



Performance evaluation of a geosynthetic reinforced pile-supported highway embankment over soft clay

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ABSTRACT

In this paper, a case history of a geosynthetic reinforced pile-supported embankment over soft clay is numerically simulated using a fully coupled finite element analysis. Field monitored data for pressures transferred from the embankment to the piles and soft foundation soil, excess pore pressures, settlements of piles and soil, and lateral deformations are compared with computed data. Results show that a significant amount of embankment load is carried by the piles and hence the excess pore pressures generated in the soil is remarkably low. The lateral effects are significant closer to the toe of the embankment, even though they are not generally appreciated in current practice.

RÉSUMÉ

Dans ce papier, une anamnèse de digue soutenue de tas renforcée d'un geosynthetic sur la glaise molle est numériquement simulée en utilisant une analyse d'élément finie complètement double. Le champ a contrôlé des données pour les pressions transférées de la digue aux tas et au sol de fondation mou, l'excès étudient soigneusement des pressions, les règlements de tas et de sol et les déformations latérales sont par rapport aux données calculées. Les résultats montrent qu'une quantité significative de charge de digue est portée par les tas et dorénavant l'excès étudie soigneusement les pressions produites dans le sol sont remarquablement basses. Les effets latéraux sont significatifs plus près à l'orteil de la digue, bien qu'ils ne soient pas généralement appréciés dans la pratique actuelle.

1 INTRODUCTION

Due to low bearing capacity and excessive settlement characteristics of soft soils, geotechnical engineers dealing with embankment construction over soft ground encounter a real challenge. As a result, a wide range of technologies has been developed to support embankments constructed on soft ground. Although vertical drains and preloading can be used to mitigate unfavourable characteristics of soft soils, it is often not economical to allow the soil to gain stiffness and strength through consolidation due to time constraints and uncertainty of soil conditions. Therefore alternative innovative construction solutions are needed. One such solution is geosynthetic reinforced pile-supported embankments, which is an attractive sustainable technology for 'fast-track' construction environments.

The inclusion of geosynthetics in pile-supported embankments has several advantages. It significantly improves the efficiency of load transfer to the piles and minimises the yielding of the soil above pile heads. It also potentially reduces the total and differential settlements, which can be reflected on to the surface of the embankment. Since pile spacing can be increased with the inclusion of geosynthetics, significant cost reductions can be achieved compared with conventional pile-supported embankments (Han and Akins 2002).

Since the early 1980s, several theoretical (e.g., Finite element method, finite difference method and analytical solutions based on principles of soil mechanics) and experimental studies have been carried out around the world to understand the role of geosynthetics on the load transfer mechanism. In a majority of the currently available design methods it is assumed that the

embankment load is transferred to the piles by the soil arching effect described by Terzaghi (1943) based on his classical trap door experiment. For the pile-supported embankment systems, Hewlett and Randolph (1988) carried out model tests and observed the formation of "vaults" across adjacent pile caps and developed expressions to obtain the load carried by piles.

In geosynthetic reinforced pile-supported embankments, the inclusion of the geosynthetic is expected to reduce the displacement of the embankment fill between the piles and thereby reduce the load transferred to the foundation soil through the soil arching effect. At the same time, the load on the piles may be increased by the vertical component of the tension in the geosynthetic. Therefore an accurate assessment of the load carried by the geosynthetic is essential not only for the design of piles but also for the design of the geosynthetic reinforcement.

In this research a fully coupled finite element analysis has been used to analyse a case history of a geosynthetic reinforced pile-supported embankment over soft soil. The site is located in Shanghai, China. The deformation characteristics of the piled embankment are investigated at the end of embankment construction and 400 days after the construction when the deformations due to consolidation are assumed to be complete. The computed results are discussed and compared with field measurements reported by Liu et al., (2007).

2 EMBANKMENT AND SITE CONDITIONS

Figure 1 shows the soil profile and the cross section of the instrumented test embankment. The ground water level is at 1.5 m below the ground surface.

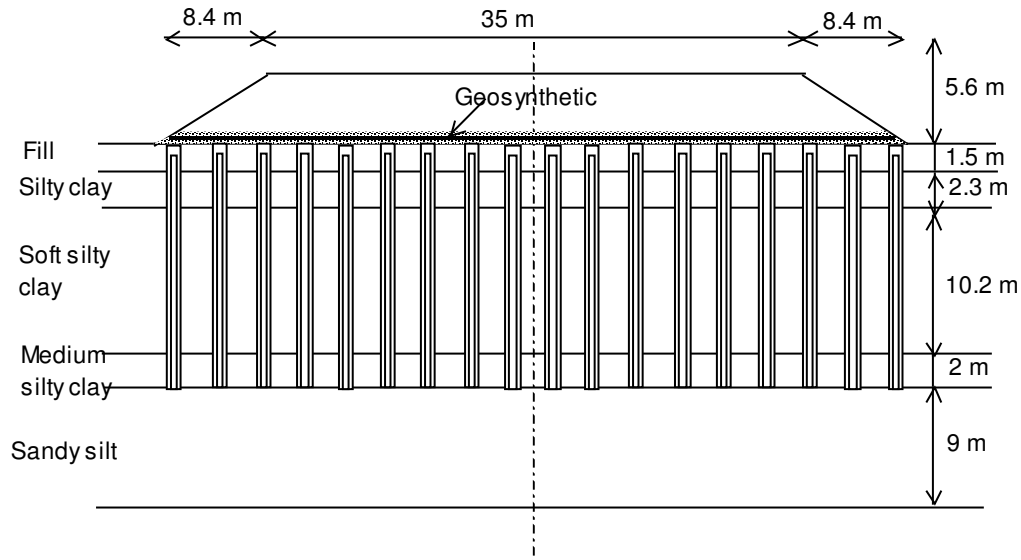


Figure 1. Cross section of highway embankment.

The embankment is supported by concrete piles with outer diameter of 1.0 m. Piles are 16 m long and founded on a sandy silt layer, which is relatively stiff compared to the silty clay layers above. The top 0.5 m of the pile section is solid cylindrical but the rest of the pile section is annulus with a wall thickness of 120 mm. The centre to centre distance between piles is 3 m.

Above the pile heads a 0.25 m thick well-graded gravel layer is placed to provide a working layer and to avoid damage to the geosynthetic above the pile heads (Liu et al., 2007). Another 0.25 m thick gravel layer is placed above the geosynthetic layer, making the total thickness of the gravel layer above the pile heads to 0.5 m.

3 NUMERICAL MODEL

Figure 2 shows the finite element mesh used for the analysis. By taking into account the symmetry of the problem, only one half of the embankment is analysed assuming plane strain conditions. The numerical model is based on the coupled theory of nonlinear porous media and the finite element analysis is carried out using ABAQUS/ Standard. It allows to investigate the excess pore pressure generation during the embankment construction and subsequent excess pore pressure dissipation during consolidation. The constitutive behaviour of the four silt layers are modelled using the Modified Cam Clay model, which requires six material parameters: slope of the virgin consolidation line, λ ; slope of the swelling line, κ ; the void ratio at unit pressure, e_1 ; slope of the critical state line, M ; Poisson's ratio, ν .

The embankment is constructed to a height of 5.6 m in ten lifts over a period of 55 days and the history of embankment construction is shown in Figure 3. Normally embankment fill materials have a relatively high permeability. Therefore, the fill material is assumed to behave in a drained manner. The constitutive behaviour

of the fill material and gravel layers are modelled using a linear elastic-perfectly plastic model with Mohr-Coulomb failure criterion, which requires five parameters: effective cohesion, c' ; friction angle, ϕ' ; dilatancy angle, ψ' ; Young's modulus, E ; and Poisson's ratio, ν .

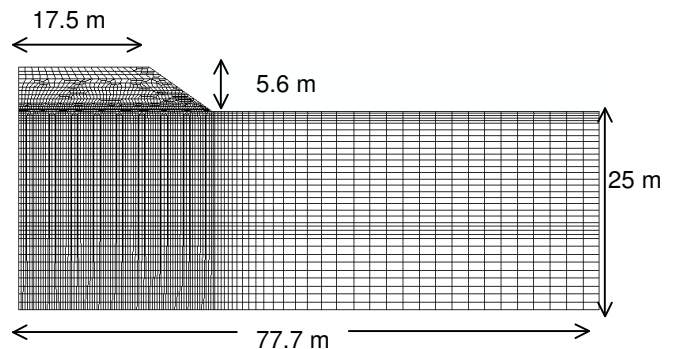


Figure 2. Finite element mesh.

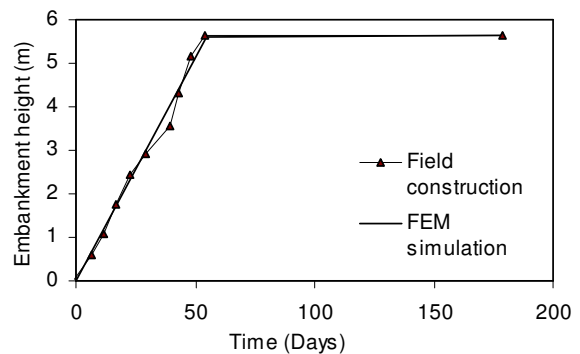


Figure 3. Embankment construction history.

The pile and the geosynthetic reinforcement are modelled as linear elastic materials. The parameters used in the analyses are listed in Table 1.

In the first increment block of the analysis, the finite element mesh corresponding to the embankment is removed and an initial stress state is established in the foundation soil based on the unit weight of each soil layer. Then the embankment construction is simulated by adding a set of elements corresponding to each lift of the fill material within a duration of 5.5 days. After adding ten lifts of the fill material, the foundation soil is allowed to consolidate for a period of 400 days.

Table 1. Properties of soil, pile and geosynthetic

Material	Parameters
Silty clay	$\lambda = 0.06$, $\kappa = 0.012$, $M = 1.2$, $e_1 = 0.87$, $OCR = 1$, $\nu = 0.35$, $k = 8.64 \times 10^{-4}$ m/day, $\gamma = 19.8$ kN/m ³
Soft silty clay	$\lambda = 0.15$, $\kappa = 0.03$, $M = 0.95$, $e_1 = 1.79$, $OCR = 1$, $\nu = 0.4$, $k = 4.32 \times 10^{-4}$ m/day, $\gamma = 17.3$ kN/m ³
Medium silty clay	$\lambda = 0.05$, $\kappa = 0.01$, $M = 1.1$, $e_1 = 0.88$, $OCR = 1$, $\nu = 0.35$, $k = 4.32 \times 10^{-4}$ m/day, $\gamma = 20$ kN/m ³
Sandy silt	$\lambda = 0.03$, $\kappa = 0.005$, $M = 0.28$, $e_1 = 0.97$, $OCR = 1$, $\nu = 0.35$, $k = 4.32 \times 10^{-3}$ m/day, $\gamma = 19.8$ kN/m ³
Embankment	$E = 20$ MPa, $c' = 10$ kPa, $\phi' = 30^\circ$, $\psi = 0$, $\nu = 0.3$, $\gamma = 18.5$ kN/m ³
Gravel	$E = 20$ MPa, $c' = 10$ kPa, $\phi' = 40^\circ$, $\psi = 0$, $\nu = 0.3$
Coarse grained fill	$E = 7$ MPa, $c' = 15$ kPa, $\phi' = 28^\circ$, $\psi = 0$, $\nu = 0.3$, $\gamma = 18$ kN/m ³
Pile	$E = 20$ GPa, $\nu = 0.2$
Geosynthetic	$J = Et = 1180$ kN/m, $\nu = 0.3$

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

4 LOAD TRANSFER FROM THE EMBANKMENT TO PILES

Figure 4 shows the computed pressure acting on the soil surface between the first and the second piles, and the second and the third piles. The measured pressure is also shown in Figure 4. Although the pressure acting on the soil around the second pile is measured during the embankment construction, the measured value between the first and the second piles are shown in Figure 4 because the finite element analysis is carried out assuming plane strain conditions. Also Figure 4 shows embankment load, γH , where, H is the embankment height and γ is the unit weight of the fill material. The computed pressure acting on the foundation soil agrees well with the measured data and the pressure applied on the foundation soil is about 37% of the embankment load.

Figure 5 shows the pressure acting on pile 2. The average pressure acting on the pile head is 680 kPa, which is about 6.6 times the embankment load. Figure 6 shows the vertical stress acting on the geosynthetic and it reaches a maximum of 170 kPa at the pile head and decreases to 40 kPa mid-span between piles, which is nearly 40% of the embankment load. At the mid-span between piles, the vertical pressure acting on the foundation soil and the geosynthetic are nearly the same. These values clearly show that a large portion of embankment load is transferred to the piles and only about 40% of the embankment load is transferred to the foundation soil.

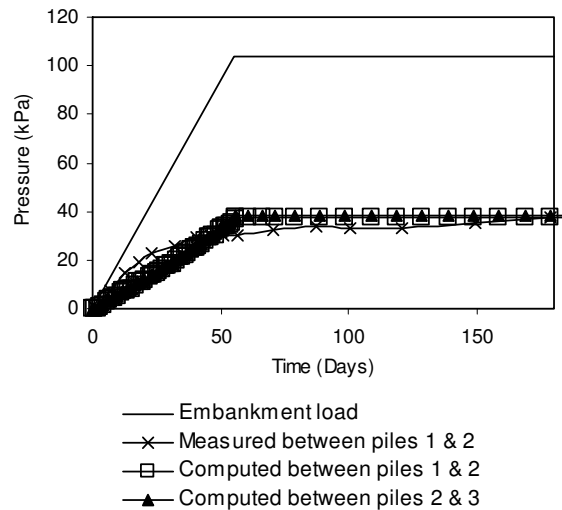


Figure 4. Pressure acting on the soil surface between piles.

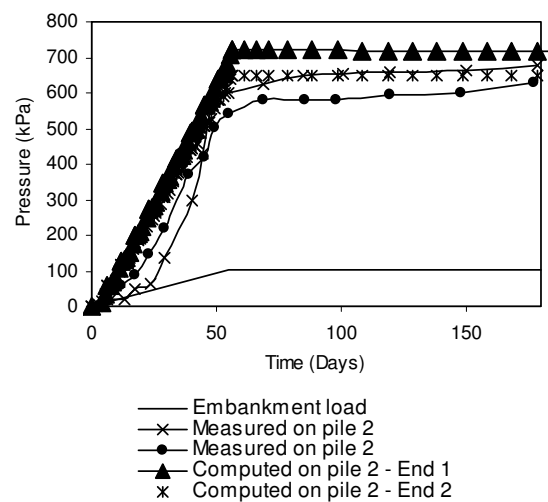


Figure 5. Pressure acting on the piles.

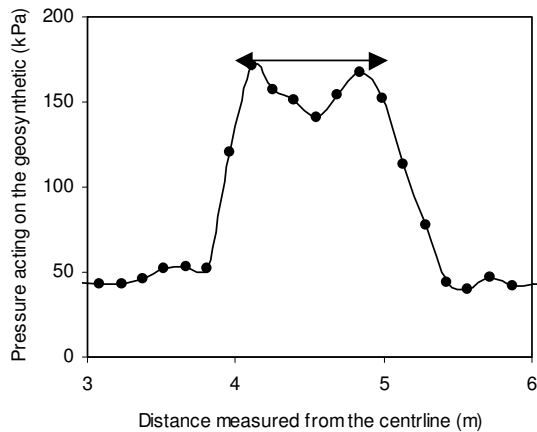


Figure 6. Pressure acting on the geosynthetic at the end of embankment construction.

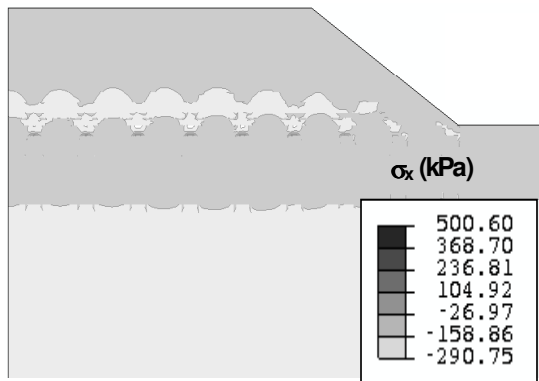


Figure 7. Horizontal stresses above piles at the end of embankment construction.

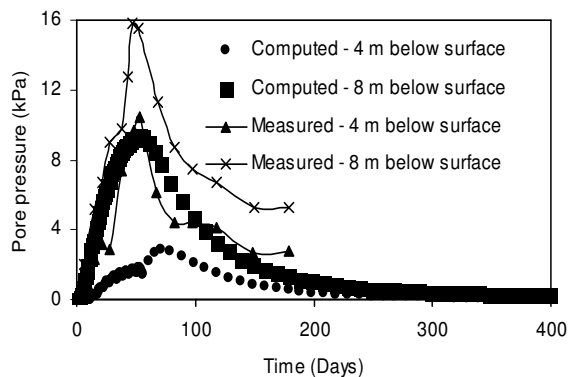


Figure 8. Measured and computed pore pressure between pile 2 and pile 3.

Figure 7 shows the horizontal stress distribution within the embankment. This figure clearly illustrates the dome shaped horizontal stress contours in between piles in the fill material. In a vertically loaded arch, the horizontal component of the stress is a constant. Therefore this figure confirms soil arching, with the majority of the embankment load being transferred to the piles.

The load transfer mechanism due to soil arching can be quantified using the stress concentration ratio, which is defined as the ratio between the average vertical stress applied on piles to the average vertical stress applied on the foundation soil (Liu et al. 2007). The computed stress concentration ratio for the problem analysed in this study is about 16.5, while Liu et al. (2007) reported a stress concentration ratio of 14 for the same highway embankment, based on measured data. In the field, pressure acting on foundation soil is computed as the average of pressures measured around the second pile but in this finite element study, the pressure is computed based on the average of pressures computed on either side of pile 2 due to the two-dimensional idealisation of the actual three-dimensional problem.

Barksdale and Goughnour (1984) and Greenwood (1991) reported that the stress concentration ratios for piled embankments without any geosynthetic reinforcement to be typically in the range of 1 to 8. However, they used stone columns as piles. The contrast between the stress concentration ratios may be due to the difference in stiffness between the pile material and the foundation soil as well as the presence of the geosynthetic. When geosynthetic reinforcement is used in piled embankments, the vertical component of the tension in the geosynthetic will also contribute to the load transferred to the piles. Hence, the stress concentration ratio for geosynthetic reinforced piled embankments should be higher than that of piled embankments without geosynthetic reinforcement.

5 VARIATION OF EXCESS PORE PRESSURE

Figure 8 shows the variation of excess pore pressure in the foundation soil between pile 2 and pile 3 at the end of embankment construction after 55 days at 4 m and 8 m below the surface. The computed pore pressure at 4 m below the surface, which is closer to the top of the soft silty clay layer and 2.5 m below the zero pore pressure boundary, is about 30% of the measured value. At 8 m depth, which is closer to the mid-depth of the soft silty clay layer, the measured excess pore pressure is about 60% of that measured. Although piles used in the embankment are cast-in-place piles, a temporary double-wall casing is driven into the ground by a vibrating machine. During this process also excess pore pressures may have generated in the ground. However, the excess pore pressures computed based on the finite element analysis simulated only the pore pressure generation during embankment construction. This may be the reason for low computed excess pore pressures shown in Figure 8 compared to the field measured excess pore pressures.

Figure 9 shows the excess pore pressure distribution at the end of embankment construction. Excess pore pressure is a maximum beneath the centre of the embankment. Although the increase in surcharge load is

about 100 kPa, the maximum increase in excess pore pressure in the foundation soil is about 20 kPa. The relatively small pore pressure increase compared to the increase in surcharge load is due to the load transfer from the embankment to the piles beneath it due to soil arching. Foundation soil is subjected to smaller compressive loads and therefore the pore pressure increase in foundation soil is small. In addition, there will be some excess pore pressure dissipation during the embankment construction, which took as mentioned before 55 days.

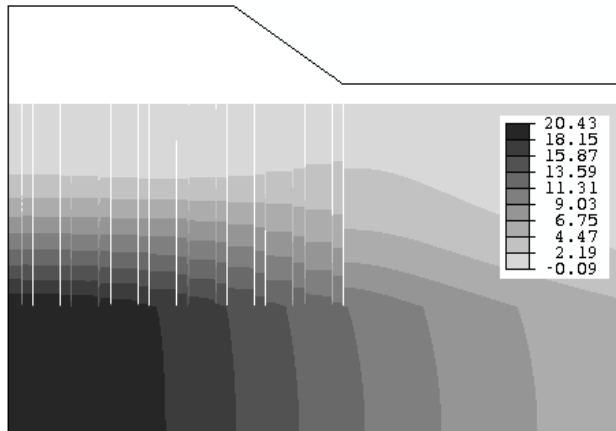


Figure 9. Excess pore pressure distribution in foundation soil at the end of embankment construction.

6 TOTAL AND DIFFERENTIAL SETTLEMENT

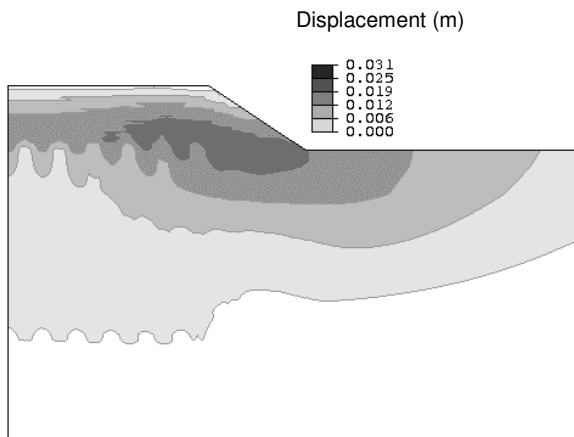


Figure 10. Excess pore pressure distribution in foundation soil at the end of embankment construction.

Figure 10 shows the displacement contours at the end of embankment construction. Closer to the toe of the embankment, displacement of soil and piles are comparable. However, when moving towards the centre of the embankment, contours show clearly that soil

displacements are greater than pile head displacements. This indicates differential settlement has occurred at the level of pile heads; it has not transferred to the embankment level.

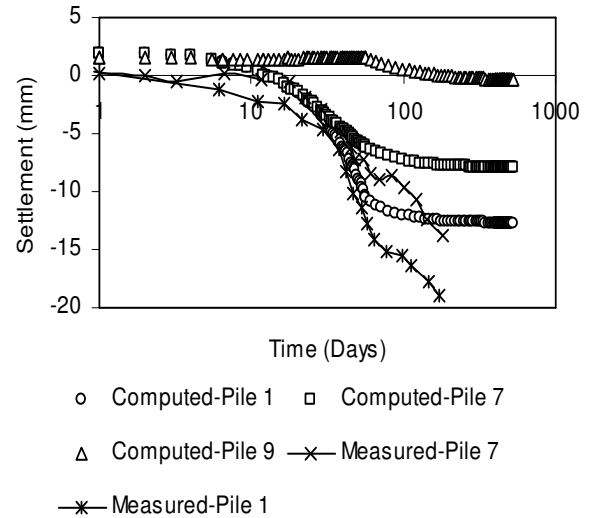


Figure 11. Computed and measured pile settlements.

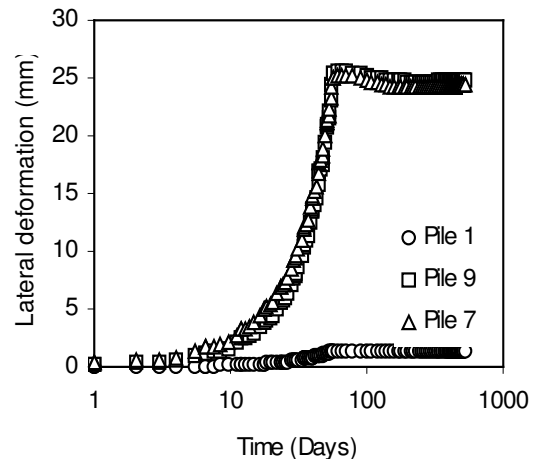


Figure 12. Computed pile lateral deformations.

Figure 11 shows the measured and computed settlements for pile 1 and pile 7. During the period of embankment construction, measured and computed pile settlements agree well. Results show further increase in settlement after embankment construction due to soil consolidation. According to the computed settlements, consolidation completes after about 150 days but the measured results show a rapid increase in settlement due to soil consolidation after the conclusion of the embankment construction. Even after 150 days, the rate

of settlement is significant and has not stabilised. These results indicate that excess pore pressure dissipation is rapid in computer simulation compared to that encountered in the field. During consolidation, voids ratio of the soil decreases and this will result in a decrease in permeability of the soil (Liu et al., 2007). However, this effect has not been taken into account in the numerical simulation. As a result, the numerical simulation shows a rapid pore pressure dissipation.

Also Figure 11 shows settlement of pile 9, which is at the toe of the embankment. Compared to the settlement of pile 7, at the end of consolidation, settlement of pile 9 is negligible.

Figure 12 shows lateral deformations of the pile 1, pile 7 and pile 9. Lateral deformation is critical for the pile closest to the embankment toe and at the end of embankment construction the lateral deformation of the pile closest to the toe is 25 mm. However, the computed lateral deformation is 1 mm for the pile 1. The current practice used in the design of geosynthetic reinforced piled embankments, the lateral effects are not taken into consideration although within the profession there is general awareness about this issue. Results of this study clearly show that the lateral effects are significant especially for piles and soil closer to the toe of the embankment.

7 DESIGN TECHNIQUES

The main difficulty in the design of geosynthetic reinforced pile supported embankments is the calculation of vertical load transferred from the embankment to the geosynthetic, piles and foundation soil. Still the mechanism of load transfer from the embankment to the geosynthetic is not well understood. It is generally assumed that the embankment load is transferred to the piles by the geosynthetic and by soil arching (Russell and Pierpoint 1997). In this section, four design methods are compared with the finite element results obtained for the case history described in previous sections: BS8006, Terzaghi's arching theory (Terzaghi 1943), Hewlett and Randolph's arching theory (Hewlett and Randolph 1988) and Guido et al.'s design method (Guido et al. 1987). These methods have been compared using the arching ratio, S , which is defined as,

$$S = \frac{P_r}{\gamma H} \quad [1]$$

where, P_r is the average vertical stress carried by the geosynthetic.

7.1 BS8006

The method given in BS8006 has been developed by Jones et al. (1990). The load carried by the piles is computed based on the Marston's formula for positively projecting conduits (Russell and Pierpoint 1997). For embankment heights greater than $1.4(s-a)$, where s is the

pile spacing and a is the pile cap width, the distributed load applied on the geosynthetic is given by,

$$P_r = \frac{2.8s\gamma}{(s+a)^2} \left[s^2 - a^2 \left(\frac{P_c}{\gamma H} \right) \right] \quad [2]$$

where s is the pile spacing, a is the pile cap width and,

$$\frac{P_c}{\gamma H} = \left(\frac{C_c a}{H} \right)^2 \quad [3]$$

where C_c is the arching coefficient. For the friction piles used in this study,

$$C_c = 1.5 \frac{H}{a} - 0.07 \quad [4]$$

7.2 Terzaghi's arching theory

Terzaghi (1943) examined soil arching effects using his classical trap door experiment. Russell and Pierpoint (1997) have extended Terzaghi's analysis for piled embankments with geosynthetic. They derived an expression for the distributed load applied on the geosynthetic due to arching given by,

$$P_r = \frac{(s^2 - a^2)\gamma}{4aK \tan \phi'} \left[1 - e^{\frac{-4aHK \tan \phi'}{(s^2 - a^2)}} \right] \quad [5]$$

where K is the coefficient of earth pressure at rest. K can be related to friction angle of the soil ϕ' by,

$$K = 1 - \sin \phi' \quad [6]$$

7.3 Hewlett and Randolph's arching theory

For the pile-supported embankment systems, Hewlett and Randolph (1988) carried out model tests and observed the formation of vaults across adjacent pile caps and developed expressions to obtain the load carried by piles. In the design, it is assumed that the proportion of the embankment load that has not been transferred on to the pile caps has been supported by the geosynthetic. The critical location for the vertical stress is located either at the crown of the arch or at the pile. The average pressure applied on the geosynthetic based on the conditions at the crown of the arch is given by (Russelle and Pierpoint 1997),

$$P_r = \gamma H \left(1 - \frac{a}{s}\right)^{2(K_p-1)} \left[1 - \frac{s}{\sqrt{2H}} \frac{2(K_p-1)}{(2K_p-3)} \right] + \frac{(s-a)}{\sqrt{2H}} \frac{2(K_p-1)}{(2K_p-3)} \quad [7]$$

Based on the conditions at the pile, average pressure applied on the geosynthetic is given by,

$$P_r = \frac{\gamma H}{\left(\frac{2K_p}{K_p+1}\right) \left[\left(1 - \frac{a}{s}\right)^{(1-K_p)} - \left(1 - \frac{a}{s}\right) \left(1 + \frac{a}{s} K_p\right) \right] + \left(1 - \frac{a^2}{s^2}\right)} \quad [8]$$

where K_p is the passive earth pressure coefficient. Usually the higher value given by Equations 7 or 8 is used in the design.

7.4 Guido et al.'s method

The design method used at the Second Severn Crossing embankment in the UK used Guido's method (Russell and Pierpoint 1997). In this method it is assumed that a rectangular pyramid of fill material above the geosynthetic spanning between piles is transferred to the geosynthetic. Then the load carried by the geosynthetic is given by,

$$P_r = \frac{\gamma(s-a)}{3\sqrt{2}} \quad (9)$$

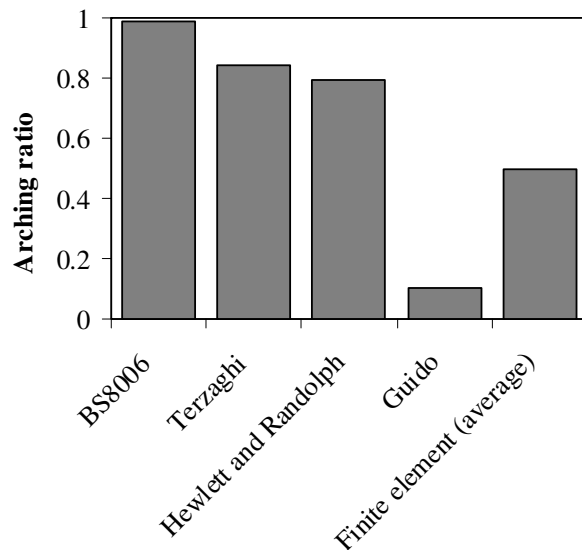


Figure 13. Comparison of different design methods.

Figure 13 compares the values of arching ratio obtained from the design methods described in this section. This

figure clearly shows the inconsistencies in different methods.

Finite element analysis results in an arching ratio of 0.5 whereas BS8006 gives an arching ratio of 0.99, which indicates no soil arching for the example problem. Terzaghi's arching theory and Hewlett and Randolph's method yield similar arching ratios closer to 0.8. However, Guido et al.'s method provides an arching ratio of 0.1, which is the lowest.

The domes of constant horizontal stress demonstrated in Figure 7 based on the finite element analysis clearly demonstrate soil arching and are consistent with an arching ratio of 0.5 obtained from the finite element analysis. Therefore it can be concluded that the BS8006 over-predicts the arching ratio whereas Guido's method under-predicts the soil arching ratio for a geosynthetic reinforced piled-embankment. Terzaghi's method and Hewlett and Randolph's method use arching theory. Therefore, they yield comparable arching ratios, but they do not agree with the finite element results as both methods are developed without incorporating geosynthetic reinforcement. An arching ratio of 0.5 appears more accurate for the case considered.

8 CONCLUSION

A numerical study is carried out to investigate the load transfer mechanism in geosynthetic reinforced pile supported embankments over soft clay. The plane strain conditions assumed for the analysis yielded results in agreement with the measured data during a highway embankment construction in China. Results show that the majority of the embankment load is carried by the piles due to the soil arching effect. The lateral effects have a significant influence on the performance of the embankment, even though they are not incorporated in current practice. Finite element results have been compared with four design techniques used in practice and the results clearly demonstrate the inconsistencies in current practice.

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