

FINITE ELEMENT ANALYSIS OF GEOSYNTHETIC REINFORCED PILE SUPPORTED EMBANKMENTS

Anjana Bhasi¹, K.Rajagopal²
¹ Ph.D. Research Scholar, ² Professor
Department of Civil Engineering
Indian Institute of Technology, Madras, Chennai, India
Email: anjanabhasi@yahoo.com, gopalkr@iitm.ac.in
Telephone: 91-44 2257 4263, 91-44 2257 4252 (FAX)

Abstract: Embankments constructed on soft soils undergo large deformations and lateral movements which results in long construction delays and premature failure. Several researches in the past have shown that Geosynthetic Reinforced Piled Embankment Systems (GRPES) is an attractive solution for such problems. This system consists of horizontal layers of geosynthetic reinforcement, soil embankment and piles. In this paper, finite element based numerical study has been carried out using ABAQUS to study the interactions between friction pile-soil-geosynthetics during and after construction of the embankment. The influences of various parameters like the geosynthetic stiffness, pile height and embankment height on the performance of GRPES has been studied in this work. The paper discusses the model details and the various results obtained along with design implications for practitioners.

Keywords: Embankment, clay, friction pile, geogrid, axisymmetric model, coupled analysis

1. Introduction

Rapid growth and construction in urban and coastal areas have forced the use of all available land for development. Coastal regions are generally covered with soft compressible clayey deposits and mud. Construction of road and rail embankments over soft clay and peat foundations often result in large differential settlements occurring at the surface of the embankments which results in long construction delays and premature failure. These differential settlements between the structure and the embankment surface will gradually build up requiring subsequent rising of the embankment.

Stabilization of soft clay is thus one of the important construction techniques in geotechnical engineering. A variety of techniques are available to address these issues. They include preloading, deep mixing columns, stone columns, use of light weight fill, and soil replacement.

According to Chen et al., 2008 due to their low costs, small total and differential settlements compared to other traditional methods of soil improvement Geosynthetic Reinforced Piled Embankment Systems (GRPES) can be used when there is a need for faster construction.

Geosynthetic reinforced embankment systems consists of piles/columns founded on a hard-bearing stratum below the embankment and rise up through the soft soil to the ground surface. Above these columns coarse-grained soil or aggregate consisting of one or more layers of geosynthetics usually in the form of geogrids is placed which acts as the reinforcement layer. Embankment is then constructed above this reinforcement layer. Conceptually, the geosynthetic pile embankment is similar to the pile-raft systems except that only flexible mat is provided in this case. Figure 1 shows a schematic of the geosynthetic reinforced piled embankment system (GRPES).

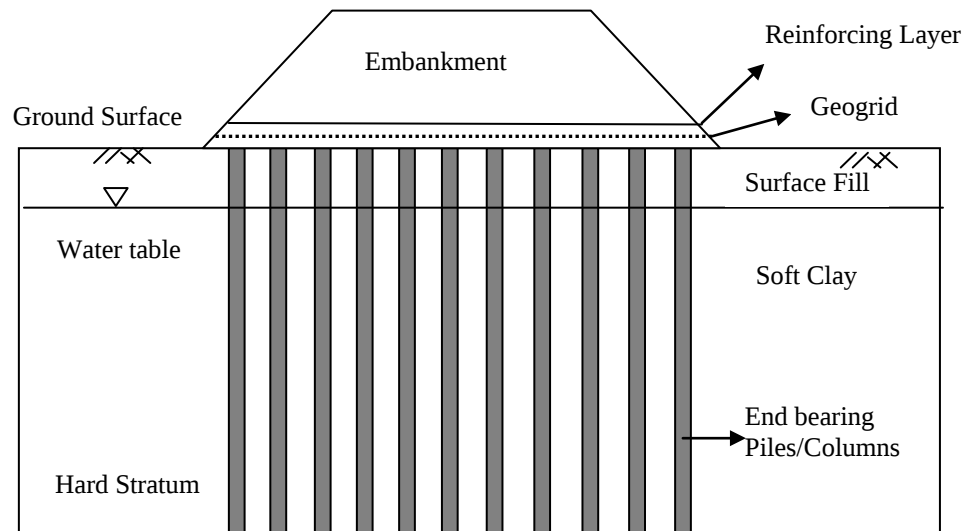


Figure 1. Geosynthetic Reinforced Piled Embankment System.

The load transfer mechanism in such a system is a combination of the following three phenomena- (1) Soil arching (2) Membrane effect of the geosynthetic and (3) Stress Concentration due to the difference in the stiffness between the pile and the surrounding soil. Many different methods are currently used to determine the load on the embankment base. Some of the widely used are: BS 8006, Terzaghi's, Hewlett and Randolph, Guido, and Russell et al. Current design methods for geosynthetic-reinforced embankments are based on a wide range of conservative assumptions, which give rise to conflicting results (Smith and Filz, 2007). To eliminate the uncertainty in these methods and to better understand the performance of GRPES, numerical modeling can be used as an effective tool.

For a complete representation of the geosynthetic reinforced piled embankment system, a full three dimensional analyses is required. To simplify the analysis an axisymmetric unit cell

approach has often been adopted. Much work has been based on the unit cell concept (BS 8006, 1995; Russell and Pierpoint, 1997; Kempton et al., 1998; Han and Gabr, 2002; Yoo and Kim, 2009). A unit cell approach represents the interior portions of embankments; away from the influence of side slope (Smith and Filz, 2007). Generation and dissipation of excess pore water pressure is inevitable during and after construction due to the presence of high groundwater table in the site. As a result, the behaviour of most geosynthetic reinforced embankments is time-dependent and it requires coupled hydraulic and mechanical modeling (Huang and Han, 2010). In this paper a two-dimensional parametric study based on coupled analysis was conducted to study the time-dependent behaviour of GRPES under various conditions. Study of friction piles have been neglected till now. Therefore the authors aim to look into the behaviour of friction piles in effectively transferring the embankment load to the soil below.

2. Finite element modeling

2.1 Case history considered

A case history concerning Geogrid-Reinforced and Pile-Supported Embankment reported by Liu et al., 2007 was used for modeling. The embankment was 5.6 m high and 120 m long with a crown width of 35 m. Concrete piles were placed in a square pattern at 3m center to center spacing. The area ratio was about 8.7% which was close to the lowest limit suggested by Han and Gabr, 2002. Details of the embankment construction can be found in Liu et al., 2007.

2.2 Description of the finite element model

Finite element analyses were performed using the program ABAQUS (SIMULIA, 2009) coupled with Biot's consolidation theory (1941). In the coupled problems, the force equilibrium and fluid continuity equations are solved simultaneously. ABAQUS was selected for the analysis in order to take advantage of its robustness in numerical solution strategy for soil nonlinearity and stress–pore pressure coupled problems (Helwany, 2007).

Figure 2(a) below shows the axisymmetric unit cell model considered in the analyses. Size of the mesh was decided based on trial analyses with different number of elements. As the analysis is effective stress analysis involving both fluid flow and stresses, the analyses were performed using special coupled displacement/pore pressure elements (Figure 2(b)) available in Abaqus/Standard for modeling the clay soil with both displacements (u, v) and pore pressure (p) degrees of freedom (dofs) at the corner nodes and only the displacements at the interior nodes. Eight node stress–pore pressure coupled axisymmetric elements with reduced integration (CAX8RP) were used to represent the clay layer. The embankment fill, reinforced gravel layer and the surface coarse-grained fill were assumed to behave in a drained manner. The piles are considered to be fully impermeable. Hence, eight node stress only elements (CAX8R) were used for these materials.

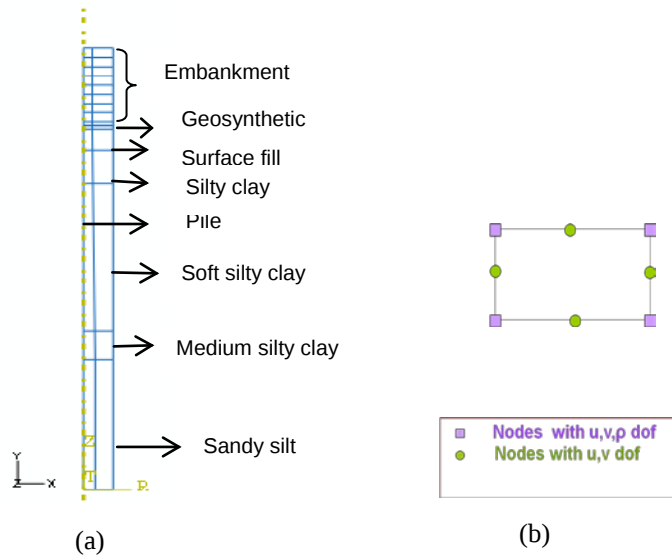


Figure 2. (a)Axisymmetric model used in the analysis; (b) Coupled elements

The geosynthetic reinforcement was modeled using the membrane elements, MAX2. The membrane elements are surface elements that offer strength in the plane of the element but have no bending stiffness. They are useful in modeling the geosynthetic encasement as they offer resistance against out-of-plane bulging (Yoo and Kim, 2009). The “top” surface of a membrane is the surface in the positive normal direction and is called the SPOS face for contact definition. The “bottom” surface is in the negative direction along the normal and is called the SNEG face for contact definition. For axisymmetric membrane elements the positive normal is defined by a 90° counterclockwise rotation from the direction going from node 1 to node 2 as shown in Figure 3.

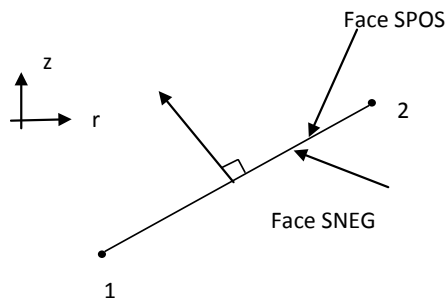


Figure 3. Positive normal for axisymmetric membrane.

2.3 Boundary conditions

The model was horizontally fixed on the vertical sides and full fixity on the base was assumed. The two vertical boundaries and the bottom surface were treated as impermeable boundaries. With regard to the drainage boundary conditions, the water table was assumed to be at a depth of 1.5 m below ground level and the initial pore pressures prior to the embankment construction are taken to be hydrostatic. A zero pore pressure boundary condition was applied at the top boundary of the clay layer to model free drainage.

2.4 Constitutive model

A linear-elastic, perfectly plastic model with the Mohr–Coulomb failure criterion was used to model the embankment fill (PFA), gravel and the surface coarse-grained fill. The soft clay was modeled using the Modified Cam Clay material model as Cam Clay model can detect failure within the clay layer (Helwany, 2007). The pile was modeled as an isotropic linear elastic material with a Young's Modulus of 20 GPa, and a Poisson's ratio of 0.15. The geogrid was modeled as an isotropic linear elastic material with a tensile stiffness of 1180 kN/m and a Poisson's ratio of 0.3.

2.5 Contact interaction

Surface to surface contact was used to model the interaction between the reinforcement and embankment. Surface-based contact simulations generally need to define mechanical contact property models in two directions: normal direction and tangential direction. The “hard contact” is assumed in normal direction. To model the tangential behaviour between the gravel and the geogrid, yield stress was determined by the Mohr–Coulomb failure criterion with zero cohesion. An elastic slip of 1 mm was used to prescribe the allowable relative displacement along the interface of soil and reinforcement.

2.6 Methodology

In Abaqus/Standard, pore pressures can be accounted for either by a total pore pressure formulation or by an excess pore pressure formulation. For the present analysis an excess pore pressure formulation has been used. Initially the clay layer is constructed in one step and its effective self-weight is applied using the body force option as shown in Figure 4(a). The embankment construction was then simulated in nine lifts including the composite reinforced bearing layer of 0.5 m by adding layers of elements representing the embankment. The embankment was constructed to a height of 5.6 m over a period of about 55 days. Figure 4(b) and 4(c) shows the vertical stresses (S22) developed at different stages of embankment construction. After full placement of the embankment, the analysis was carried out until the excess pore water pressure fell below 1 kPa.

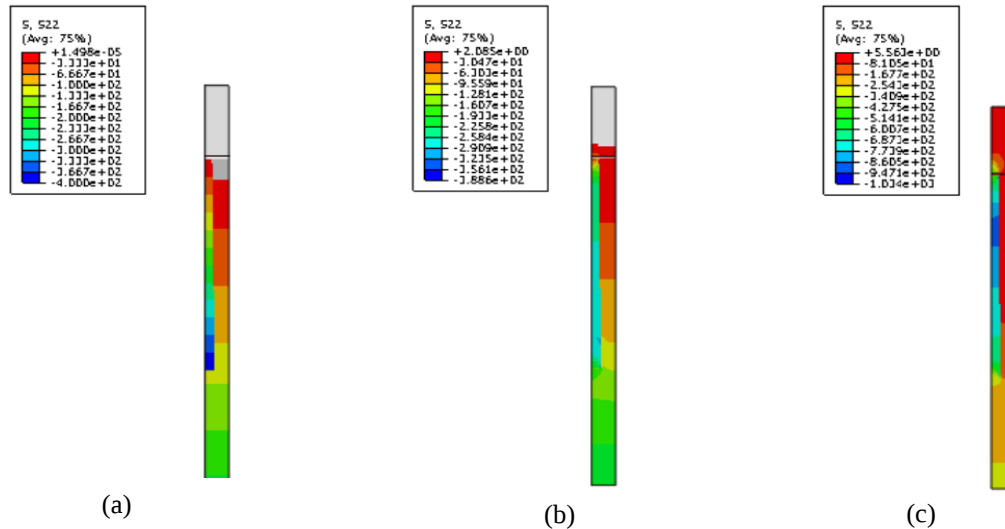


Figure 4. (a) Vertical stress (S22) on soil at the end of geostatic step ;(b) Stresses developed after construction of 1.14m embankment (c) Stresses developed at the end of 5.6m.

3. Results

Piles of lengths 16m, 14m and 12m were considered in the analyses. In all the cases piles act as friction piles. In the case of friction piles load is transferred to the soil through friction with the soil surrounding the pile. Pressure acting on the pile head and the soil surface, settlements and the tension in the geosynthetic are summarized and presented in this section.

3.1 Pressure acting on the pile and soil surface

Pressure acting on the soil surface and at the pile head was plotted at each stage of construction. Figure 5. below shows the increase in pressure acting on the pile due to the arching effect of geosynthetics. Because of arching, there was load transfer from the soft soil to the concrete pile. The amount of stress concentration from the soil to the piles can be quantified using Stress Concentration Ratio, which is the ratio of an average vertical stress on top of a pile to an average vertical stress applied on the foundation soil. The stress concentration ratio in this study comes to about 8.

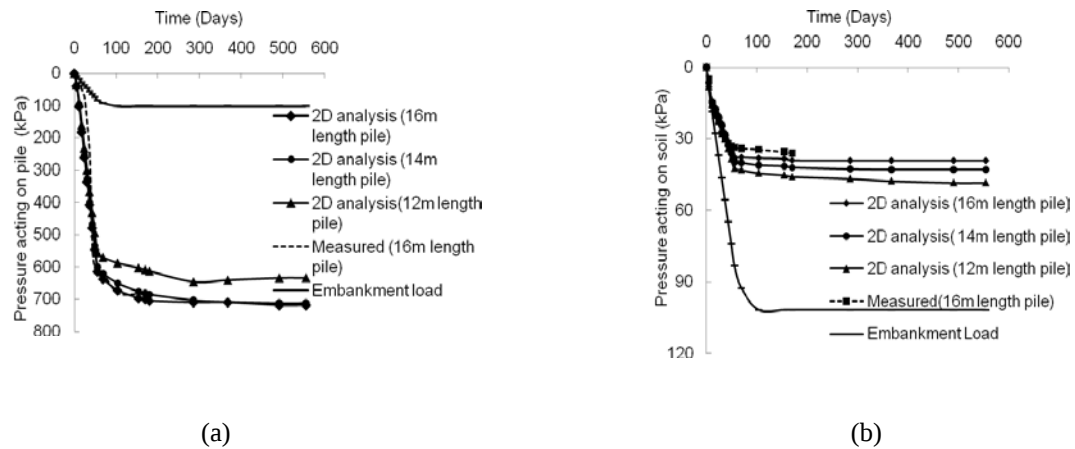


Figure 5: (a) Pressure acting on the top of the pile; (b) Pressure acting on the soil surface between the piles.

For an unreinforced embankment, when the embankment height was increased to 5.6 m the embankment pressure was about 104 kPa. i.e. the inclusion of geosynthetic reinforcement had enhanced the stress transfer from the soil to the piles.

According to Liu et al. (2007) the measured pressures(E4) acting on the soil surface at a height of 5.6m was about 40 kPa, whereas, the pressure acting on the pile head increased to 674 kPa which is 6.5 times larger than the embankment load. From the 2D analyses, the pressure acting on the soil surface was 37.6 kPa and that on the pile head was 614kPa. When the pile length was reduced to 14m and then again reduced to 12m, the pressure acting on the soil surface increased from 39.6 kPa to 42.4kPa. The pressure transferred onto the pile head decreased from 599.7kPa to 554.5kPa when pile length got reduced from 14 to 12m.

3.2 Settlements on the pile and soil surface

Maximum settlement of the soil occurred at the midpoint between the piles. At the end of the construction of the embankment, the maximum measured settlements (Liu et al., 2007) were 14 and 63 mm at the pile head and on soil surface. The measured maximum settlements increased to 19 and 87 mm, 125 days after the completion of the embankment due to soil consolidation. From axisymmetric unit cell model, the settlements at the end of construction were about 11 and 54mm respectively for a pile of length 16m. This settlement increased to about 18.7 and 80mm 125 days after completion of the embankment as shown in Figure 6. Surface settlements at the pile head and soil surface increased to 18 and 58mm at the end of embankment construction (55days) for a pile of length 14m. When the pile length was reduced to 12m, the settlement at the pile head increased to 24.8mm and the soil settlement increased to 62mm (Figure 7).

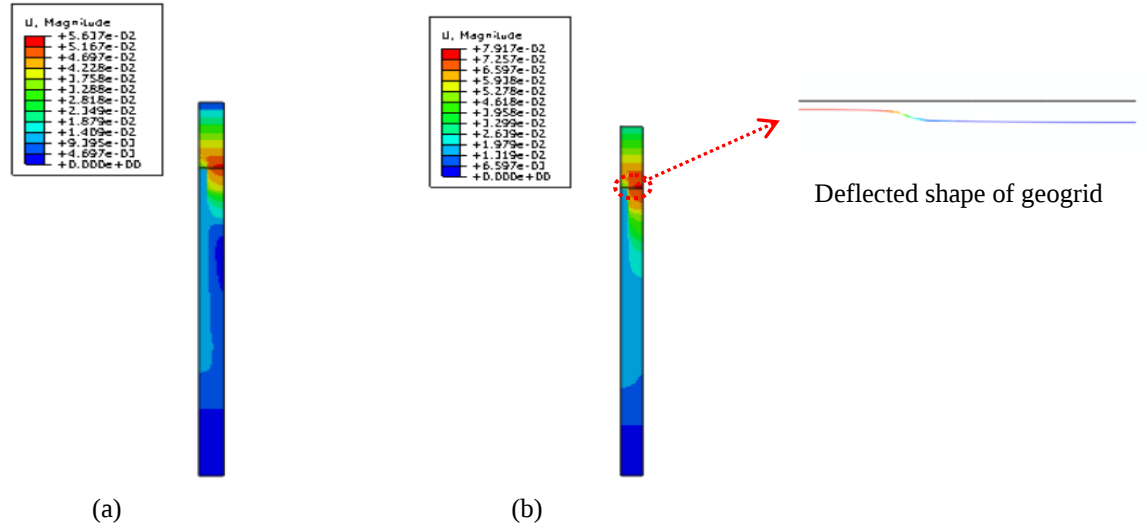


Figure 6: Settlement (U2) with pile length 16m (a) at the end of 55 days; (b) at the end of 180days.

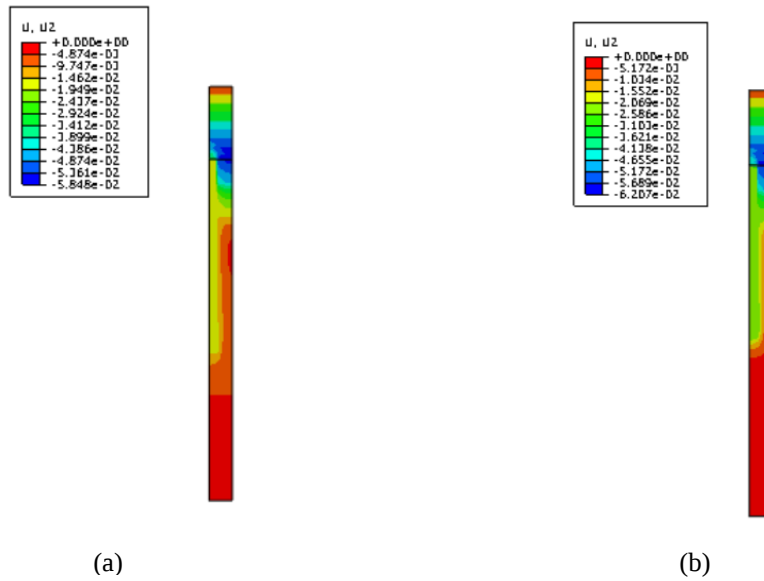


Figure 7: Settlement (U2) at the end of 55 days (a) with 14m length pile; (b) with 12m length pile

3.2 Tensile forces in geogrid

Figure 8 shows the computed tensile force developed in the geogrid around the pile at the end of the embankment construction. It should be noted that the diameter of each pile was 1 m and the pile spacing was 3 m center to center. It can be seen from Figure 8 that the maximum computed tensile force developed in the geogrid is about 16 kN/m from 3Dimensional numerical analysis. Whereas from the 2Dimensional unit cell analysis the maximum tensile force obtained was about 12.8kPa. ie. the Axisymmetric unit cell model tended to give 20% smaller results than the 3D models as far the geosynthetic tension is concerned.

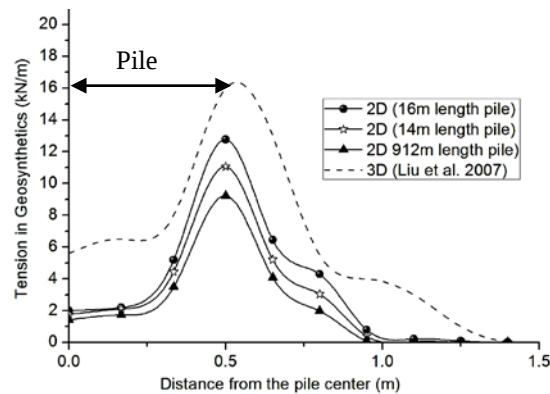


Figure 8. Computed geogrid tensile force around the pile at the end of embankment construction

From the curves in Figure 8 it is clear that the tension is not uniform along the geosynthetic and the maximum tension occurs at the edge of the pile. It was also observed that the maximum tension in the geosynthetic increased with the height of embankment fill.

4. Conclusion

Suitability of the unit cell approach for modeling geosynthetic reinforced piled embankment systems was examined by comparing the results from axisymmetric analysis with those from the full 3Dimensional model (Liu et al., 2007) in terms of stresses, settlement and strains in the geogrid. The axisymmetric unit cell model gave 10-20% smaller results than the fully 3D models. It was observed that for GRPES installed in soft ground, the use of geosynthetic reinforcement layer above the piles decreased the embankment induced vertical stresses in the clay soil which in turn reduced the associated settlement. This is attributed to the increased stiffness of the

foundation system due to the additional reinforcement effect of the geogrid layer. Stress concentration ratio and maximum tension in geosynthetic was also observed to increase with increasing in the height of embankment fill. The axisymmetric unit cell approach based numerical study on GRPES supported by friction piles showed that when the pile length was reduced from 16 m to 14 m with the pile tip resting on medium silty clay, there was not much increase in settlement. But pile settled to very large magnitudes when the pile length was further reduced to 12m with the pile tip resting on soft clay.

5. References

1. "ABAQUS User's Manual," SIMULIA, Pawtucket, Rhode Island, 2009.
2. Biot, M. A., "General Theory of 3-Dimensional Consolidation," Journal of Applied Physics, 12, 155-164, 1941.
3. BS 8006, "Code of Practice for Strengthened/Reinforced Soils and Other Fills," British Standards Institution, London, 1995.
4. Chen .Y. , W. Cao. , and R.P. Chen, "An Experimental Investigation of Soil Arching Within Basal Reinforced and Unreinforced Piled Embankments," Geotextiles and Geomembranes, 26, 164–174, 2008.
5. Jenner, C.G., R.A. Austin., and D. Buckland, "Embankment Support Over Piles Using Geogrids," Sixth International Conference on Geosynthetics, Atlanta, Georgia, 763-766, 1998.
6. Han, J., and M.A. Gabr, "Numerical Analysis of Geosynthetic-Reinforced and Pile-Supported Earth Platforms Over Soft Soil," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 128(1), 44-53, 2002.
7. Helwany, S., "Applied Soil Mechanics with Abaqus Applications," John Wiley & Sons, New Jersey, 2007.
8. Hewlett, W.J., and M.F. Randolph, "Analysis of Piled Embankments," Ground Engineering, 21(3), 12-18, 1988.
9. Huang, J., and J. Han, "Two-Dimensional Parametric Study of Geosynthetic-Reinforced Column-Supported Embankments by Coupled Hydraulic and Mechanical Modeling," Computers and Geotechnics, 37(5), 638-648, 2010.
10. Kempton, G.T., D. Russell, N. Pierpoint, and C.J.F.P. Jones, " Two and Three Dimensional Numerical Analysis of The Performance of Piled Embankments, 6th International Conference on Geosynthetics, Atlanta, Georgia, 767-772, 1998.
11. Liu, H.L., W.W. Charles, and K. Fei, "Performance of A Geogrid-Reinforced and Pile-Supported Highway Embankment Over Soft Clays-Case Study," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 133(12), 1483-1493, 2007.
12. Russell, D., and N. Pierpoint "An Assessment of Design Methods for Piled Embankments," Ground Engineering, 30(11), 39-44, 1997.
13. Smith, M., and G. Filz, "Axisymmetric Numerical Modeling of A Unit Cell in Geosynthetic-Reinforced, Column-Supported Embankments, Geosynthetics International, 14(1), 13–22, 2007.

14. Yoo, C. and S.B. Kim, "Numerical Modeling of Geosynthetic Encased Stone Columns, Geosynthetic International, 16(3), 116–126, 2009.
15. Terzaghi, K., "Theoretical Soil mechanics," Wiley, New York, 1943.