

Review of Existing Design Methods of Geosynthetic Reinforced Piled Embankments (GRPE)

H.D.S. Mithila and N.H. Priyankara

Abstract: Construction of highway embankments over soft compressible soil or weak foundation soil results in many issues such as intolerable settlement, potential bearing failure, and global and local instability. In order to overcome these problems, many ground improvement methods are in use such as preloading with or without vertical drains, gravel compaction piles, piled embankments, etc. However, among these ground improvement techniques, Geosynthetic Reinforced Piled Embankment (GRPE) stands out due to its efficient load transfer mechanism and less construction time. In terms of design of GRPE, various researchers have introduced different methods based on different load transfer mechanisms. As such, there is an uncertainty among design engineers regarding the applicability of these design methods. This paper investigated the load transfer mechanism of GRPE using finite element analyses and currently available design methods were compared with the results of finite element modelling. Design methods were compared using output parameters of total load on a pile and geosynthetic tension by varying input parameters pile width, surcharge, soft soil stiffness, geosynthetic stiffness and embankment height. Based on the finite element analyses, the inconsistencies in the currently available design methods were identified.

Keywords: Finite element modelling, Geosynthetic reinforced piled embankment, Ground improvement

1. Introduction

Embankments are man-made structures put in place to provide elevated platforms for roads, railways and runways which are integral elements of infrastructure development. With the intense rise of population, unoccupied good lands left for such development activities are quite low. Therefore, considerable attention has been paid on embankment construction on low-lying marshy lands with deep soft clay deposits which were previously labelled as unusable. These unfavourable ground conditions result in intolerable settlement, potential bearing failure and global and local instability.

In order to face these issues, various ground improvement techniques are in use such as preloading with or without Prefabricated Vertical Drains (PVD), granular columns, deep mixing techniques, piled embankments, etc. Among the above mentioned ground improvement techniques, providing pile supports stand out since all other methods are consolidation based methods which require excessive waiting time prior to construction of the superstructure. In contrast, pile supported embankments reduce settlement by transferring the embankment load to an underlying hard stratum. Therefore, less construction time is a

major advantage of pile supported embankments [1].

Researchers found that adding a single or multiple geosynthetic reinforcements on top of the piles contributes to increase the load transfer to piles and thus the pile cap size and area replacement ratio can be reduced [2], where area replacement ratio denotes percentage coverage of pile cap area over total foundation area tributary by a pile. Also, in a traditional piled embankment system, corner piles are placed as raked piles to resist lateral thrust induced by horizontal earth pressure. The inclusion of geosynthetic reinforcement at the base of the embankment resists the lateral thrust and avoids horizontal displacement within the structure, thereby eliminating the requirement of raking piles [3]. Even though very soft soil exists below the geosynthetic

Eng. H.D.S. Mithila, AMIE(SL), B.Sc. Eng. (Hons) (Ruhuna), Department of Civil and Environmental Engineering, University of Ruhuna.
Email: mithilahds@gmail.com

 <http://orcid.org/0000-0003-3994-0481>

Eng. (Dr.) N.H. Priyankara, C.Eng, MIE(SL), B.Sc.Eng.(Hons) (Moratuwa), M.Eng.(AIT), Ph.D.(Tohoku), MSLGS, Senior Lecturer, Department of Civil and Environmental Engineering, University of Ruhuna.
Email: nadeej@cee.ruh.ac.lk

 <http://orcid.org/0000-0002-8775-7246>

reinforced piled embankment, only minor differential settlement is expected between top of pile cap and fill material above the clear span between pile caps [3].

Since, piles are stiffer in comparison to soft soil around them, a significant portion of the embankment load, dead and live load of pavement, is transferred to piles and causes only minor differential settlement. Hence, shear stress is mobilized within embankment fill resulting in directly transferring vertical load from embankment fill to pile caps. This mechanism is known as soil arching [4] as illustrated in Figure 1. Also, due to the inclusion of geosynthetic reinforcement, some portion of vertical load is mobilized as tension in the membrane and its vertical component is indirectly transferred to piles. Therefore, total load on pile is the combination of directly transferred load through soil arching and indirectly transferred load through geosynthetic tension [5].

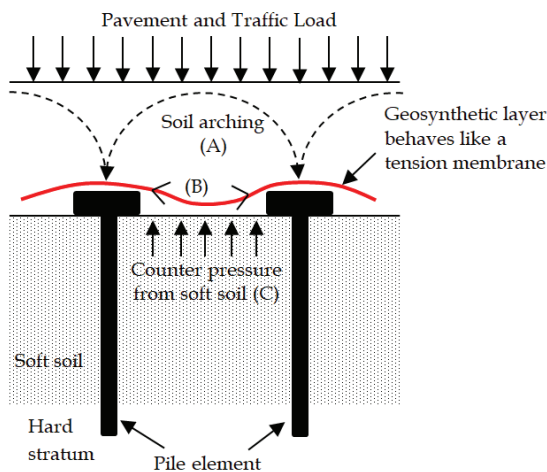


Figure 1 - Load Transfer Mechanism of GRPE

2. Design Philosophies on Piled Embankment Design

Several methods can be found in literature to obtain vertical load distribution in GRPE. Majority of the existing design methods assume that the embankment load is transferred to piles by soil arching mechanism introduced by Terzaghi [6]. Guido et al. [7] developed a design approach where arches occur in triangular formation with 45° internal angle. Hewlett & Randolph [8] presented a semi-spherical arching model to evaluate the load transfer. In this model, critical locations for failure would be at the crown of the semi-sphere or at pile caps. Anyway, this method neglects the existence of geosynthetic

reinforcement. Rogbeck et al. [9] introduced the Swedish design methodology in which triangular shaped arches were idealized with 30° angle at the tip of the slice in the soil under two dimensional plain strain conditions. It assumes that weight above the wedges is directly transferred to piles. Collin et al. [10] presented an improved version of that proposed by Guido et al. [7]. In this method, it is advised to use multiple layers of reinforcement to create a stiff platform of reinforced soil. Reinforced soil mass which consists of multiple geotextiles acts as a beam and transfers the embankment load to underlying piles. Kempfert et al. [11] developed a new design method after performing laboratory model tests of piled embankments. According to the above recommendations, initially the load induced on soft soil without geosynthetic reinforcement was calculated which was followed by estimating the tension to be developed in geosynthetic reinforcement to carry that calculated load. Based on above findings, more comprehensive design codes were developed by British and German genre ([12], [13]).

2.1 British Approach - BS 8006 (1995)

The Code of practice for Strengthened /reinforced soils and other fills, also known as BS 8006, recommends to use only one layer of reinforcement, but if total required strength cannot be provided by single layer use of multiple layers are allowed. This method accounts for process of arch development from the embankment construction perspective. Different arching conditions occur, when

- $0.7(s - a) < H < 1.4(s - a)$ - partial arching
- $H > 1.4(s - a)$ - full arching. This is due to the development of the complete soil arch within embankment layers (Figure 2)

where, H is embankment height, s is pile spacing and a is pile width.

Code suggests two expressions to calculate line load on geosynthetic reinforcement based on embankment height (Refer section 4.1). It can be noted that, stiffness of geosynthetic layer is neglected in BS 8006 method when developing stress-strain relationship.

Also, BS 8006 method considers that foundation support provided by soft soil cannot be relied upon, therefore it is excluded from calculating net vertical loading acting on reinforcement. Therefore, it provides conservative design outputs.

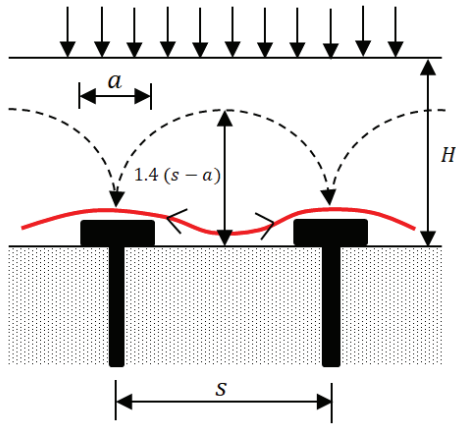


Figure 2 - Soil Arching Mechanism at Full Arching According to BS 8006

2.2 German Approach - EBGeo (2004)

Recommendations for Design and Analysis of Earth Structures using Geosynthetic Reinforcements, known as EBGeo, was introduced by German Geotechnical Society. It adopts multi-shell arching theory proposed by Kemfert et al. [14]. Unlike many other design methods, EBGeo considers foundation support beneath the geosynthetic reinforcement to be substantial and incorporates modulus of subgrade reaction (K_s) in load calculation. Therefore, a graphical solution has been developed by EBGeo which provides reinforcement strain (ϵ) based on K_s (Refer section 4.2), thus having soft soil with higher K_s resulting in lower values for reinforcement strain and tension.

Although, various design approaches are available for designing GRPE, a very few

attempts have been made by researchers to compare these methods to identify their inconsistencies [15], [16] and [17]. Therefore, a universally accepted method is yet to be introduced. This paper presents a parametric study involving BS 8006, EBGeo and PLAXIS-2D finite element analysis.

3. Methodology

Typical GRPE geometry used for the numerical modelling is shown in Figure 3. The soil profile at ground surface consists of a 8.0 m thick peat layer followed by a 3.0 m thick medium dense sand layer. Underlying the medium dense sand layer, there is a 3.0 m thick dense sand layer. The ground water table was located at the ground surface. The crest width of the embankment is 16.0 m and side slopes are 1:1.25 (vertical: horizontal).

The embankment is supported by precast driven square piles with 300 mm size and an embedded length of 10.0 m. Piles act as friction piles since they are not installed down to the bed rock. Piles are placed at 1.5 m centre to centre spacing. Geosynthetic layer was placed in between embankment soil and soft soil layer to increase the efficiency of load transfer mechanism. Embankment was constructed over a period of 28 days. Finally, embankment was left for 365 days to consolidate.

3.1 Numerical Study

A numerical study was performed with finite element program PLAXIS 2D under two dimensional plain strain condition. Very soft

Table 1 - Material Properties of GRPE

Parameter	Embankment soil	Peat	Sand	Pile
Constitutive model	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Linear-Elastic
Material behaviour	Drained	Undrained	Drained	Undrained
Unit weight above phreatic level (kN/m^3)	18	12	16.5	24
Unit weight below phreatic level (kN/m^3)	20	14	20	24
Horizontal permeability (m/d)	1	0.0001	1	-
Vertical permeability (m/d)	1	0.0001	1	-
Material stiffness E_{ref} (kN/m^2)	22000	3400	18000	2.8×10^7
Poisson's ratio (μ)	0.3	0.33	0.3	0.1
Cohesion c' (kN/m^2)	5	1	1	-
Drained Friction Angle ϕ ($^\circ$)	30	20	32	-

peat layer, embankment soil and sand layer were modelled with Mohr-Coulomb constitutive model while pile and geosynthetic layer were modelled as linear elastic material as shown in Table 1.

length in out of plane direction as shown in Equations (1) and (2) [18].

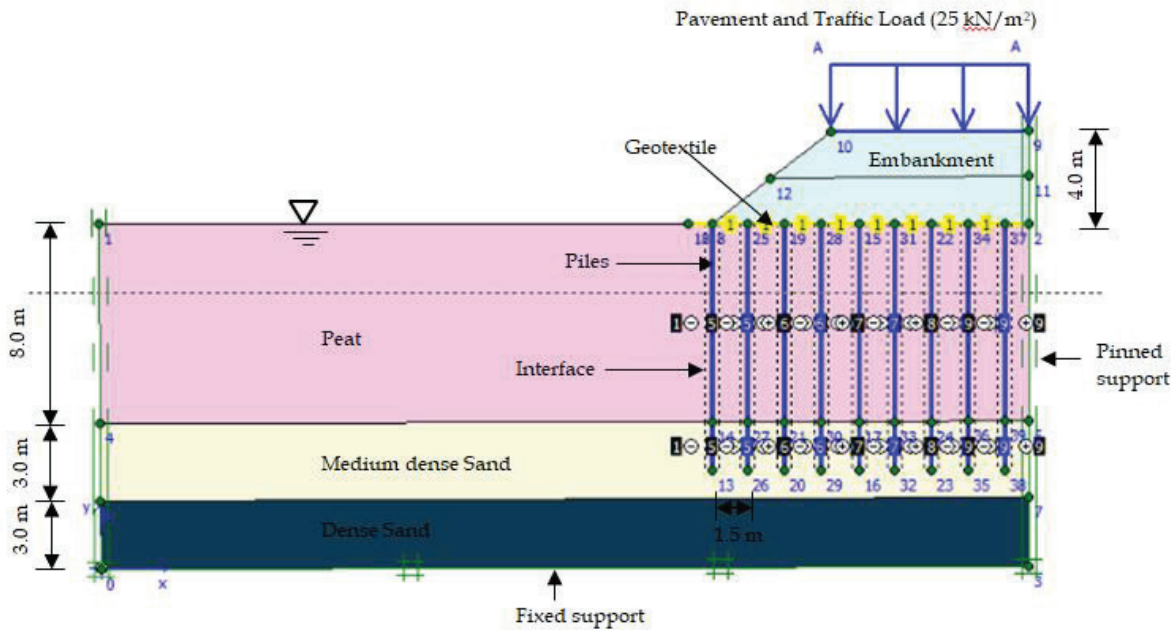


Figure 3 – Plain Strain Model of GRPE in PLAXIS 2D

In terms of boundary parameters, the x axis is taken as horizontal, y axis considered to be vertical, and out of plane direction taken as z axis. Due to the symmetry of the embankment along the centreline (y axis), only one half of the GRPE was modelled under plain strain conditions as illustrated in Figure 3. Piles were modelled as “plate” geometry component where flexural rigidity (EI) and axial stiffness (EA) can be attributed for the analysis. Also, geosynthetic reinforcement was modelled using “geogrid” geometry component with only input parameter being the normal stiffness since it doesn’t have bending stiffness. Therefore, plate and geogrid were the ideal geometry components to realistically represent the piles and geosynthetic reinforcement. Also, interface was modelled to simulate the pile-soil interaction where no slip condition was assumed. The displacement of the embankment is limited to bottom boundary since standard fixities are applied to GRPE boundaries.

When piles are modelled as plates under plain strain conditions, they are idealized as walls spread out in the out of plane direction with infinite length. So, when inputting pile parameters (EA and EI), they should be adjusted to equivalent properties with unit

$$E^{\wedge} = E \times \frac{A_p}{h_{eq} \times L_{spacing}} \quad (kN/m/m) \quad \dots(1)$$

where,

Equivalent axial stiffness = $E^{\wedge} \cdot A$

Equivalent flexural rigidity = $E^{\wedge} \cdot I$

$$\gamma^{\wedge} = \gamma \times \frac{A}{L_{spacing}} \quad (kN/m/m) \quad \dots(2)$$

where,

E = Stiffness of pile material

$E^{\wedge}$ = Equivalent stiffness of pile material

γ = Unit of weight of pile material

$\gamma^{\wedge}$ = Equivalent unit weight of pile material

$h_{eq} = \sqrt{\frac{12I}{A}}$ (m) = Equivalent thickness of a plate

$L_{spacing}$ = Out of plane centre to centre pile spacing (m)

A_p = Cross sectional area of actual pile

A = Cross sectional area per unit width in out of plane direction of pile modelled as wall

I = Moment of inertia per unit width in out of plane direction of pile modelled as wall

Actual pile and its model configuration is illustrated in Figure 4 and model equations are shown in Equations (3) to (5).

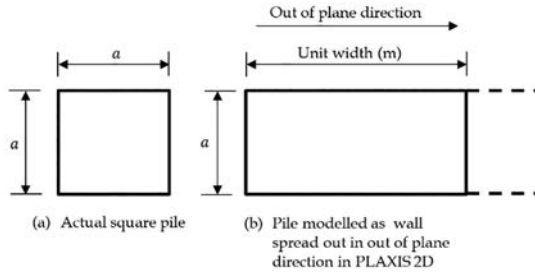


Figure 4 - Actual Pile and its Model Configuration

$$A_p = a \times a \text{ (m}^2\text{)} \quad \dots(3)$$

$$A = 1 \times a \text{ (m}^2\text{/m)} \quad \dots(4)$$

$$I = \frac{1}{12} \times a^3 \text{ (m}^3\text{/m)} \quad \dots(5)$$

When the geometry model is fully defined and material properties are assigned to clusters and structural objects, an initial mesh with 15 noded biquadratic elements was generated where the geometry is divided into finite elements as depicted in Figure 5. There are 482 elements and 4739 nodes in the model after meshing. The average element size is 1.19 m. A medium size mesh was used as global coarseness in the analysis, to reduce the processing time. However, local refining was done near the piles to get more accurate results.

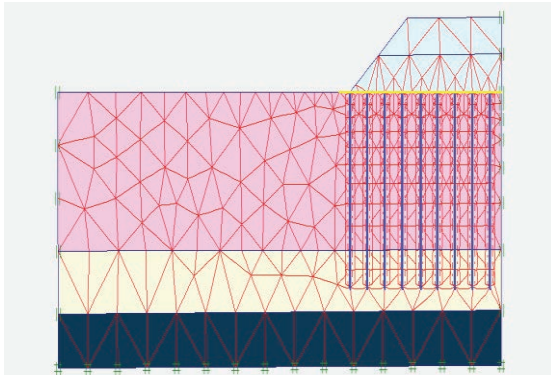


Figure 5 - Initial Mesh Generation

Then, embankment layers, piles and geogrid were deactivated to represent the initial state of GRPE followed by generating initial stress (K_0 procedure) and initial pore water pressures after defining water level. The next step of analysis was the staged construction procedure where the realistic process of construction was simulated. The phases of construction are as follows:

- Phase 1: Piles were installed in unimproved ground and geogrid was constructed on it.

- Phase 2: Application of embankment layer 1 (plastic state)
- Phase 3: Consolidation of embankment layer 1 (consolidation 14 days)
- Phase 4: Application of embankment layer 2 (plastic state)
- Phase 5: Consolidation of embankment layer 2 (consolidation 14 days)
- Phase 6: Simulation of road opening to traffic by applying surcharge (consolidation 365 days)

The deformed mesh obtained after loading application is illustrated in Figure 6.

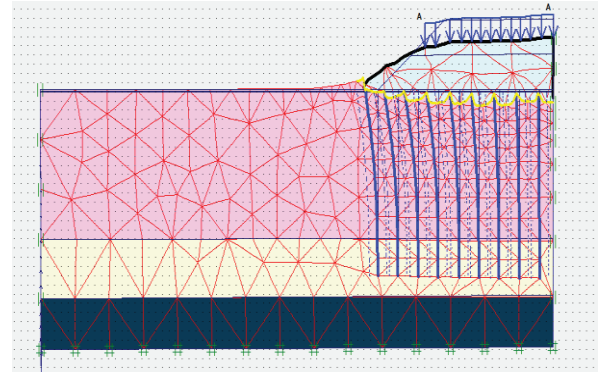


Figure 6 - Deformed Mesh Generated after Loading

3.2 Parametric Study

Parametric study was incorporated to compare the existing design methods. Among the existing analytical design methods, BS 8006 and EBGeo were chosen for the parametric study since they are the most widely used design methods. Once the numerical analysis was performed for initial geometry, it was further extended for different values in peat layer stiffness, traffic load, geogrid layer stiffness, embankment height and pile width. Similarly, an analytical study was performed while varying input parameters mentioned above. This will result in understanding why the results from different design methods vary from each other. Only one parameter was changed at a time, while the others were kept at their baseline case values as summarized in Table 2. Based on parametric study using PLAXIS 2D, BS 8006 and EBGeo, total load transferred to pile and geosynthetic tension were observed as output parameters.

4. Results and Discussion

The results were analysed with respect to the total load on the pile and geosynthetic tension.

4.1 BS 8006 Method

Case 01: Total load on pile

$$\text{Total stress on pile top } (\sigma_v) = \gamma H + P \quad \dots(6)$$

where,

γ = Unit weight of embankment soil

H = Embankment height

P = Traffic load (Surcharge)

Using Marston formula,

$$P_c = \left(\frac{C_c a}{H} \right)^2 \times \sigma_v \quad \dots(7)$$

where,

P_c = Direct vertical stress induced on piles due to arching

C_c = Arching coefficient

a = Pile width

It is noted that BS8006 method neglects the support from soft soil underneath. It considers that the total load induced on equivalent pile grid (Figure 7) is ultimately transferred to the pile. Therefore,

Total load on pile = $\sigma_v \times$ individual pile grid area (s^2)

where,

s = centre to centre pile spacing

Case 02: Geosynthetic tension

Equations (8) and (9) are used to obtain the line load on the strips of reinforcement between adjacent piles (W_T) and Equation (10) is used to obtain tensile stress in reinforcement (T_{rp}) where ξ is geosynthetic strain.

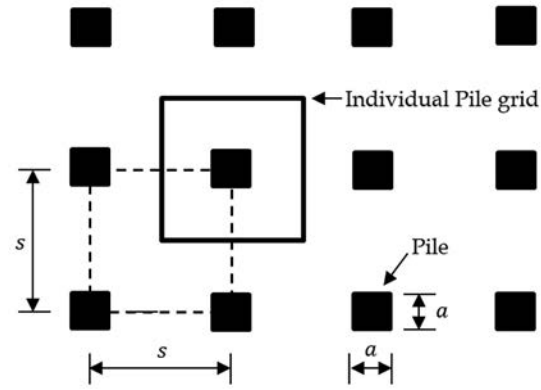


Figure 7 - Typical Pile Grid in GRPE

Full arching

$$W_T = \frac{1.4 \gamma (s-a)}{s^2 - a^2} \times \left[s^2 - a^2 \left(\frac{P_c}{\sigma_v} \right) \right] \quad \dots(8)$$

Partial arching

$$W_T = \frac{s \cdot \sigma_v}{s^2 - a^2} \times \left[s^2 - a^2 \left(\frac{P_c}{\sigma_v} \right) \right] \quad \dots(9)$$

$$T_{rp} = \frac{W_T x(s-a)}{2a} \times \sqrt{1 + \frac{1}{6\xi}} \quad \dots(10)$$

ξ = Estimated strain in geogrid

4.2 EBGEO Method

Case 01 : Total load on pile

Figure 8 is used to obtain the normal stress developed on geosynthetic between piles ($\sigma_{z0,k}$) due to embankment loading condition [13].

Table 2 - Input Parameters Used for the Analysis

Variable parameter	Pile width(m)	Pile spacing (m)	Embankment height (m)	Peat layer stiffness (kN/m ²)	Pavement+Traffic load(kN/m ²)	Geogrid layer stiffness (kN/m ²)
Peat layer stiffness	0.3	1.5	4.0	800 1000 1500 3400	10	1500
Traffic load	0.3	1.5	4.0	3400	0 10 20 30	1500
Geogrid layer stiffness	0.3	1.5	4.0	3400	10	1500 3000 4000
Embankment height	0.3	1.5	0.5 1.0 1.5 3.0 4.0	3400	10	1500
Pile width	0.3 0.4 0.5	1.5	4.0	3400	10	1500

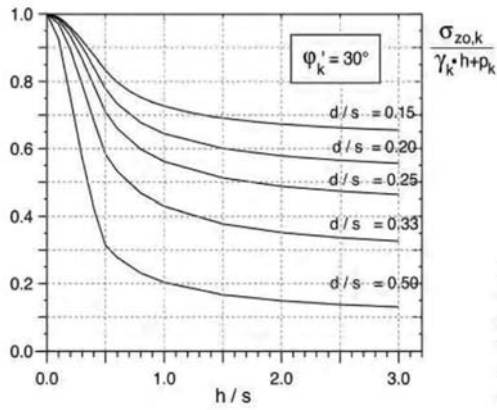


Figure 8 - Graph for Obtaining Normal Stress ($\sigma_{z0,k}$) between Piles(after [13])

where,

h = Embankment height

s = Maximum spacing between piles

d = Equivalent pile width

Equation (11) is used to obtain the stress developed on piles due to arching ($\sigma_{zs,G+Q,k}$).

$$\sigma_{zs,G+Q,k} = ((\gamma h + P_k) - \sigma_{zs,G+Q,k}) \times \left(\frac{A_E}{A_S}\right) + \sigma_{z0,G+Q,k} \quad \dots(11)$$

where,

A_E = Equivalent area for single pile grid

A_S = Cross sectional area of single pile

Then, direct load on piles due to arching

$$= \sigma_{zs,G+Q,k} \times A_S$$

Case 02: Geosynthetic tension

Unlike in BS 8006, EBGeo integrates the support from soft subsoil into calculation of geogrid tension by introducing a parameter, namely, subgrade reaction, as shown in Equation (12).

$$K_{s,k} = \frac{E_{s1} \times E_{s2}}{E_{s1} \times t_1 + E_{s2} \times t_2} \quad \dots (12)$$

where,

$K_{s,k}$ = Mean modulus of subgrade reaction

E_s = Constrained modulus of stratum

t_w = Thickness of considered stratum

w = Layer 1 and 2

Figure 9 is used to calculate the maximum strain developed in the geogrid.

where,

J_k = Tensile stiffness of geogrid

L_w = Clear distance between two piles

b_{ERS} = Pile width

F_k = Resultant vertical strip load on geogrid

ξ_k = Maximum strain in geogrid

Ultimately, from Equation (13), mobilized geosynthetic tension on geogrid ($E_{M,K}$) can be obtained.

$$E_{M,K} = \xi_k \times J_k \quad \dots(13)$$

4.3 Comparison of Different Design Philosophies

4.3.1 Effect of Pile Width on Total Load on a Single Pile

The variation of total load transferred to a single pile with respect to pile width is illustrated in Figure 10. It can be seen that, in terms of BS 8006, total load on a pile is independent from the pile width where load on the pile is not varied with the increase in pile width. However, PLAXIS 2D results illustrate that pile width is an influential parameter in determining total load on a pile. When pile width is increased, total load on a pile gradually increases too. EBGeo follows the same theory and it is clear that its values are quite close to that of PLAXIS 2D. When pile width is increased, composite stiffness of entire system is increased and the stress concentration ratio on piles is increased. Therefore, total load transferred to piles is increased while load transferred to soft soil is decreased. Since BS 8006 neglects the effect of soft soil this variation cannot be captured.

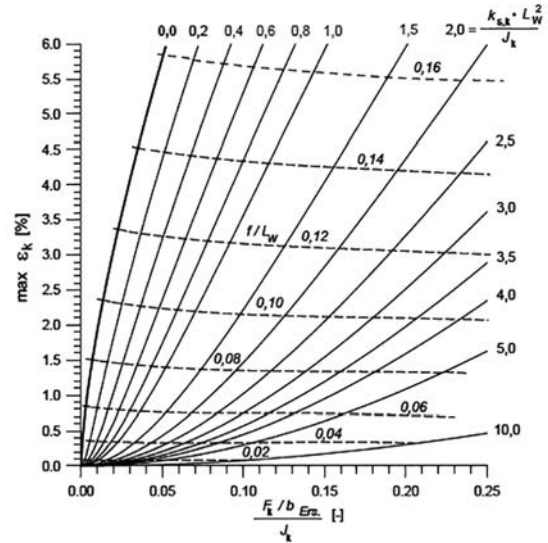


Figure 9 - Maximum Strain in Geogrid (after[13])

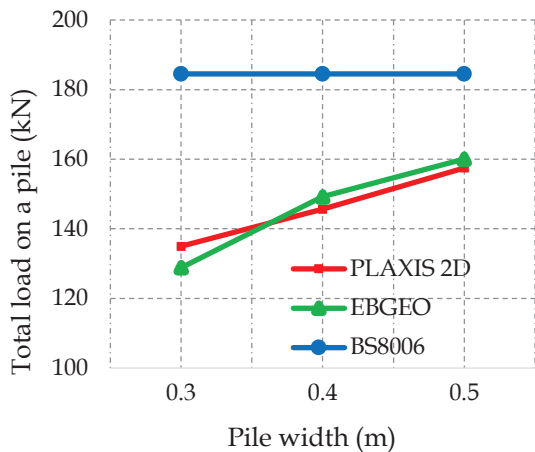


Figure 10 - Variation of Total Load Transferred to a Single Pile Over Pile Width

4.3.2 Effect of Surcharge on Total Load on a Single Pile

The variation of total load transferred to a single pile with respect to surcharge is depicted in Figure 11. It is clearly visible that BS 8006 overestimates the total load on a pile when compared with that of PLAXIS 2D results. When considering load equilibrium, total load of surcharge and soil weight induced on influence area of a single pile should be carried by the load transferred to a pile (A+B) and load carried by soft subsoil (C) as shown in Figure 1, where

- A - Direct load on pile due to arching,
- B - Indirect load transferred via geogrid to pile and
- C - load transferred to soft soil.

Since BS 8006 assumes a void below geogrid and neglects the support from soft soil *i.e.* $C = 0$, it considers that total top load ultimately is transferred to the piles which provides highly conservative values for pile design. Ultimately, EBGEO results are quite similar to PLAXIS 2D results since EBGEO considers the support from soft subsoil.

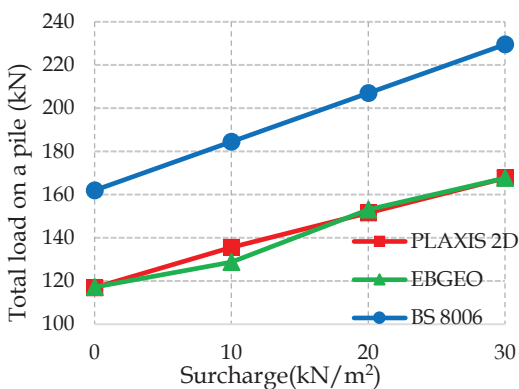


Figure 11 - Variation of Total Load Transferred to a Single Pile Over Surcharge

4.3.3 Effect of Peat Layer Stiffness on Geogrid Tension

Influence of stiffness of soft soil layer directly below geogrid on geogrid tension was studied as shown in Figure 12. BS 8006 neglects the effect of soil layer directly below the geogrid. Therefore, its predicted values are independent from stiffness of soil layer directly below the geogrid. However, when observing the PLAXIS 2D and EBGEO results, it is clear that there is a considerable effect from its stiffness. When peat layer's stiffness is increased, the mobilized tension in geogrid is decreased. It is mainly because, when peat layer gets stiffer a higher portion of embankment load is transferred to it, which reduces the indirectly transferred embankment load to geogrid. Therefore geogrid tension is decreased with increasing peat layer stiffness, as illustrated by PLAXIS 2D and EBGEO outputs.

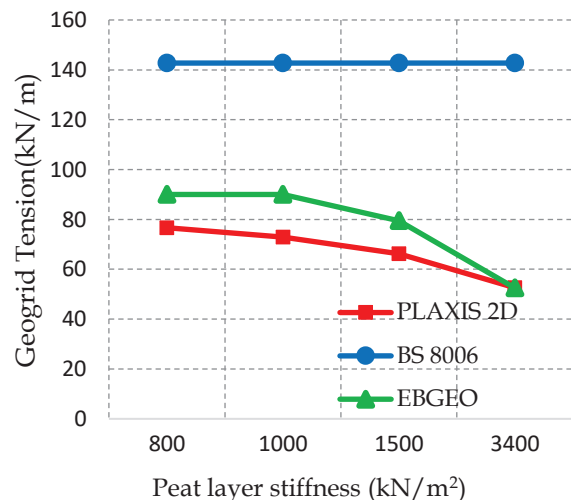


Figure 12 - Variation of Geogrid Tension with Peat Layer Stiffness

4.3.4 Effect of Geogrid Stiffness on Geogrid Tension

The effect of geogrid stiffness on geogrid tension is illustrated in Figure 13. It can be seen that BS 8006 predicts much higher values for geogrid tension due to dual application of load on geogrid [19]. This implies that BS 8006 predicts much higher values for geogrid tension compared to numerical analysis (PLAXIS 2D) as BS 8006 neglects the effect from subsoil below the geogrid and considers a void below the geogrid. On the other hand, EBGEO considers the support from soft soil by incorporating subgrade modulus and its predicted values are quite similar to that of PLAXIS 2D.

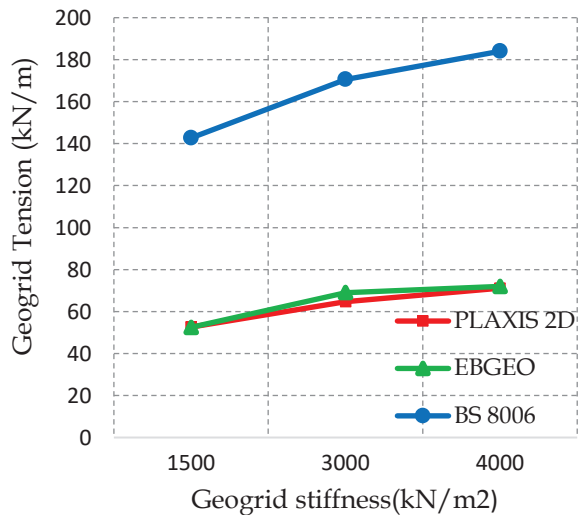


Figure 13 - Variation of Geogrid Tension with Geogrid Stiffness

4.3.5 Effect of surcharge on geogrid tension

Influence of surcharge on geogrid tension is shown in Figure 14. According to BS 8006, full arching occurs when embankment height $H > 1.4x(s - a)$. When full arching occurs, BS 8006 assumes that both the weight of soil above critical height $[H_e = 1.4x(s - a)]$ and surcharge are completely transferred to pile, and load transferred to geogrid remains constant even with increasing surcharge. However, in contrast to BS 8006, PLAXIS 2D clearly shows an increment in geogrid tension with increasing surcharge, which is closely followed by EBGEO results.

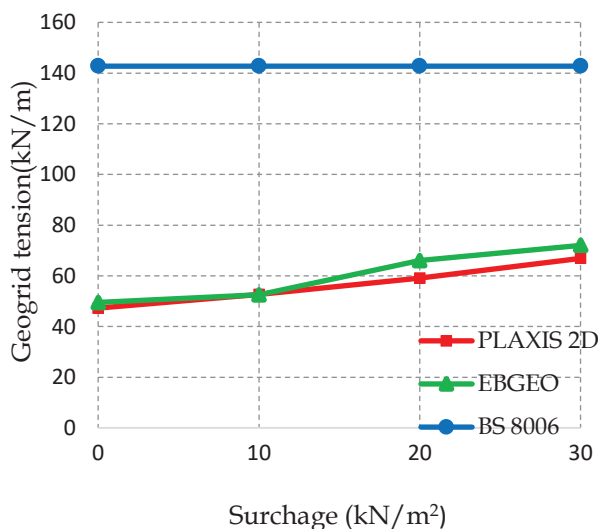


Figure 14 - Variation of Geogrid Tension with Surcharge

4.3.6 Effect of Embankment Height on Geogrid Tension

Variation of geogrid tension over embankment height is shown in Figure 15. It can be seen that geogrid tension under BS 8006 is

considerably higher than that of PLAXIS 2D and EBGEO, because BS 8006 method neglects the support from soft subsoil. Also, BS 8006 depicts a quick change when embankment height is between 1.5 m and 2.0 m. It is due to transition from partial arching to full arching, which occurs when

$$H = 1.4(s - a) = 1.4(1.5 - 0.3) = 1.68 \text{ m.}$$

After full arching occurs, geogrid tension remains constant even when increasing embankment height since top load beyond the critical height is directly transmitted to pile by arching and no effect on geogrid. However, PLAXIS 2D shows constant increment in tension when increasing embankment height, which is closely followed by EBGEO.

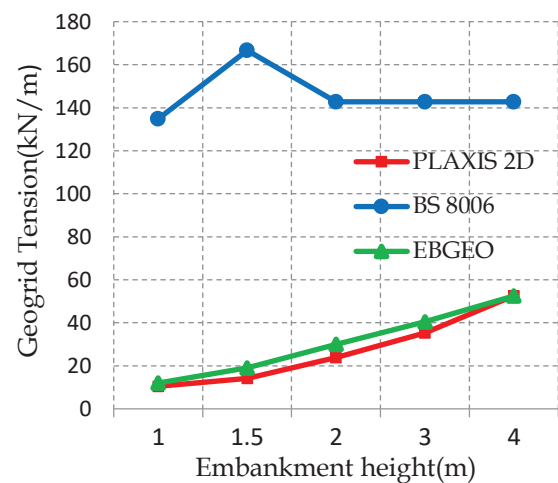


Figure 15 - Variation of Geogrid Tension with Embankment Height

5. Conclusions

This study presented a review of existing design methods and then it was extended to perform a parametric study using most widely used analytical design methods BS 8006, EBGEO and PLAXIS 2D finite element analysis.

The above two design philosophies were compared in terms of total load on a single pile and geogrid tension with respect to pile width, soft soil stiffness, surcharge load, embankment height and geogrid stiffness.

Results clearly showed that BS 8006 is much more conservative than EBGEO method. Stiffness of soft soil layer directly below the geogrid was found to be an influential parameter in designing of GRPE which is neglected by BS 8006. It assumes total embankment load is ultimately transferred to

piles due to negligence of support from soft soil. Furthermore, behaviour of geosynthetic tension in design of BS 8006 at the transition between partial arching and full arching is inexplicable. The parametric study clearly depicted that EBGeo complies considerably well with PLAXIS 2D results.

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