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Numerical Modelling of Gravel Compaction Piles in Soft Peaty Clay

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Abstract: Soft soil has become a major challenge in expressway construction and Gravel Compaction Piles (GCP) have been used as one of the most common ground improvement techniques to overcome this problem. However, the knowledge on the behaviour of GCP improved ground under drain condition remains limited. Hence, the most effective way to study the behaviour of GCP improved ground is through numerical modelling. Accurate modelling of the existing ground conditions is critical for precise analysis, even though replicating the exact ground conditions is not entirely feasible. Based on the simplification and time consumption, PLAXIS-2D software has been used in this study for numerical modelling. In order to convert the 3D actual scenario into 2D condition, two types of conversion methods, namely Geometry conversion and Parameter conversion methods, have been used. The accuracy of above conversion methods was verified by comparing the numerical model results with field data collected from expressway projects in Sri Lanka. A detailed analysis of settlement and pore water pressure behaviour was used to identify the most suitable conversion method for local context. Based on the results it can be concluded that the parameter conversion method is the most effective approach for accurately representing the behaviour of GCP improved ground in numerical modelling.

Keywords: Geometry Conversion, Gravel Compaction Pile, Parameter Conversion, Soft Soil

1. Introduction

Approximately 4.5% of the Earth's land surface is covered with peat [1]. Peat is an organic soil characterized by a water content exceeding 200%, low undrained shear strength (typically less than 15 kPa), a high void ratio (ranging from 2 to 8), and high compressibility. These properties classify peat as one of the principal soft soil types globally [2-7]. Due to these characteristics, peat soils are generally considered unsuitable for construction. Structures built on such soils are at risk of uneven settlement or even collapse if their foundations rest on such unfavourable subsurface conditions. These limitations have historically influenced construction decisions in regions with peat and similar soft soils.

In the past, construction projects typically avoided areas with soft soils. However, rapid urbanization and global development have led to a scarcity of land with favourable geotechnical conditions. As a result, civil engineers are increasingly required to design and construct buildings and infrastructure on marginal or problematic ground, including peatlands, to meet growing human and societal needs.

This global challenge is also evident in Sri Lanka, where approximately 2,500 hectares of land are classified as peatlands [8-9], with a significant portion located along the island's western coast. This region, which includes the capital city and major business cities, has experienced rapid infrastructure development over the past decade. As a result, identifying land with suitable subsoil conditions has become one of the major challenges currently facing the construction industry in this economically vital area.

To address these geotechnical challenges in Sri Lanka and similar regions, engineers have adopted two different techniques as illustrated in Figure 1. The first method is transferring the heavy superstructure loads of multi storey buildings and bridges etc. to the underlying hard stratum through piles [Figure 1(a)]. However, pile foundations are not an economical form of foundations when the roads

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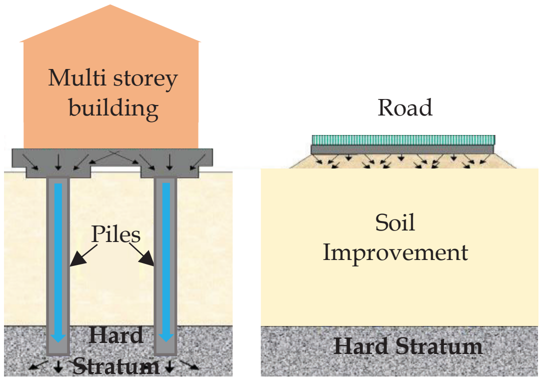
and service lines, which occupy a large plan area and moderately loaded structures are to be constructed. In such situations, the second method, the use of shallow foundations after soft ground improvement, will be a more economical option [Figure 1(b)].

Various ground improvement techniques are available to reduce the compressibility and enhance the shear strength characteristics of soft soils. Among them, Gravel Compaction Piles (GCPs) are considered one of the most effective and economical methods, attracting significant attention from geotechnical engineers worldwide [10-11]. GCPs are constructed by driving a casing into the ground, filling it with gravel, and compacting the material to form a dense column. Common installation methods include vibro-replacement, driving with displacement mandrels, or using vibratory hammers. The post-construction performance of GCPs is highly dependent on the installation method, as it influences stiffness, drainage, and load transfer characteristics [11].

In Sri Lanka, this technique has been successfully implemented in several major infrastructure projects, including the Colombo–Katunayake Expressway (CKE) project (2009–2013) and the Outer Circular Highway (OCH) project (2014–2019). However, the application of GCPs failed in the trial section of the Southern Expressway Extension Project from Matara to Beliatta (2015–2020), resulting in significant financial losses. Investigations identified the primary technical causes of failure as inadequate site investigations, improper GCP spacing, and limited understanding of GCP behaviour in thick, soft soil layers [12].

The CKE and OCH traverse marshy regions such as the Kelani River floodplain, Peliyagoda, and Muthurajawela, while the Southern Expressway Extension Project crosses Nilwala floodplain. These areas are all characterized by very soft peat deposits [3].

Both the successful and failed projects were constructed on improved soft soils using GCPs as the primary ground improvement method in certain sections of the roadways. However, the failure of the most recent project, despite prior experience, highlights the challenges and limitations associated with the use of GCPs in complex subsurface conditions. As noted by Nekasiny et al. [12], uncertainties remain regarding the applicability and reliability of GCPs under varying site conditions. Therefore, it can be concluded that the performance of GCPs in Sri Lanka's soft soils requires further evaluation under specific geotechnical conditions.



(a) Transfer superstructure loads to underlying hard stratum through piles

(b) Soft ground improvement

Figure 1 - Different load transfer mechanisms

Modelling GCPs prior to implementation can provide reliable predictions of their behaviour in the field. It enables engineers to estimate settlement, pore pressure development, and potential failure mechanisms in complex soil–structure interaction factors that are often not fully captured by empirical methods. Such predictive modelling can help prevent failures like those observed in the Southern Expressway Extension Project. However, for these models to be effective, they must accurately simulate the actual behaviour of GCPs installed in soft soil conditions.

While three-dimensional (3D) modelling of GCP offers the advantage of capturing complex geometries and more realistic stress distributions, it comes with significant drawbacks. 3D models are often computationally intensive, require more detailed input data, and demand greater modelling expertise. As a result, they are less suitable for routine or preliminary analyses. In contrast, two-dimensional (2D) modelling is more straightforward, efficient, and cost-effective, making it ideal for parametric studies and conceptual evaluations.

According to many research studies [13-20], simulation of a 3D practical problem in a 2D model is usually done either by simplification of the model geometry or modification of field material parameters. "Unit cell model" in which a circular influence domain of GCP can be considered as one of the most well-known simplified 2D modelling concept [16]. In this model, the group effect of columns is not considered assuming that unit cell works independently and no lateral displacement.

Schweiger and Pande [17] presented an "Equivalent material" model where an equivalent material is used in the stone columns improved ground assuming that the effect of columns is uniform and homogeneous.

Hird et al. [20] developed a method to model vertical drains in plane strain finite element analyses of embankments on soft soil. They evaluated a drainage element's effectiveness in replicating the impact of a single drain under both axisymmetric and plane strain conditions by modifying the drain spacing and/or the soil permeability.

Indraratna and Redana [13] proposed a method for incorporating the smear effect of vertical drains into a 2D plane strain model by converting the smear zone radius and its permeability from axisymmetric conditions into equivalent values suitable for plane strain analysis.

Tan et al. [14] introduced two methods to convert an axisymmetric model to an equivalent plane strain model. In the first method, soil permeability values are modified keeping the model geometry similar to the field condition. In the second method, column width is altered based on the equivalent column area. However, the cross section was developed only as a 2D model and, at the end, it was suggested that the best method is the second method.

Weber et al. [18] simulated embankment foundations reinforced with stone columns in a plane strain model as a series of parallel trenches by modifying the geometry and some geotechnical parameters such as stiffness and permeability in the same 2D model. In this analysis, the effect of geometry and geotechnical parameters were not considered separately. However, the effect of smear zone was taken into consideration.

Castro [16] reviewed the main modelling techniques for GCP including plain strain modelling and 3D modelling and presented a new theoretical approach for the modelling that assists in selecting appropriate modelling approaches based on the problem type and highlighted calibration techniques for more simplified approaches like gravel trenches. Therefore, this newly proposed framework provides a structured basis to evaluate and select modelling strategies for GCP applications in both research and practice, contributing to more consistent and realistic analysis of column-treated ground.

In order to study the GCP behaviour, Alam and Islam [19] presented two types of models. In the first model, 2D plane strain condition was analysed by equating the drainage paths of actual case. In the other model, 3D analysis was done by keeping the stiffness and permeability to be same as in the field. Note that smear effect was not considered under this analysis.

Despite these valuable contributions, current literature still presents inconsistencies regarding the accuracy and reliability of 2D simulations, particularly in representing critical aspects such as smear effects, internal drainage conditions, complex load transfer mechanisms, and applicability to the Sri Lankan context. Moreover, most published case studies and numerical analyses focus on isolated or idealized conditions, often neglecting the interaction effects within groups of GCPs that typically exist in actual ground improvement projects. This gap limits the applicability of existing 2D modelling approaches to real-world scenarios. To address these limitations, the present study aims to develop an improved 2D modelling framework for GCPs by systematically evaluating both the parameter conversion and geometry conversion methods proposed by Tan et al. [14], while explicitly incorporating the findings of separate literature studies together. Two case studies will be analysed to verify the most suitable 2D modelling techniques applicable to the Sri Lankan context.

2. Equivalent Plane Strain Modelling Approaches

In order to convert the 3D actual field condition into 2D Plane Strain condition, two types of modelling approaches were used in this research study, namely, Parameter conversion method and Geometry conversion method.

2.1 Modelling Approach 1 – Parameter Conversion Method

In Approach 1, the permeability and stiffness of soft clay and gravel in GCP were modified by keeping width of the gravel trenches in the plane strain model same as the diameter of GCPs in the field. The spacing of GCPs was used as the spacing between gravel trenches [13] [18].

In general, water flow into the GCP from the surrounds is axisymmetric. For 2D analysis, it is necessary to convert axisymmetric model to plane strain condition (Figure 2). The method proposed by Indraratna and Redana [13] was used for this conversion. The axisymmetric condition consists of a unit cell which includes GCP (central drain) and surrounding soft soil cylinder as illustrated in Figure 2(a). The radius of the unit cell is R and length is l. The radius of the GCP (central drain) and smear zone are r_w and r_s, respectively.

The axisymmetric unit cell system is converted into equivalent parallel drain walls (plane strain condition) by adjusting the width of the drain wall, and the coefficient of permeability and stiffness of the surrounding soft soil, as shown in Figure 2(b). In the conversion, it is assumed that width of the plane strain unit cell is 2B. The width of the GCP drain (plane strain condition) is determined by considering the total pore water drain capacity of GCP in the axisymmetric condition and drain in the plane strain condition to be the same. Radius of the drain (half width of trench) and smear zone under the plane strain condition are denoted as b_w and b_s, respectively. In this approach, it is assumed that width of the trench ($2b_w$), width of the smear zone ($2b_s$) and width of unit cell ($2B$) under plane strain condition are equivalent to the GCP diameter ($2r_w$), smear zone ($2r_s$) and diameter of unit cell (D) under axisymmetric condition.

Based on Hird et al. [20], the horizontal permeability (k_h) under axisymmetric condition can be modified to adopt plane strain drainage condition as presented in Equation 1. The horizontal permeability in smear zone under axisymmetric condition is denoted by k_{hs} while the horizontal permeability of soft clay under plane strain condition is denoted as $k_{h,p}$, where subscript p represents the plane strain condition.

$$k_{h,p} = \frac{2}{3} k_h \frac{1}{[ln\left(\frac{R}{r_s}\right) + \frac{k_h}{k_{hs}} ln\left(\frac{r_s}{r_w}\right) - \frac{3}{4}} \qquad \text{... (1)}$$

By ignoring the well resistance, Indraratna and Redana [13] presented an equation to determine the horizontal permeability in the smear zone under plane strain condition ($k_{hs,p}$) as illustrated in Equation 2.

$$\frac{k_{hs,p}}{k_{h,p}} = \frac{\beta}{\frac{k_{h,p}}{k_h}\left[ln\left(\frac{R}{r_s}\right) + \frac{k_h}{k_{hs}} ln\left(\frac{r_s}{r_w}\right) - \frac{3}{4}\right] - \alpha} \qquad \text{... (2)}$$

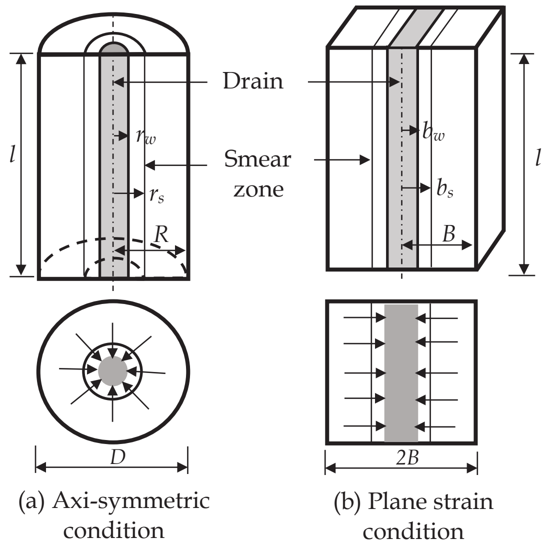


Figure 2 – Conversion from radial drainage to plane strain conditions [13]

where,

$$\alpha = \frac{2}{3} - \frac{2b_s}{B}\left(1 - \frac{b_s}{B} - \frac{{b_s}^2}{3B^2}\right) \qquad \text{... (3)}$$

$$\beta = \frac{1}{B^2}(b_s - b_w)^2 + \frac{b_s}{3B^3}\left(3{b_w}^2 - {b_s}^2\right) \qquad \text{... (4)}$$

When converting an axi-symmetric problem to plane strain conditions, it is important to consider the equivalent stiffness of unit cell in order to maintain the consistency as shown in Equation 5 [18].

$$[EA]_{3D} = [EA]_{2D} \qquad \text{... (5)}$$

where $[EA]_{3D}$ is the equivalent stiffness in actual condition (axisymmetric condition) whereas $[EA]_{2D}$ is the equivalent stiffness in plane strain condition. E is the elastic modulus while A is the cross-sectional area. Subscript $3D$ represents the axisymmetric condition (actual condition) and subscript $2D$ represents the plane strain condition. Equation 5 can be expanded as shown in Equation 6 to give more meaningful equation.

$$\left(E_{c,p} \times a_{s,p}\right) + \left[E_{s,p} \times (1 - a_{s,p})\right] = (E_c \times a_s) + [E_s \times (1 - a_s)] \qquad \text{... (6)}$$

where E_c and E_s are the elastic modulus of the GCP (column) and the surrounding soft clay, respectively, and subscript p represents the plane strain condition. The parameters without the subscript p represent the axisymmetric (actual) condition. The parameter a_s depicts the area replacement ratio. The area replacement ratio is defined as the area of the granular pile to unit cell area as shown in Equation 7.

$$a_s = \frac{A_{col}}{A_{col}+A_{soil}} \quad \text{... (7)}$$

where, A_{col} and A_{soil} are the cross-sectional area of the GCP and area of surrounding soft soil within the unit cell. In this method, for simplicity, it is assumed that $E_{s,p} = E_s$ and hence $E_{c,p}$ can be calculated using Equation 6.

2.2 Modelling Approach 2 – Geometry Conversion Method

In Approach 2, model geometry was modified by keeping the permeability and stiffness of the materials same as in the field. This approach is based on the equivalence of the drainage capacity in both axi-symmetric and plane strain conditions [13]. Hence, the cross-sectional areas of drain and surrounding soil are same for both axi-symmetric and plane strain conditions. In other words, area replacement ratio is kept constant during this geometrical transformation.

Schematic diagram of conversion of axi-symmetric model geometry to plane strain model geometry is illustrated in Figure 3 and the plane strain GCP width is given by Equation 8 [14].

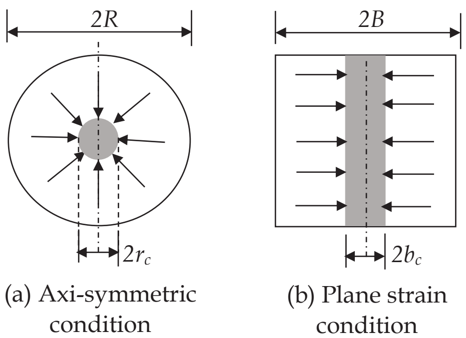


Figure 3 – Conversion of axi-symmetric model geometry to plane strain model geometry [14]

$$b_c = B\frac{r_c^2}{R^2} \quad \text{... (8)}$$

For simplicity, in this approach, it was assumed that width of the trench ($2b_c$) under plane strain condition is equal to the diameter of the GCP ($2r_c$) under axi-symmetric condition (b_c = r_c). Hence, Equation 8 is modified as shown in Equation 9 to compute the spacing of the gravel trenches ($2B$).

$$2B = \frac{2R^2}{r_c} \quad \text{... (9)}$$

3. Application of 2D Finite Element Modelling Approaches to Selected Case Studies

Finite element modelling was performed using PLAXIS 2D version 20 software. Two case studies, namely GCP improved embankment from the Southern Expressway Extension Project from Matara to Beliatta section and the Outer Circular Highway Northern Section-1, were analysed to identify the most reliable 2D finite element modelling method. Although GCP-improved embankment at the Southern Expressway Extension Project failed, only the field- monitored load-settlement data were used for the analysis, as such, it was not a concern for this study, which focuses only on converting 3D conditions into 2D scenarios, not on long-term behaviour.

3.1 Case Study 1 – Southern Expressway Extension Project from Matara to Beliatta Section (Modelling Approach – 1)

3.1.1 Geometry

The typical cross section of the GCP improved embankment used for the analysis is shown in Figure 4.

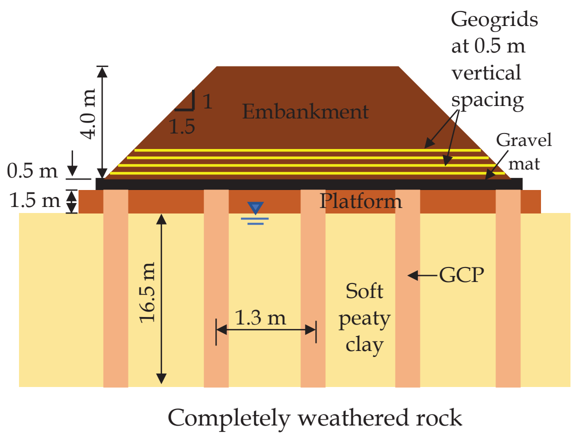


Figure 4 - Typical GCP improved embankment at Southern Expressway Extension Project from Matara to Beliatta section

The thickness of soft peaty clay in the area is around 16.5 m. The water table is at the ground surface. The average embankment height is 4 m and side slope of the embankment is 1:1.5 (Vertical: Horizontal). The embankment was constructed on a 1.5 m thick working platform. 0.5 m thick gravel mat was placed prior to the embankment filling. Four geogrids have been laid in 0.5 m vertical spacing to enhance the slope stability.

According to field records, the length of the GCPs is about 16.5 m. The diameter of GCP is 0.7 m and GCPs were installed in a square grid pattern with a spacing of 1.3 m. Hence the area replacement ratio can be calculated as 22.8%. The generalized PLAXIS 2D model used for the analysis is illustrated in Figure 5. Due to symmetricity, only right half of the embankment was considered for the analysis.

In the analysis, bottom boundary was kept fixed since completely weathered rock was encountered underneath and it was closed for the water flow. Left and right boundaries were restrained in the horizontal direction only, but free in the vertical direction. Free flow was not allowed at both left and right boundaries. Left boundary represents the line of the symmetry while right boundary depicts the end of the unimproved area.

3.1.2 Model parameters

Material parameters and constitutive models adopted in the analyses are shown in Table 1. For this analysis, Soft Soil Creep (SSC) model was used to model the soft peaty clay while Hardening Soil (HS) model was used for the Gravel mat, GCP fill material and embankment. All of these models were selected based on the recommendations of Brinkgreve [21], previous studies cited in Kaliakin et al. [22] and computations done to verify the most reliable constitutive model for the application.

It is a well-known fact that same material is used for the construction of both embankment and working platform, as such, same material parameters were used for the embankment and the working platform. The material parameters presented in Table 1 were obtained from the design parameters used in the GCP design in the Southern Expressway Extension Project from Matara to Beliatta section. Geogrid tension was taken as 200 kN/m^2 in both transverse and longitudinal directions. Geogrids were represented using geogrid structural element in the PLAXIS analysis.

Based on Weber et al. [18], the ratio between horizontal permeability and vertical permeability in the undisturbed zone is equal to 2.0. Similar ratios have also been reported in Hird et al. [20] for vertical drain modelling. Hence same ratio was taken for this study. It is well known that horizontal permeability is equal to the vertical permeability in the smear zone due to soil disturbance during GCP installation [23]

Table 1 – Material Parameters– Case Study 1

Parameter	Embankment	Gravel Mat	GCP	Soft peaty clay
Constitutive model	HS	HS	HS	SSC
Drainage condition	Drained	Drained	Drained	Undrained
Unit weight, γ (kN/m^3)	20	20	22	12
Initial void ratio, e	0.5	0.5	0.5	3.675
Secant stiffness for primary loading, E_{50}^{ref} (kN/m^2)	25000	50000	40000	2000
Unloading/reloading stiffness, E_{ur}^{ref} (kN/m^2)	80000	170000	130000	-
Tangent stiffness for oedometer loading, E_{oed}^{ref} (kN/m^2)	20000	40000	32000	-
Compression index, C_c	-	-	-	1.5
Swelling index, C_s	-	-	-	0.15
Secondary compression index, C_a	-	-	-	0.08
Effective cohesion, c'(kPa)	5	0	0	2
Effective friction angle, φ'(deg)	32	40	36	20
Poison ratio, υ	0.3	0.3	0.3	-
Horizontal permeability, k_h (m/day)	0.1	9.0	1.0	8.64x10^{-4}
Vertical permeability, k_v (m/day)	0.1	9.0	1.0	4.32x10^{-4}

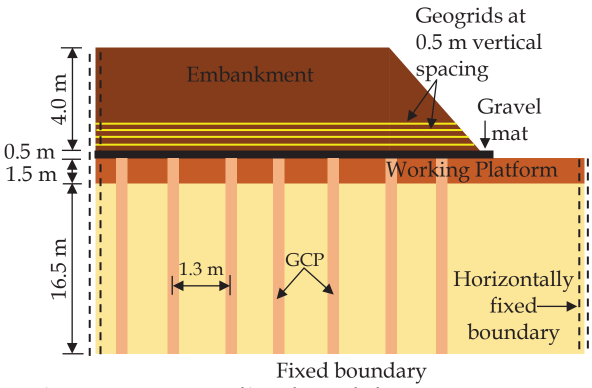


Figure 5 - Generalized model geometry-Case study - 1

As such, the ratio between horizontal permeability in the undisturbed zone (k_h) and horizontal permeability in the smear zone (k_{hs}) in the real situation was taken as 2.0 for this analysis. Further, it was assumed that width of the smear zone is 2 times the width of the GCP diameter ($r_s = 2r_w$) [18].

The PLAXIS model used for the Case Study-1, Modelling Approach-1 is illustrated in Figure 6. It can be noted that GCP diameter and spacing are same as that of axisymmetric (actual) condition. For simplicity, in the analysis, space between GCPs were considered as smear zones.

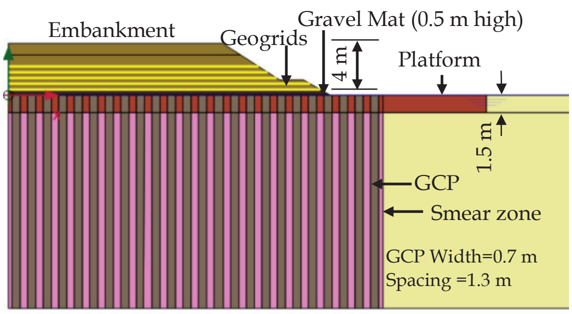


Figure 6 – Case study-1 (Approach 1)

3.2 Case Study 1 – Southern Expressway Extension Project (Modelling Approach – 2)

The PLAXIS model used for the Case Study-1, Modelling Approach-2 is depicted in Figure 7. The equivalent diameter ($2R$) of the unit cell for square pattern under axi-symmetric condition is given by Equation 10proposed by Bergado et al. [23], where S is the GCP spacing.

$$2R = 1.13\,S \qquad \dots (10)$$

Since spacing of the GCP is 1.3 m, equivalent diameter ($2R$) can be computed as 1.469 m. As such, spacing of the trenches ($2B$) can be estimated as 3.0 m by taking the GCP radius ($r_c = b_c$) as 0.35 m. According to Weber et al. [15], width of the smear zone was considered as two times the width of the trench.

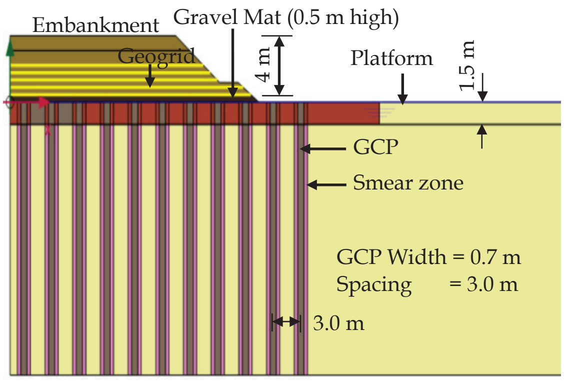


Figure 7 – Case study -1 (Approach 2)

3.3 Case Study 2 – Outer Circular Highway Project (Modelling Approach – 1 & 2)

3.3.1 Geometry

The typical cross section of GCP improved embankment used for the analysis is shown in Figure 8. The subsurface soil profile consists of 1.5 m thick silty clay layer followed by 4.0 m thick very soft peat layer. 3.0 m thick loose silty sand and 4.5 m thick medium dense sand are below the very soft peat layer. Water table was at the existing ground surface. The side slope of the embankment is 1.8H:1V (Horizontal: Vertical) and the average embankment height is about 12 m. The embankment was built on a0.5 m thick working platform. Prior to the embankment construction, a 0.5 m thick gravel mat was placed, and one geogrid was laid on the gravel mat.

In this numerical model, the Gravel Compaction Piles were designed with a length of 9 m to improve the strength and stability of the soft ground. Each pile had a diameter of 0.7 m, and they were arranged in a square grid pattern. The centre-to-centre spacing between the piles was maintained at 1.2 m. As such, area replacement ratio (a_s) is 26.7%.

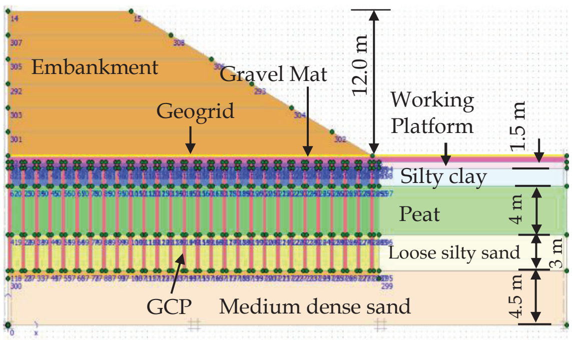


Figure 8 - Case study –2 (Approach 1)

3.3.2 Model parameters

Material parameters used for the analysis is depicted in Table 2. Soft Soil (SS) model was used to model very soft peat and Mohr-Coulomb (MC) model was used to model GCP, embankment and rest of the soil layers [21].

The Mohr-Coulomb model was selected as it captures essential strength and deformation behaviour with a limited number of parameters, making it a practical and efficient choice, especially when data availability is limited. For the very soft peat, the Soft Soil model was used to better represent its high compressibility and non-linear stress-strain behaviour. These two models are commonly adopted in practice, are relatively simple to implement in PLAXIS, and have been recommended by Brinkgreve [21]. In contrast, the other project adopted the Hardening Soil model and Soft Soil Creep model, which provide more detailed and realistic predictions, particularly when focusing on long-term settlement and time-dependent behaviour. These models require more advanced input parameters and are suitable when the goal is to capture the full range of soil behaviour over time. Therefore, the selection of constitutive models in each project reflects a balance between the complexity of soil behaviour, and the availability of model parameters.

Standard fixities (fixed bottom boundary and roller supports for vertical boundaries) were used as the geometry boundary conditions, and a closed consolidation boundary was used for the flow. Similar to Case Study -1, the material parameters of the working platform and embankment were considered to be the same. Majority of the material parameters used for the analysis were obtained from Karunawardana et al. [2] and rest were obtained through literature [3-6] [21] [23]. Geogrid tension in transverse direction was given as 300 kN/m^2 [2].

Table 2 – Material Parameters – Case Study

Parameter	Embankment	GCP	Gravel mat	Silty clay	Very soft peat	Loose silty sand	Medium dense sand
Constitutive model	MC	MC	MC	MC	SS	MC	MC
Drainage condition	Drained	Drained	Drained	Undrained	Undrained	Drained	Drained
Unit weight, γ (kN/m^3)	18	22	20	17	11.3	16.5	17
Horizontal Permeability, k_h (m/day)	0.1	1.92	1	0.03	0.0003	1	0.5
Vertical Permeability, k_v (m/day)	0.1	1.92	1	0.01	0.0001	1	0.5
Youngs modulus, E (kN/m^2)	25 000	40 000	50 000	25 000		20 000	30 000
Poisson ratio, υ	0.3	0.3	0.3	0.3		0.3	0.3
Cohesion, c'(kN/m^2)	5	1	1	3	1	1	1
Friction angle, φ' (deg)	32	36	36	20	16	23	30
Dilatancy angle, ψ (deg)	0	0	0	0	0	0	0
Modified compression index, C_c'					0.28		
Modified swelling index, C_s'					0.028		

3.3.3 Modelling approaches
Both parametric conversion method stated in Section 2.1 and geometric conversion method stated in Section 2.2 were adopted as modelling approaches. However, in the geometric conversion method, by keeping the GCP spacing same for both axi-symmetric and plane strain conditions, GCP diameter was modified. It can be noted that the original GCP diameter of 0.7 m has been modified to 0.32m width trench in the plane strain condition. For both approaches the generalized model shown in Figure 8 was used for the analysis.

4. Results and Discussion

The validity of the two approaches mentioned in Section 2 were evaluated by comparing the numerical model results with field measurements with respect to variation of settlement and pore water pressure over time.

4.1 Case Study -1
The results of the plain-strain analysis under two approaches, namely Parameter conversion method and Geometry conversion method, together with the measured settlement are plotted for the analysis. Settlement comparisons of the centre and Right-Hand Side (RHS) of the embankment are illustrated in Figure 9 and Figure 10, respectively.

According to the presented settlement results, the settlement behaviour of model developed under Approach - 2 highly deviated from the measured field settlement behaviour. On the other hand, an excellent match is obtained when the model was developed using Approach -1.

The predicted and measured pore water pressures at centre line of the embankment at a depth of 3.75 m below the ground surface are compared in Figure 11. The variation of embankment fill height is also plotted in the same figure for a proper comparison. It can be noted that field measurements have commenced approximately two months after the embankment construction started.

Based on the results illustrated in Figure 11, relatively higher porewater pressures have been developed in the model created under Approach - 2. The porewater pressure variation in model created under Approach - 1 and field measurements show a closer behaviour until the end of embankment filling. After the filling process, field porewater pressures have increased with time. The reason for the unusual pore water pressure increment at the end of embankment construction is due to significant amount of GCPs area partially clogged and more details are explained in Nekasiny et al. [12].

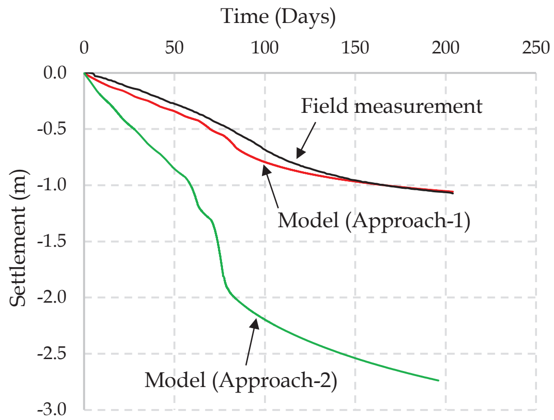


Figure 9 – Settlement at center of the embankment for Case study -1

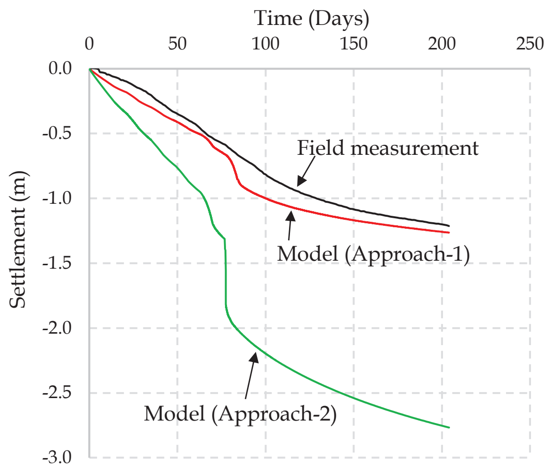


Figure 10 – Settlement at RHS of the embankment for Case study -1

4.2 Case Study -2
The variation of settlement against time behaviour for both conversion methods under numerical modelling together with field measurements [2] is depicted in Figure 12.

It is evident that the parameter conversion method (Approach-1) shows a closer match to the field data, while the geometry conversion model (Approach-2) overestimates the settlement caused by the embankment load.

Similarly, the predicted and measured pore water pressures under centre line and toe of the embankment at a depth of 3.5 m below the ground surface (center of peat layer) predicted under parameter conversion method (Approach-1) is illustrated in Figure 13 whereas that of under geometric conversion method (Approach-2) is illustrated in Figure 14.

Based on the data presented in Figure 13 and Figure 14, it can be seen that predicted pore water pressure variations under both approaches do not well agree with the field measured data. It can be noted that predicted pore water pressure is higher than that of measured at centre line while predicted pore water pressure is lower than that of measured at toe of the embankment. Since there is considerable disturbance to the soft soil during GCP installation, there is a possibility to change the geotechnical parameters within the subsurface, which may cause the difficulty of prediction of pore water pressure. This is well agreed with the findings presented by Indraratna and Redana [13].

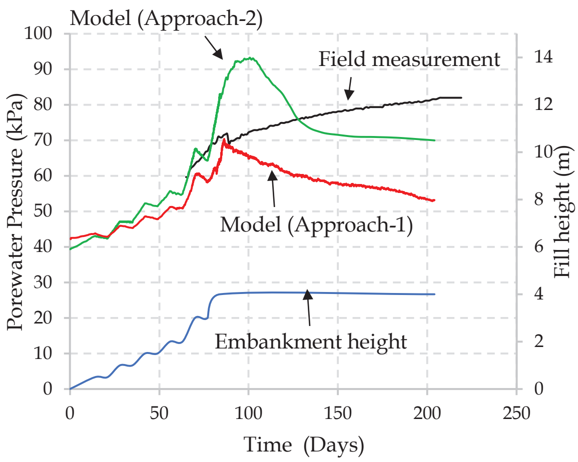


Figure 11 – Porewater pressure variation with time for model -1

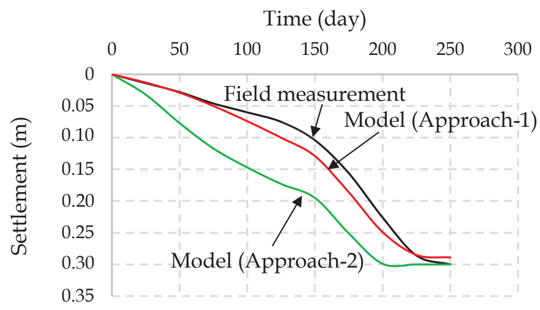


Figure 12 – Settlement at center of the embankment for Case study-2

5. Conclusions

It is not possible to model the exact behaviour of GCP improved ground in 2D due to the geometrical distortion. However, in this research study, authors have made an effort to develop a 2D model which can give results much closer to the behaviour of GCP in the field.

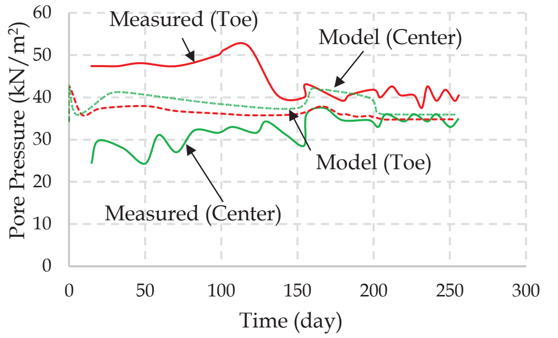


Figure 13 – Porewater pressure variation with time under Approach-1 for Case study-2

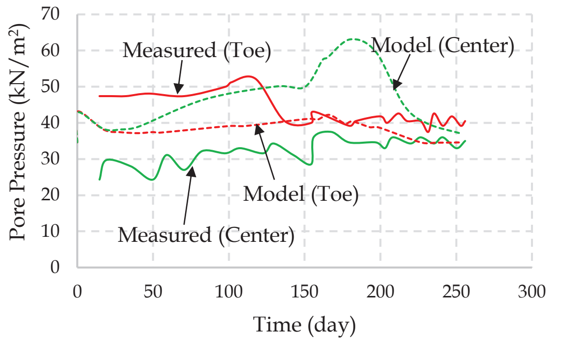


Figure 14 – Porewater pressure variation with time underApproach-2 for Case study-2

In order to achieve above objectives, a mathematical formulation has been used to transform the actual axisymmetric condition, incorporating the equal vertical drainage under both conditions. Based on this research study, the following conclusions can be drawn:

1. The settlement behaviour of 2D plane strain models developed by modifying the permeability and stiffness of materials (Parameter conversion method - Approach 1) agreed well with the field behaviour with respect to settlement. However, the 2D plane stain models developed by modifying the geometry of GCP improved ground (Geometry conversion method - Approach 2) show a highly deviated settlement behaviour from field behaviour. As such, it can be concluded that most appropriate method to convert the actual axi-symmetric condition into 2D plane-strain condition is the Parameter Conversion method, *i.e.* modify the permeability and stiffness of materials.

2. It is really difficult to predict the actual field behaviour with respect to pore water development of the smear zone during GCP installation and clogging of drainage paths

may lead to difficulty of accurately predicting the pore water pressure within the GCP improved ground.

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