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A Comparison of Two and Three Dimensional Finite Element Modelling of a Geosynthetic Reinforced Pile-Supported Embankment

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ABSTRACT

Embankment construction on soft ground has increased considerably over recent years as a result of the increase in infrastructure development activities and due to the unavailability of suitable land. Geosynthetic reinforced pile-supported (GRPS) embankments provide an effective and reliable solution to the problem of constructing embankments over soft ground. The combination of geosynthetic reinforcement and piles can alleviate the uneven surface settlements on the embankment crest while reducing the embankment load transferred to the soft foundation soil. This paper presents a numerical analysis based on the finite element method carried out on a GRPS embankment. Analysis was carried out in both two-dimensional plane strain condition for different two-dimensional idealizations of piles and in three-dimensional condition. The interaction between geosynthetic and soil was taken into consideration during the analysis. The results obtained for the two-dimensional models are compared with the three-dimensional model results. The stress transferred to the piles and foundation soil, development of settlements at the base of the embankment on both foundation soil and piles and the generation and dissipation of excess pore water pressures during and after construction for both two-dimensional and three-dimensional models are discussed.

Keywords: numerical modelling, GRPS embankment, two-dimensional idealizations, soft soil

1 INTRODUCTION

Embankments are frequently used in many infrastructure development projects in order to elevate the ground surface for construction. Due to the unavailability of proper land for construction, many projects are currently being undertaken on soft grounds which were previously considered unsuitable. Construction on soft soil is a challenging task due to the undesirable characteristics of soft soil such as low bearing capacity, insufficient shear strength and high compressibility and therefore, requires special attention on stability and settlements. Different ground improvement methods to overcome these problems include using lightweight fill materials for the embankment fill, reducing the slope of the embankment, preloading, constructing in stages leaving time for consolidation, over excavating the existing ground and replacing with suitable materials, providing vertical drains, vacuum consolidation, providing reinforcement for the embankment and adding pile supports. The focus of this study is an embankment supported on piles and reinforced with geosynthetics.

Addition of geosynthetic reinforcement will enhance the load transfer from the embankment to the piles. This system has several advantages such as the embankment can be constructed in a single stage hence, suitable for fast-track construction, the total and differential settlements of the embankment are significantly reduced along with the lateral displacements and this method can be used reliably in any unpredictable ground irrespective of the sub soil properties. Numerous studies (both numerical and experimental) have been carried out in the last few decades on pile-supported embankments with or without geosynthetic, to investigate their behaviour and the load transfer mechanism (Hewlett and Randolph 1988; Low et al. 1994; Han and Wayne 2000; Han and Gabr 2002; Li et al. 2003; Collin 2004; Han et al. 2004).

A geosynthetic reinforced pile supported embankment is a three dimensional problem. But, three-dimensional finite element modelling can be complicated as well as time consuming and requires computers with a very high processing power. Therefore, three-dimensional modelling is not always economical. Due to this reason, a large number of studies in the area of pile-supported embankments have been carried out in two-dimensional plane strain condition and the results also can be achieved to a reasonable accuracy.

Different approaches to transform a real three dimensional problem to a two dimensional plane strain idealization can be found in literature (Figure 1). The three dimensional piles are modelled as continuous walls with equivalent thickness t_{eq} or equivalent elastic modulus E_{eq}. Bergado and Long (1994) have adopted a method for this purpose by keeping the ratio of pile area to the total soil cell area containing the pile (A_C/A_E) also known as the area replacement ratio (a_S), a constant as shown in Figure 1(a). The equivalent thickness of the walls can be found using the following equation where, s_x and s_y are the pile spacing in x and y directions respectively. This method is later referred to as the Equivalent Area (EA) method.

$$A_c/A_E = (t_{eq} \, . \, s_y)/(s_x \, . \, s_y) \qquad (1)$$

Another way of modelling the piles in two dimensional plane strain condition is the use of an equivalent elastic modulus for the pile walls considering the pile-soil system, keeping the thickness of the wall the same as the pile diameter (Figure 1.b). The equivalent modulus is calculated using the area replacement ratio as mentioned in Huang et al. (2009) as shown in Equation (2); where E_c and E_s are the elastic moduli of pile and soft soil respectively. This method is referred to as the Area Replacement Ratio (ARR) method.

$$E_{eq} = E_c \, . \, a_s + E_s \, . \, (1- a_s) \qquad (2)$$

Another means of achieving this is by taking the normal stiffness of the piles into consideration (Figure 1.c). Equivalent elastic modulus of the walls can be found by;

$$E_{eq} \, . \, A_w = E_c \, . \, A_c + E_s \, . \, (A_w - A_c) \qquad (3)$$

where, A_w is the plan area of pile wall. This is referred to as the Equivalent Normal Stiffness (ENS) method.

For the same purpose, Stewart et al. (1993) has adopted a different approach using the flexural stiffness of the piles. The thickness of the wall remains at the same value as the pile diameter (Figure 1.c) and the equivalent elastic modulus is calculated using the following equation where, I_w, I_c and I_s are the second moment of areas of the pile wall, pile and soft soil respectively. This method is referred to as the Equivalent Flexural Stiffness (EFS) method.

$$E_{eq} \, . \, I_w = E_c \, . \, I_c + E_s \, . \, I_s \qquad (4)$$

The results are compared with the three dimensional model results as well as the field measurements to investigate the most appropriate two dimensional idealization method.

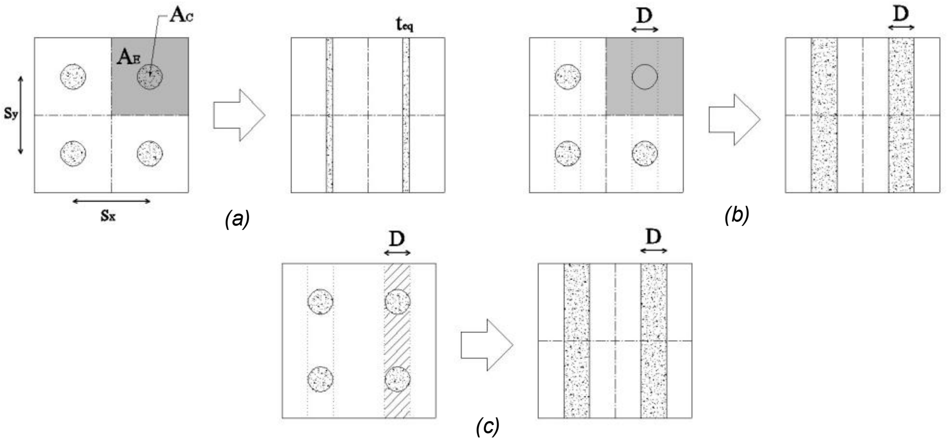

Figure 1. Different two-dimensional idealization methods

2 NUMERICAL ANALYSIS

The geosynthetic reinforced pile supported embankment considered in this study is located in a northern suburb of Shanghai, China. A detailed description of the site conditions, instrumentation and the embankment construction process is described by Liu et al. (2007). The cross section of the instrumented embankment and the soil profile are shown in Figure 2. The embankment has a height of 5.6 m and a crest width of 35.2 m. The side slopes are 1:1.5 (vertical:horizontal). The piles used to support the embankment are cast in-situ concrete annulus piles with a wall thickness of 120 mm, 1m diameter and 16 m of length. The top 0.5 m of the piles were cast as solid cylindrical piles as a measure to recover any damage to the top part of the annulus caused by the withdrawal of the double wall casing. Piles are supported on the sandy silt layer and have a centre to centre spacing of 3 m. The ground water table (GWT) is located at a depth of 1.5 m below the ground surface. The geosynthetic reinforcement layer is sandwiched between two gravel layers each having a thickness of 0.25 m in order to prevent damage. The embankment has been constructed over a period of 55 days.

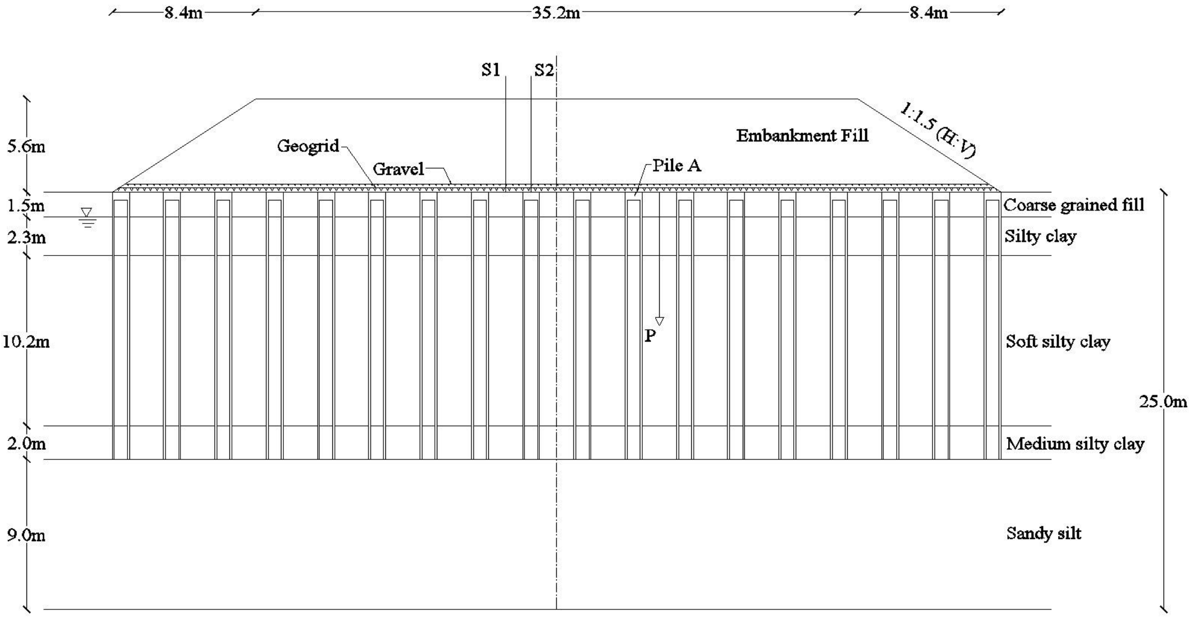

Figure 2. Geometry of the embankment with soil profile

2.1 Material model and parameters

The material parameters used in the model are summarised in Table 1.

Table 1: Material properties used in the finite element simulations

Material	Parameters
Coarse grained fill	E = 7 MPa, c' = 15 kPa, φ' = 28^0, ψ = 0^0, ν = 0.3, γ = 20 kN/m^3
Silty clay	λ = 0.06, κ = 0.012, M = 1.2, e_1= 0.87, ν = 0.35, k = 4.32x10^{-4} m/day, γ = 20 kN/m^3
Soft silty clay	λ = 0.15, κ = 0.03, M = 0.95, e_1= 1.79, ν = 0.4, k = 4.32x10^{-4} m/day, γ = 17 kN/m^3
Medium silty clay	λ = 0.05, κ = 0.01, M = 1.1, e_1= 0.88, ν = 0.35, k = 4.32x10^{-4} m/day, γ = 20.5 kN/m^3
Sandy silt	λ = 0.03, κ = 0.005, M = 0.28, e_1= 0.97, ν = 0.35, k = 4.32x10^{-3} m/day, γ = 20 kN/m^3
Embankment fill	E = 20 MPa, c' = 10 kPa, φ' = 30^0, ψ = 0^0, ν = 0.3, γ = 18.5 kN/m^3
Gravel bed	E = 20 MPa, c' = 10 kPa, φ' = 40^0, ψ = 0^0, ν = 0.3, γ = 18.5 kN/m^3
Piles	E = 20 GPa, ν = 0.2
Geosynthetic	$J = Et$ = 1180 kN/m, ν = 0.3

J – Stiffness of the geosynthetic, *t* – thickness of the geosynthetic, γ – unit weight of soil

The constitutive behaviour of the four silty layers was modelled using the Modified Cam Clay (MCC) model. The required parameters for the MCC model are slope of the virgin consolidation line, λ; slope of the swelling line, κ; void ratio at unit pressure, e_1; slope of the critical state line, M and Poisson's ratio, ν. These four layers are considered to be normally consolidated. A linear elastic-perfectly plastic model with Mohr-Coulomb failure criterion was used to model the embankment fill, coarse grained fill and the gravel bed. The parameters used for this model are effective cohesion, *c'*, friction angle, φ', dilation angle, ψ, Young's modulus, E and Poisson's ratio, v. The geosynthetic layer and piles were modelled as linear elastic materials. Interface friction is considered between the gravel bed and the geosynthetic layer during the analysis and the interface friction angle is assumed to be as same as the friction angle of gravel.

2.2 Three dimensional modelling

Three dimensional finite element simulation was carried out using ABAQUS/Standard finite element modelling software. The soil layers, piles, gravel and embankment fill were modelled using 20-node brick elements. Fully drained condition was assumed in the coarse grained fill layer, embankment fill and the gravel bed due to the generally high permeability of these layers. The geosynthetic layer was modelled using 8-node quadratic membrane elements.

Since the embankment is symmetrical along the centreline, only half of the embankment is modelled. The foundation soil is taken to be 25 m deep overlying a rigid impermeable stratum. The horizontal length of the model is taken to be 78 m so that the boundary effect can be minimised. The displacements in all three directions at the bottom (z=0 plane), are set to zero. Along the centreline of the embankment (x=0 plane), symmetrical boundary conditions were assigned. In the far end of the model (x=78 plane), displacements along x direction are set to zero. Symmetrical boundary conditions were assigned for the two vertical planes (y=0 plane and y=6 plane). All these boundaries are considered to be impermeable and pore fluid flow is permitted only through the bottom surface of the coarse grained fill (z=23.5 plane) by setting a zero pore pressure boundary condition on that plane.

2.3 Two dimensional modelling

Two dimensional models were also analysed with ABAQUS using different plane strain idealizations mentioned in the previous section. The same embankment geometry with same material properties were used for the analysis. The soil layers below the ground water table were analysed using eight-node plane strain elements with reduced integration and pore pressure degrees of freedom and all the soils which were assumed to be fully drained (fill and gravel layers) and piles were modelled using eight-node plane strain elements with reduced integration but without pore pressures. The geosynthetic layer was modelled using three-node truss elements which transfer loads axially and has no bending stiffness.

3 ANALYSIS OF RESULTS

3.1 Load transfer to the piles and foundation soil

The main objective of adding pile supports and geosynthetic reinforcement to an embankment founded on soft soil is to obtain an effective load transfer. Different factors contribute to this load transfer mechanism such as soil arching, stress concentration due to the stiffness difference between piles and surrounding soil and the membrane action of the geosynthetic layer. In the real three dimensional nature, soil arches develop in both lateral and transverse directions which form a dome shape. But, when two dimensional models are considered, soil arches can only be developed in the lateral direction since pile walls are spanning all the way through the transverse direction. Therefore, it is hard to expect the same load transfer characteristics from a two-dimensional model as from a three-dimensional model. Computed and measured vertical stresses on pile A and foundation soil around that are presented in Table 2.

Table 2: Comparison of measured and computed stresses

Model	Average Stress on Pile A (kPa)	Average stress on Surrounding Soil (kPa)
Field Measured	567.9	40.31
Three Dimensional (3D)	603	56.3
Equivalent Area (EA)	581	52
Area Replacement Ratio (ARR)	364	51.5
Equivalent Normal Stiffness (ENS)	368	50.9
Equivalent Flexural Stiffness (EFS)	357	51

It can be seen that out of all the two-dimensional models analysed, the equivalent area method gives the closest values to the three-dimensional results as well as the field measurements. It can be seen that the field measured vertical stress values on the foundation soil are somewhat lesser than the computed values by both three-dimensional and two-dimensional models. A reason for this can be the disturbance occurred to the surrounding soils when the pile casting took place. Because, a double wall casing has been used for this purpose and it was withdrawn as the casting proceeded. Due to this disturbance the stiffness of the surrounding soil might have decreased thus reducing their load carrying capacity. The two dimensional idealizations except for the equivalent area method yield considerably low values for the stress on the piles. This might be due to the fact that the plan areas of the piles in all these cases are significantly higher than the plan area of the equivalent area method piles hence, giving a lower stress.

3.2 Settlements of piles and soil

This section presents the settlements in the foundation soil and piles. For comparison purposes, the settlement values corresponding to settlement plates S1 and S2 are selected. Settlement plate S1 represents the middle point on the foundation soil surface between first and second piles from the centre of the embankment as shown in Figure 2 and S2 is located on the first pile from the centreline. Vertical settlements at these two points are computed by both three dimensional and two dimensional models and are shown in Figures 3 and 4.

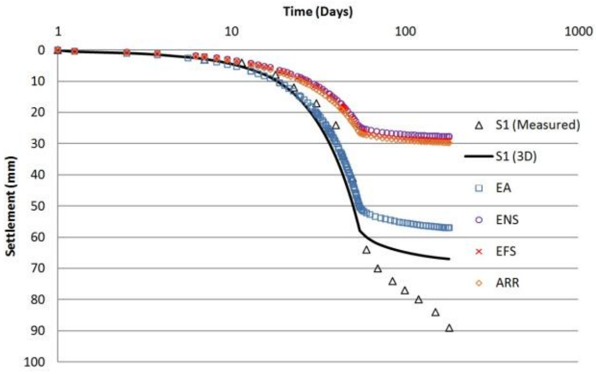

Figure 3. Settlement of foundation soil around pile A

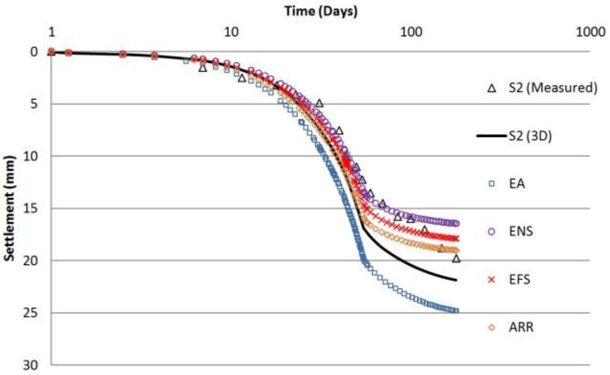

Figure 4. Settlement of pile A

Figure 3 shows the vertical settlement of the foundation soil between the first two piles from the center of the embankment. According to the results, the ENS, EFS and ARR methods largely under predict the settlements in the foundation soil layer. This is because the thickness of the pile wall was assumed to be the diameter of the pile for all these three methods. This makes the plan area of the foundation soil to decrease so that the volume of the embankment below the soil arches which will stand on this foundation soil will also reduce. Therefore the total load transferred to the foundation soil reduces thus reducing the settlements. According to Figure 4 the settlement of the pile head is closely predicted by all models both three dimensional and two dimensional. The equivalent area method slightly over predicts the settlements and the other three methods slightly under predict when compared with the three dimensional model. However, those three methods have given closer values to the field measurements for the pile head displacement. Out of all the two dimensional methods, the EA method yields the most agreeable values to the three dimensional model results as well as to the field measurements.

3.3 Variation of excess pore water pressure

Excess pore pressure developed within the foundation soil is calculated using the piezometer P which is installed at a depth of 8 m below the foundation soil surface and 6 m from the centre of the embankment as shown in Figure 2. The plotted results are shown in Figure 5. The three dimensional model predicts closer results to the actual measured values of excess pore water pressure and among the two dimensional models, the EA method predicts the closest excess pore pressure variation to the actual case and three dimensional model. All the other two dimensional models give low excess pore pressure values. This can be due to the lower stress transferred to the foundation soil during the embankment construction. Since the permeability reduction of the soft soil with the decrease of void ratio during consolidation is not considered in any of these finite element simulations, excess pore water pressure dissipates much quicker than in the actual field condition in all cases.

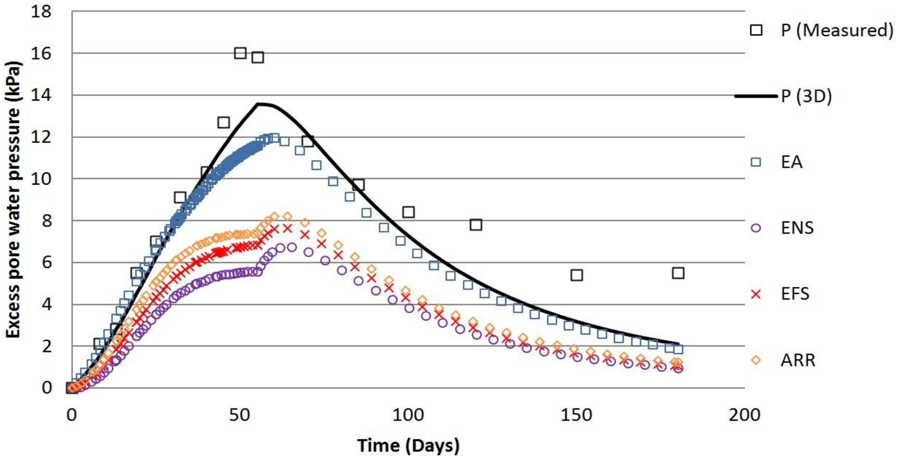

Figure 5. Excess pore pressure variation with time

4 CONCLUSION

An actual GRPS embankment was analysed in this study using both three-dimensional and two-dimensional numerical models using finite element method. Two dimensional models are analysed using different plane strain idealizations of piles and the results are compared with the three-dimensional model results and filed measurements. According to the results it is clear that the three-dimensional model gives the closest results to the actual field measurements. On the other hand, out of all the two-dimensional idealizations adopted, the equivalent area method yields the closest results to the three-dimensional model results and field measurements. Therefore, it can be considered as the most appropriate two-dimensional idealization method.

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