered in the effective drained condition, so the properties designated do not require to assign permeability. Abstract of properties, which were provided by Rail Vikas Nigam Limited, Bhubaneswar, Odisha, India, considered for different materials can be seen in Table 2.

Consolidation and Slope Stability Analysis

Consolidation analysis was performed by SIGMA/W module of Geostudio-2012. Materials and models as explained in the earlier sections were used for analysis purpose. Simulations were continued with varying time steps until 90% degree of consolidation is achieved by simultaneous monitoring of dissipation of excess PWP. Predicted excess PWP dissipation with time was used to calculate the percentage degree of consolidation using Eq. (1).

Slope stability analysis was conducted using SLOPE/W module of GeoStudio-2012. The stability was analyzed through the known stresses at each node. This method uses the stresses derived by SIGMA/W module, which fed the rate of consolidation and dissipation of excess PWP as input parameters, and analyzes the critical factor of safety as described in the engineering manual of SLOPE/W. The stresses were calculated based on FE analysis performed over a previously defined FE model. Critical factor of safety (FOS) for the system along with the failure slope was predicted by finite element stress based approach. The material behavior was defined jointly by Mohr–Coulomb constitutive model and those properties considered in SIGMA/W analysis. The boundaries for developing critical slip surfaces were defined by the “entry and exit method” option by selecting the optimization of the slip circle. The value of FOS was determined as a function of degree of consolidation up to 90% with an increment of 10% starting from 10%.

Measurement of Bulging and Lateral Deformations

Upon loading of ESC in the form of a surcharge pressure, two fundamental behaviors of ESC (i.e. bulging deformation and lateral deflection) takes the control to offer the resistance against applied pressure. The bulging is defined as increase in diameter of ESC on the application of load, and it can be calculated as the difference between the original and enlarged diameter of ESC. While lateral deflection is the movement of one edge of the stone column relative to another. Bulging and lateral deformations were measured

Table 2 Elastic and geotechnical properties of materials considered for FEM analysis

	E (kPa)	μ	γ (kN/m^3)	c' (kPa)	ϕ' (°)	e	k_x (m/days)	k_y/k_x
Soft clay (top soil)	2000	0.4	15	20	10	1.2	3.51×10^{-5}	0.5
Silty clay (bottom soil)	20,000	0.3	19.5	45	15	0.76	3.51×10^{-5}	0.5
Embankment soil	10,000	0.3	19	20	25	0.38	12.96	0.667
Stone column	40,000	0.3	22	0	34	0.3	0.001	1
Sand	20,000	0.3	22	43	26	0.38	–	–

post FE analysis of columns numbered 1–10, as shown in Fig. 1a. Column 1 is located at the centre of the embankment, while column 10 is located at the toe of the embankment. Measurements are made on both sides (L: left and R: Right) of each column as well as along its length corresponding to 90% degree of consolidation.

Results and Discussion

A detailed FE analysis results for the slope with and without reinforcement are presented in the following sections. Emphasis is specifically given to delineate the change in the dissipation of excess PWP, development of settlement and effective stress, evolution of FOS, bulging, and lateral deformation with the change in spacing of ESC.

Stability Analysis of Unreinforced Ground

The embankment construction on silty clay was prolonged over a period of six months, in the year 2019. Initially, virgin soil on the ground up to a depth of 2 m, as per RDSO guidelines, was replaced with sand. The replacement is extended to a width of 1 m beyond the toe of the embankment to provide drainage path from upstream to downstream side. Failure of the embankment was witnessed during the construction it-self when the final height just reached the formation level (i.e. blanket layer). Physical inspection revealed that failure occurred over a length of 500 m predominantly at the curved portion where the embankment height is approximately 9.851 m. Figure 2a–d gives glimpses of the failed embankment. A portion of the embankment slid in the lateral direction resulting in the formation of a deep vertical cut can be seen from the pictures. Disembarked soil from the embankment further slithered foundation soil mass, causing a heave on the failure side of the embankment, as evident from photographs. Apparently, the failure mode is a base failure.

It is to be noted that the undrained shear strength of foundation soil is a mere 15 kPa and the final settlement calculated at the middle of the second foundation layer of stiffer silty clay was 2064 mm. Clearly, final settlements are exorbitantly large and at the same time, strength is significantly low. These calculations, as such, imply the occurrence of settlement induced stability failure of the embankment. A simple calculation of the permissible height of embankment based on undrained strength and bearing capacity theory shows that a surcharge height of not more than 3.5 m shall be imposed at one time without the inclusion of reinforcement. Understandably, the low strength and high settlement indicate that it is not ideal to place a surcharge height on the natural ground for an intended height, as is done in the present case study.

In addition, it is a perspective to anticipate rapid build-up of excess PWP as the construction progresses, as drainage condition is 1D which is allowed only at the ground surface.

Fig. 2 **a** Development of the first crack on the formation. **b** Crack expansion and propagation leading to slope failure. **c** Observance of deep vertical cut on the post-failed embankment. **d** Failed and intact embankment along the curved alignment

Obviously, the imposed surcharge load causes development of excess PWP to a magnitude of 217 kPa, a remarkably greater value for such a low strength foundation soil. With the build-up of excess PWP, there is a further decrease in strength. This might have one of the important factors that accentuated the failure of embankment during the construction. Confirming the above facts, the stability analysis conducted using SLOPE/W yielded the least FOS of 0.788, which is less than 1. This clearly shows that the mobilized shear stresses are far greater than the shear strength offered by the foundation soil, ultimately leading to failure of the embankment. This endorses the necessity for the adoption of a suitable ground improvement technique, encased-stone column here in the present study, to improve bearing capacity with the subsequent reduction in settlement concurrently mitigating the possibility of stability failure.

Performance Analysis of ESC for Actual Site Conditions

The degree of consolidation with time for different spacings (*S*) is shown in Fig. 3. It is, in general, seen from trends a continuous increase in the degree of consolidation with time for all spacing's considered in the study. It can further be noticed that the time required for 90% consolidation reduced as the spacing between ESC is reduced and vice-versa. It is clearly seen from Fig. 3 that, as spacing increases so does the time required for achieving the higher degree of consolidation. Evidently, the impact of spacing is phenomenally clear from the trends, as the time required for attaining a particular degree of consolidation declined with the closer spacing. The reduction in time with a decline in spacing can be attributed to the effect of reduced total stress on clay due to closely spaced ESC [59]. From the overlap of degree of consolidation curves for 3.5 and 4 m from Fig. 3, it can be stated that adoption of spacing beyond 3.5 m does not fetch any advantage. This highlights a fact that 3.5 m spacing can be considered as limiting spacing based on the degree of consolidation. The effective time required for accomplishing 90% consolidation for 1.5 and 3.5 m spacing is measured as 290 and 810 days after deducting 60 days of loading time, respectively.

Similarly, Fig. 4 depicts the variations of excess PWP measured at the centre of the clay layer with time for different spacings of ESC. It is obvious that with the decline of excess PWP, there is an improvement in the degree of consolidation within the soft clay. The same can be witnessed from Figs. 3 and 4. Initially, there is an increase in excess PWP up to approximately 60 days and thereafter, continuously declined with further increase in time, corresponding to all spacings. It must be noted that the surcharge load is imposed for a period of 60 days (refer to Fig. 1a). The rise in excess PWP, in the beginning, can be linked to the imposition of such surcharge load. Incidentally, the pattern of excess PWP dissipation shown in Fig. 4 matches with the results of Tan et al. [58]. Demonstrably, the effect of spacing on the dissipation of excess PWP is distinct. As evinced from Fig. 4, excess PWP increases as the spacing between stone columns increases. Apparently, excess PWP is greater for higher spacing and vice-versa. A maximum peak for 3.5 m or above and conversely, the minimum peak for 1.5 m spacing can be noticed from Fig. 4. This variation can be attributed to the fact that closely installed stone columns reduce the total stress acting on the clay which in turn reduces the excess PWP generated [59, 60]. Further, the reduction in spacing between stone columns also shortens the radial flow path by contracting the time required for the dissipation of excess PWP [45]. The impact of spacing on excess PWP can be attributed to the reduced area replacement ratio as well. For a given area replacement ratio, there

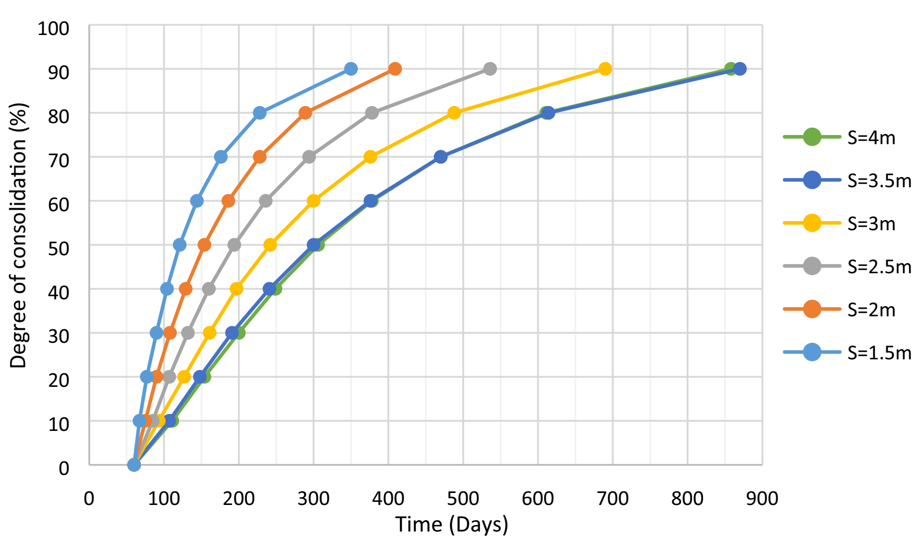


Fig. 3 The change in degree of consolidation with time for different spacing's considered in the study

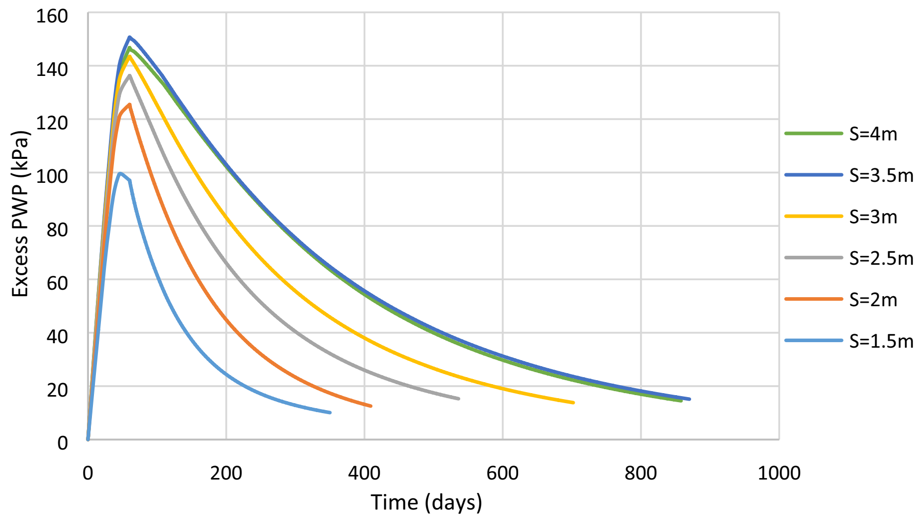


Fig. 4 Dissipation of excess PWP with time for different spacing's considered in the study

is an inclusion of higher permeability material (gravel) than the low permeability material (silty clay). This might have led to faster dissipation of excess PWP (Fig. 4) and thereby, reduction in time for 90% consolidation. The same fact can be authenticated with the interpretation of the time required for 90% consolidation at 230 and 910 days for 1.5 and 4 m spacing, respectively.

From Fig. 3, it is seen that the time required for 10% consolidation is nearly closer across all spacings. This indicates the participation of ESC in reducing the rate of consolidation is significant in later stages than at the incipient state of dissipation of excess PWP. This can be observed from Fig. 4, as the drop in excess PWP by sudden dissipation owing to the fact that the time at which the drainage is allowed is 0 days.

Figure 5 shows increments in settlements predicted at the middle of the silty clay layer for 90% degree of consolidation with the change in time and spacing. Evidently, consolidation settlements irrespective of spacing started beyond 60 days, as the excess PWP has reached a peak only at 60 days. Similar trends are reported by Tan et al. [58], who have considered a single spacing of ESC. It can be observed from Fig. 5 that, with an increase in spacing between ESC, settlements at the end of the construction are also increased. The least settlement was predicted for the closer spacing of ESC (i.e. 1.5 m) at a shorter period of time, while the maximum settlement was predicted for the greater spacing of 4 m at relatively higher time periods. A simple comparison of Figs. 3, 4, and 5 reveals a fact that there seems to be a contrast in the effect of spacing on the degree of consolidation, excess PWP, and settlement predictions. Both 3.5 and 4 m spacings have produced an identical degree of consolidation and excess PWP (Figs. 3 and 4), but not the settlement values, as is seen from Fig. 5. Though the ultimate time (i.e. 810 days) needed for achieving 90% consolidation has remained the same, settlement values are slightly differed by 25 mm. Final settlement of 375 mm for 3.5 m and 400 mm for 4 m is predicted. As such, closer spacing of 1.5 m produced the highest degree of consolidation in a minimum

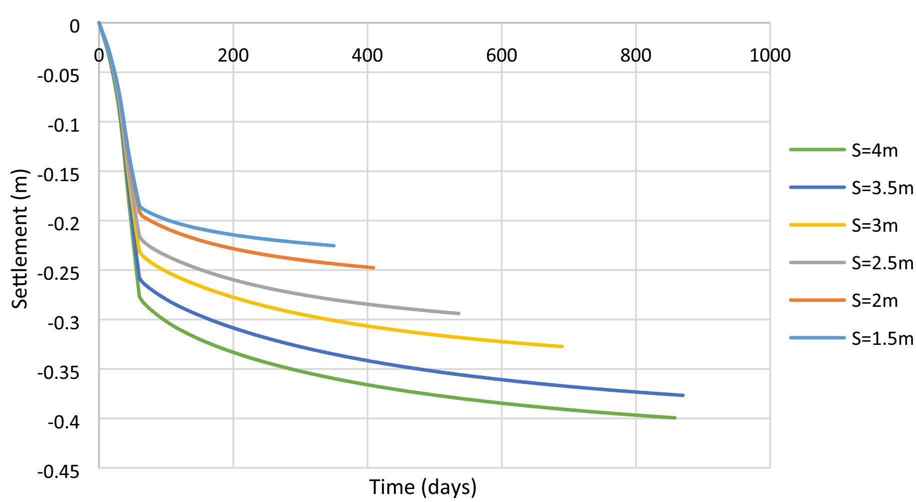


Fig. 5 Predicted settlements with time for different spacing's considered in the study

time of 260 days. But, much closer spacing also yielded the least settlement of 225 mm. On contrary, the maximum spacing of 3.5 m or above produced 90% of consolidation in a significantly longer time period of 810 days with the highest settlement of 400 mm. The reason for the higher settlement is that closely spaced ESCs reduced the total stress acting on the clay, which in turn reduced the settlement [59, 60]. As such, these results clearly imply that spacing up to 3.0 m does not fetch a greater advantage. The reason for an increase in settlement with spacing can be linked to reduced interaction between clay and ESC with an increased bearing load [5]. This can be understood from the perspective of the equivalent Young's modulus in the equivalent area method, which increases with an increase in area replacement ratio. This area replacement ratio increases when the spacing of ESCs is reduced, causing increased settlement at a greater spacing of 3 m and above.

Figure 6 presents the values of FOS determined with the progress of consolidation phenomenon. FOS is predicted pertinent to two stages. First, with the imposition of surcharge load alone, herein designated with respect to load increments of L1, L2, L3, and L4 (refer to Fig. 1). Second, with an improvement in the degree of consolidation from 10 to 90%, as shown in Fig. 3. It can be observed from the figure that the values of FOS reduced to below unity (i.e. 0.95) for all the spacing's, except for 1.5 m, by the time loading reaches to L4 stage. Apparently, L1 is neither the end of loading period nor desirable height of the embankment above 8 m. FOS of less than unity clearly indicates failure of the embankment if bearing support is not provided in the form of ESC. Development of excess PWP leading to a further reduction in undrained shear strength within clay layer with the imposition of load can be reasoned out as the cause for such lower values of FOS. Nevertheless, with the dissipation of excess PWP, there is a steady increase in FOS for all the spacings. Highest FOS for the shortest time is achieved for 1.5 m spacing. It is worth mentioning here that this spacing predicted not only the lowest settlement but adopting this spacing is not beneficial in terms of economy and required time for completion of the project. Thus, the spacing of 1.5 m can be disregarded in practical feasibility viewpoint in the real life.

Evident from Fig. 6, as the consolidation progresses so does FOS evolves for all the spacings. Furthermore, a constant improvement in FOS from the beginning of reinforcement inclusion can be noticed. This is advantageous and desirable from the bearing capacity viewpoint, as it is imperative to bear the entire surcharge load by the ESCs alone at the initial stage. At the state of reinforcement inclusion, the highest spacing of 3.5 m has yielded the lowest FOS of 0.89, while the lowest spacing of 1.5 m has produced the highest FOS of 1.23. However, a complete reversal in behaviour can be witnessed at the end of consolidation i.e. 90%. This can be corroborated with the change in the rate of the consolidation process and improvement in effective stress consequent to consolidation. Effective stress increases with the consolidation time progresses, as is clear from Fig. 7, for 4 m spacing even after 400 days, while the same is ceased at 200 days for 1.5 m spacing. A steady increase in FOS value indicates that there seems to be no plateau in the values of FOS even beyond 90% of consolidation.

The above observations related to the progression of FOS can be endorsed with the constant evolution of effective stress within the foundation soil. Figures 7 and 8 depict the increment in effective stress within ESC and clayey soil with time for different spacing conditions. Based on the trend variations, it is easy to infer a clear distinction in the rise of effective stress between ESC and surrounding clay. Understandably, the effective stress developed in the surrounding clay is nearly same for spacing's of 1.5 and 2 m, and 3.5 and 4 m, respectively. The increase in effective stress is the effect of dissipation of excess PWP, which manifests

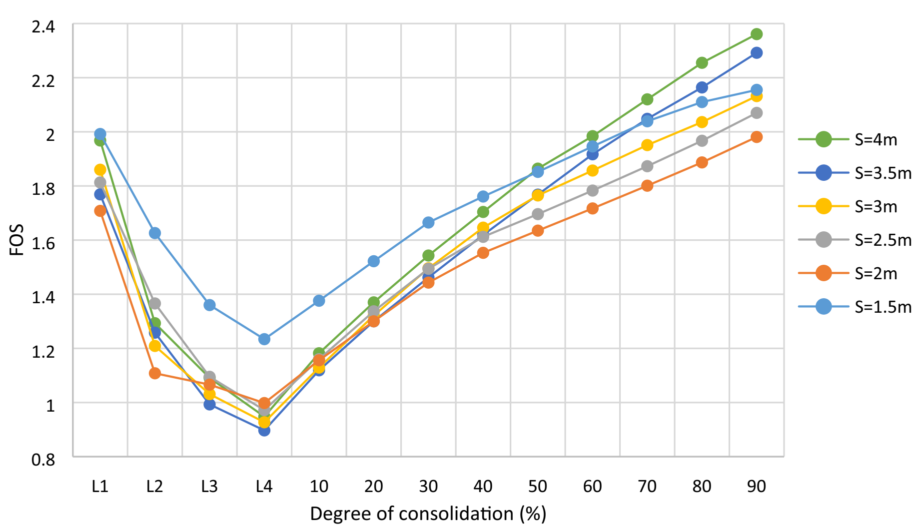


Fig. 6 Predicted FOS with the imposition of loading and change in the degree of consolidation for different spacing's considered in the study

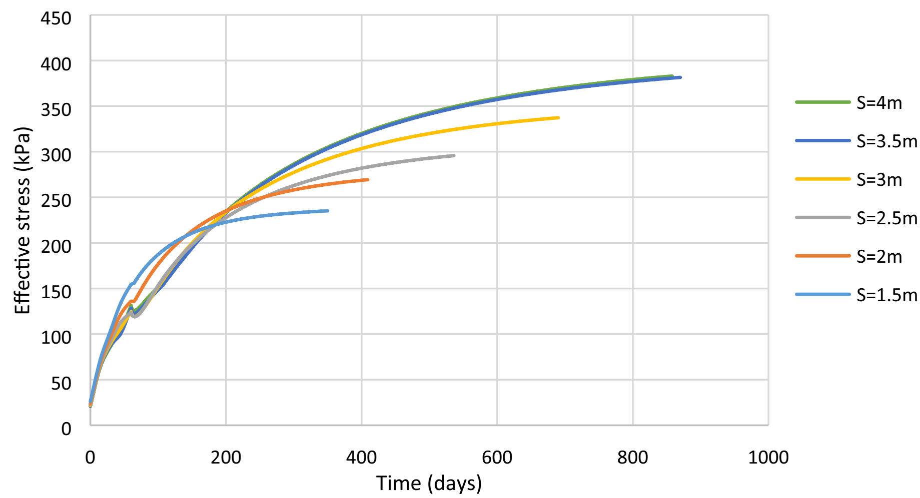


Fig. 7 Evolution of effective stress in ESC with time for different spacing's considered in the study

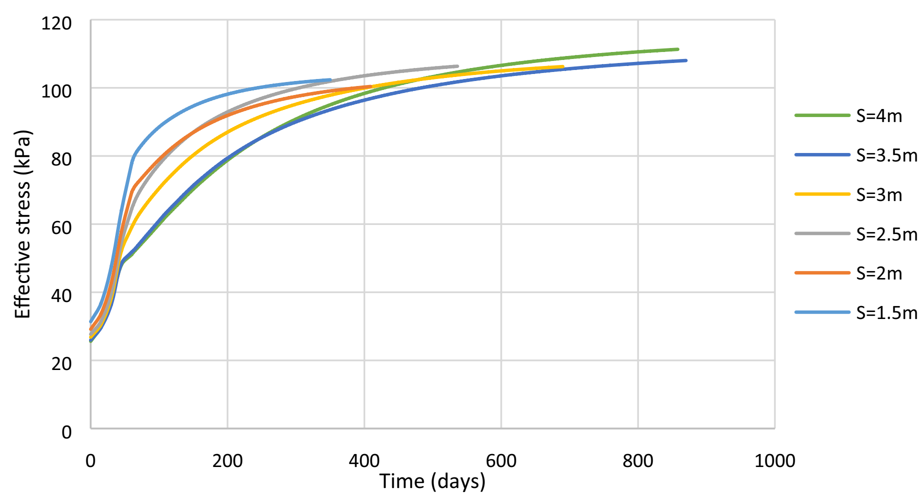


Fig. 8 Evolution of effective stress in clay soil with time for different spacing's considered in the study

its effect in re-proportioning the load transfer between ESC and the surrounding soil, with a steady increase in loading condition on the latter one. This has led to the settlement of surrounding soil by compression of voids which are emptied by the dissipation of PWP. It can be noticed from the results that, at the time of 200 days, the effective stress on clay is greater for closer spacing of 1.5 m than the larger span of 4 m. On contrary, a higher magnitude of effective stress can be found for the larger spacing of 3.5 m and above, in comparison with a closer spacing of 1.5 m. These contrarieties can be attributed to differences in the rate of dissipation of excess PWP.

A closer examination of Figs. 7 and 8 also proves a fact of inconsistency in the evolution of effective stress. A perfect non-linear increment of effective stress within ESC indicates that there is a steady load imposition on it from the beginning. From the results presented in these figures, it can be inferred that the majority of the load is shared by ESC, while the load sharing by surrounding soil depended on advancement in effective stress as the consolidation advances. Overall, these results highlight a fact that the increase in FOS with time is primarily dependant on the evolution of effective stresses within ESC and in surrounding clay.

Further attempts are made to calculate the lateral deformations and bulging of ESCs as a result of applied surcharge pressure. As such, lateral deformation and bulging are measured on both sides (L: left and R: Right) of ESC. Figures 9 and 10 show the profiles of lateral deformation and bulging variations along with the depth and with the position of ESCs from the centre to toe of the embankment (numbered from 1 to 10 as per Fig. 1a). The various results discussed on previous sections highlight that spacing up to 2 m does not fetch any advantage. Hence, results are presented for column spacing's of 2.5, 3.0, and 3.5 m only. It can be seen that the deformation is minimum at the bottom of ESCs, which is at a depth of 10 m from the ground surface. The maximum deformation occurred at a depth of 1.43 m (i.e.

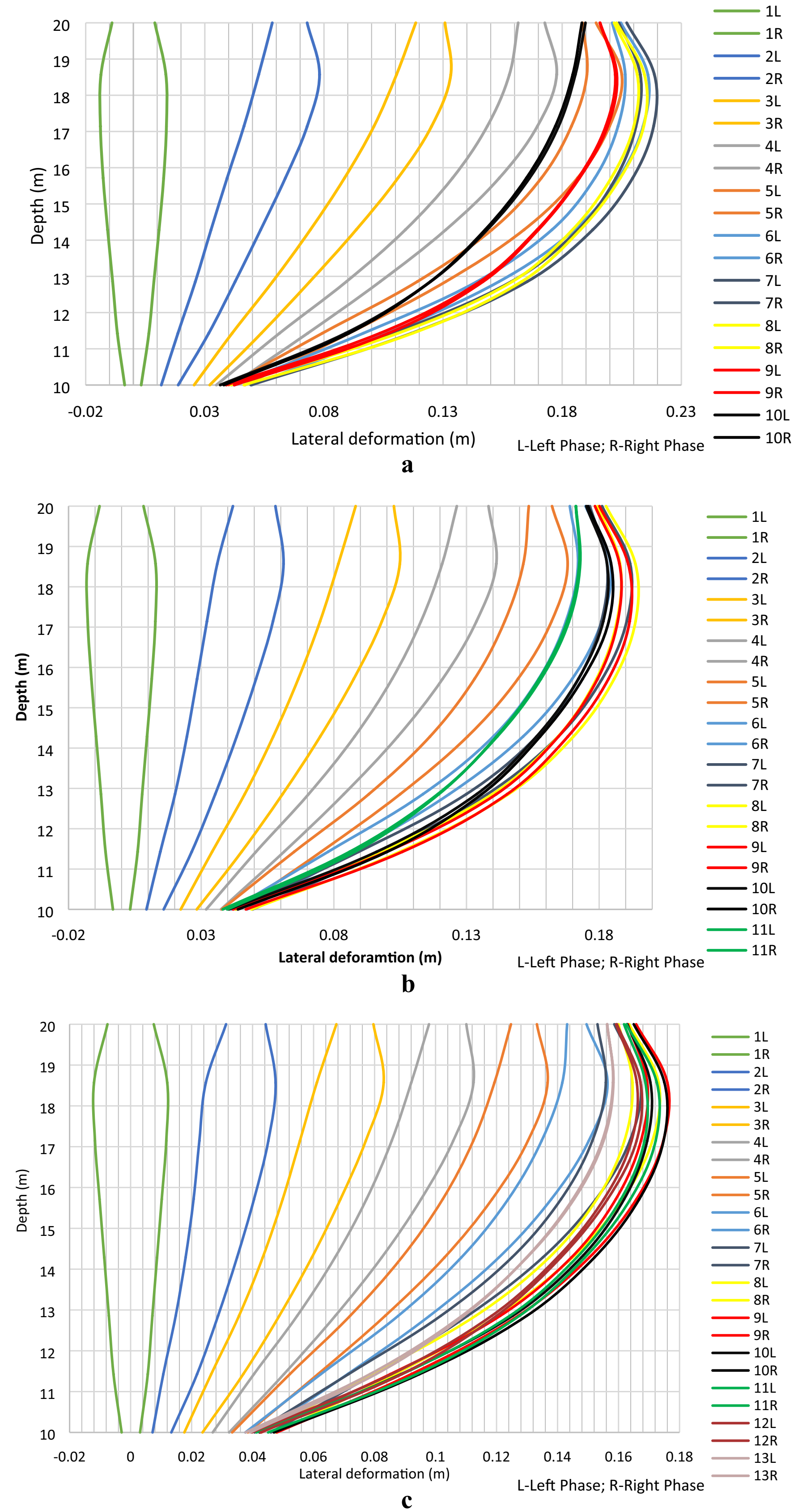
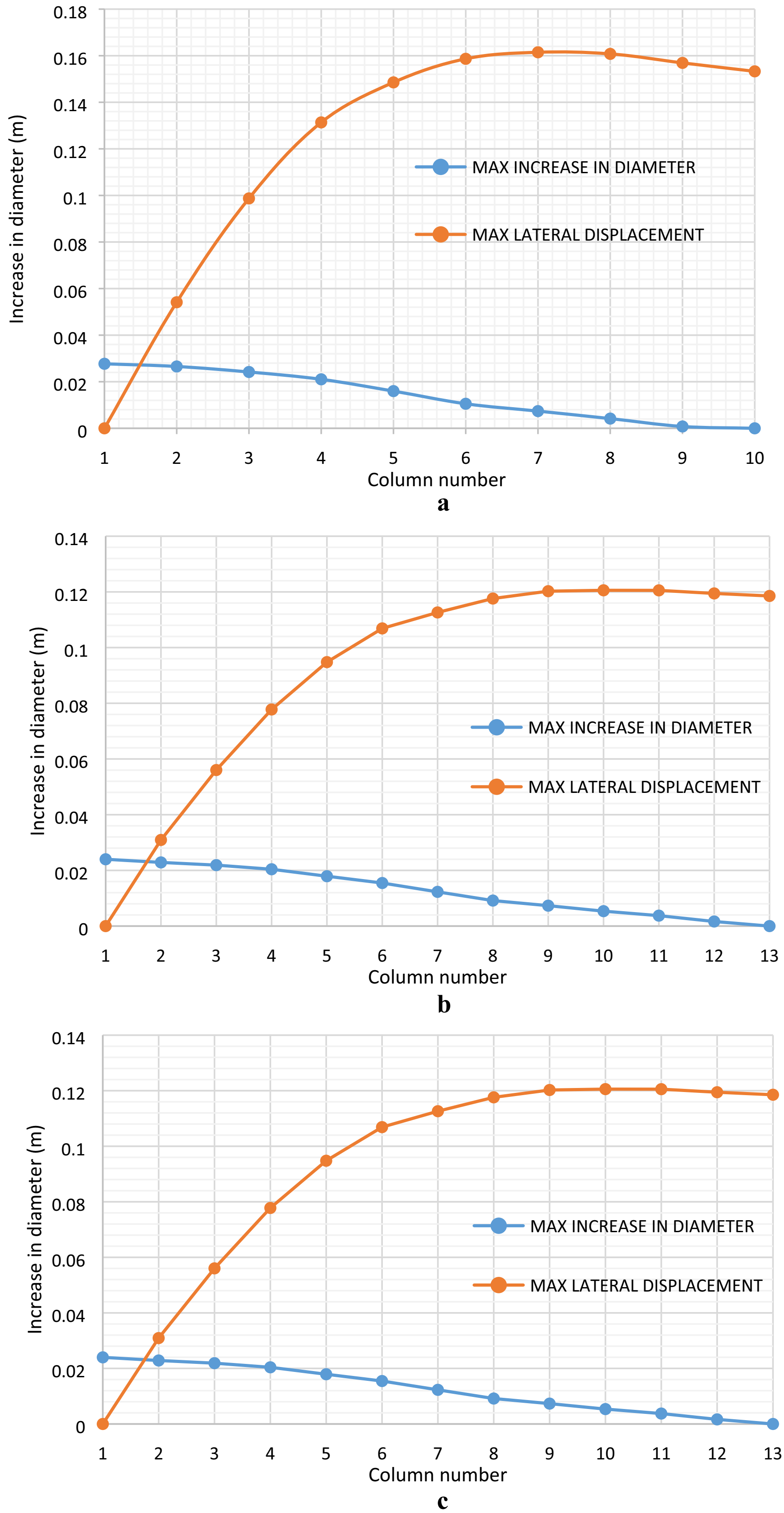


Fig. 9 **a** Profile of predicted lateral deformation and bulging of ESCs with depth for 3.5 m spacing. **b** Profile of predicted lateral deformation and bulging of ESCs with depth for 3.0 m spacing. **c** Profile of predicted lateral deformation and bulging of ESCs with depth for 2.5 m spacing

Fig. 10 **a** Comparison of maximum lateral deformation and bulging occurrence at different positions of ESCs for 3.5 m spacing. **b** Comparison of maximum lateral deformation and bulging occurrence at different positions of ESCs for 3.0 m spacing. **c** Comparison of maximum lateral deformation and bulging occurrence at different positions of ESCs for 2.5 m spacing



2.8 times of D) from the ground surface. Vividly, lateral deformations increased with the change in position of ESCs from the centre towards the toe of embankment. Figure 10 clearly substantiates the distinction between maximum lateral deformation and bulging exhibited by all the columns. It can be observed from figures that bulging decreases as the position of ESC shifts from the centre towards outer edge of the embankment. Bulging primarily occurs due to shear dilatancy in soils. As such, variations in the bulging and lateral deformation indicate a change in the shear dilatancy in soils depending upon the positon of ESC from centre to outer edge of the embankment.

Understandably, ESC-1 seemed to be predominantly subjected to axial load, as the occurrence of maximum bulging is measured on either side of this particular column. For remaining ESCs from 2 to 10, whose positions are distanced from ESC-1, lateral deformations appear to be prevalent over the bulging behaviour. A closer examination of Figs. 9 and 10 disclose a fact that ESC-7 has undergone maximum lateral deformation, while ESC-10 has exhibited the least bulging deformation. These observations draw the attention that the position of ESCs from 2 to 7 are critical, in a sense that these columns are offering maximum lateral resistance and thereby, impeding the failure against slope stability of the embankment.

As such, Figs. 9 and 10 bear very practical relevance in terms of stability against the base failure of the embankment. Demonstrably, ESCs by bending action and undergoing lateral deformation provide sufficient restraint against the base failure of the embankment. The interpretation of lateral deformation results reveals that maximum lateral deformation occurred when the degree of consolidation is less than 30%. It should be noted that, within the time frame of 30% consolidation, the majority of the load is borne by ESCs solely and there is not much development of shear strength within the surrounding soil. This clearly hypothesizes that the role of ESCs is critical at the end of construction or during the initial phases of post-construction.

Attempts further are made to understand the occurrence of maximum bulging, which is a principal cause for bearing resistance and its relation with degree of consolidation. Figure 11 shows the maximum bulging and degree of consolidation corresponding to the centrally placed ESC with spacing and time. It can be observed from the graph that the phenomenon of bulging started happening with the progression of surcharge load from the day one. On the other hand, improvement in degree of consolidation begins to happen at the end of surcharge load application i.e. 60 days. From the graph, dominant bulging occurrence measured at 85, 127 and 189 days for 2.5, 3 and 3.5 m spacings, respectively. Interestingly, these time periods are 1.4, 2, and 3 times the time needed for application of the surcharge load. The results presented in Fig. 11 clearly demonstrate that short-term time periods, especially, those corresponding to the end of construction are critical for bearing capacity and stability of the embankment.

Conclusions

In this paper, a case study on the feasibility of adopting geosynthetic-encased stone column as a ground improvement technique for a railway embankment built over the soft clay is investigated. The performance of the proposed ground improvement technique using 2D FE analysis is demonstrated by varying spacing between columns. Based on the analysis of results and interpretation of the same, the following conclusions are drawn:

- Based on the rigorous analytical simulations and prevailing site conditions, it is substantiated that geosynthetic-encased stone column is the best and economical ground

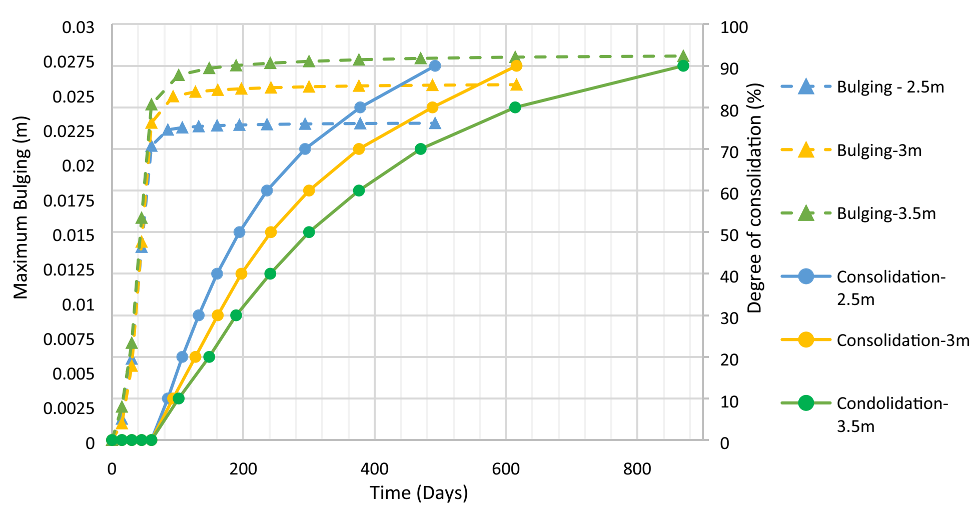


Fig. 11 Predicted maximum bulging and degree of consolidation with time and spacing for ESC-1

improvement technique for the case of an already failed railway embankment.

- The time required for attaining 9 0% consolidation for 1.5 and 3.5 m spacing is measured as 290 and 810 days excluding that the time required for construction.
- The results demonstrate a contrast of the impact of spacing on the degree of consolidation, excess PWP, and settlements. It is further noticed that 3.5 and 4 m spacings does not fetch much advantage to ESC when stability criteria is under consideration.
- The various results prove a fact that the evolution of FOS with time is primarily dependant on the improvement in the effective stress within ESC and in surrounding clay.
- A reversal in FOS values between end of the construction and 90% degree of consolidation time periods has been noticed from the interpretation of results. At the end of construction, closer spacing produced the highest FOS of 1.23, and at the end of consolidation that is in the long term, the highest spacing of 3.5 m yielded a maximum FOS of 2.14.
- The study finds that spacing of 3 m when alone given importance to bearing capacity and 2.5 m spacing considering lateral deflection alone is optimal. Optimum spacing of 2.5 m has produced acceptable least settlement, minimum lateral deflection, bulging, and maximum FOS of 2.14 at the end of 90% consolidation.
- The analyses of results revealed that both bearing capacity and stability are critical in the short-term. However, both evolved with the progression of degree of consolidation.
- It is found that the least FOS of 1.5 is obtained at 30% degree of consolidation corresponding to all spacing's, but the time required to achieve 30% degree of consolidation is different for different spacing's.
- Bulging and lateral deformation are identified as two crucially important behaviours of ESC. The role of the former factor is critical in bearing axial loads, while the latter factor prevalently counteracted the embankment failure against stability.
- The maximum bulging and lateral deformation is measured at a depth of 1.43 m (2.8 times of D) from the ground surface.
- It has been noticed that the position of column bears significance, specifically, when ESC is implemented in extremely soft soils as is dealt with in the present study. Demonstrably, columns located between the centre and toe of the embankment are critical, as they offered the greatest restraint against the failure of the embankment, and there is also a reversal in loading from axial to lateral with a shift in the position of the column from the centre towards toe of the embankment.

Author Contributions TDD, a PG student, and GB, a research scholar, are key persons in performing extensive analytical simulations and producing the necessary data for the manuscript. Co-author, SK is a key person in reviewing the relevant literature and preparing the introduction of the entire paper. The other co-author, SAKB, is an employee of RVNL, is a key person in providing the relevant field data, describing about site conditions, and producing photographs of failed the railways embankment. He is also one of the witnesses during the failure of the embankment and pivotal person in ascertaining the validity of the proposed ground improvement technique. Co-author, BHR provided the basic infrastructure facility including analytical support for successfully carrying out the investigations. The analysis and interpretation of data was also the core contribution by BHR and he has revised all the figures for enhancing their readability. All authors contributed for preparation of the manuscript and ensured the manuscript in correctness for English and grammar. All authors reviewed the manuscript for proof checking and editorial corrections.

Declarations

Conflict of interest The authors declare that they have no conflict of interest.

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