A NEW ANALYTICAL METHOD FOR DESIGN OF REINFORCED EMBANKMENT ON RIGID PILE ELEMENTS

Tuan A. Pham[1,2], Pascal Villard[1]

Abstract: *Embankments constructed over soft soils induce a significant load over a large area. The technique of reinforcing soil with piles has proven to be an interesting solution that prevents failure or excessive deformations of embankments. Several simplified design procedures were introduced, but they are still over-conservative in results, yielding uneconomical designs. This study presents an advanced analytical method for design of geosynthetic reinforced piled supported embankment, which based on the combination of arching effect, tensioned membrane action, and shear resistance mechanism. The present method describes the complex behavior and interaction between geosynthetic-soil-pile, thereby providing more suitable design approaches and believed to be a useful tool for engineers in designing soil-geosynthetic system. In addition, the numerical modeling based on discrete element method with the most advanced code description currently has been used to investigate the validity and reliability of the proposed method. Thus, the results of this study are expected to provide some guidelines for designers and to bring insight about the interesting the interacting mechanism into the design process.*

Keywords: piled embankment, geosynthetics, interaction mechanism, the proposed method, design, numerical analysis.

1. INTRODUCTION

Embankments constructed over soft soils induce a significant load over a large area and is a common problem to geotechnical engineers. In recent years a new kind of foundation was established so-called "geosynthetic reinforced pile supported (GRPS) embankments. The technique of reinforcing soil with piles has proven to be an interesting solution that prevents failure or excessive deformations of embankments (Johnes et al., 1990; Kempfert et al., 2004; Jenck et al., 2005; Han et al., 2011;). Piles are driven in a regular screen disposition into the in-situ soil down to bearing soil, transferring the loads directly downwards and decompressing the soft soil significantly. Over the pile caps, one or more layers of geosynthetic will be placed, as shown in Fig. 1a.

This technique combines three components: (1) embankment material, (2) a load transfer platform (LTP), and (3) vertical elements extending from the LTP to the stiff substratum. The surface and embankment loads are partially transferred to the piles by arching that occurs in the granular material constituting the LTP (Fig. 1b). The load re-distribution causes homogenization and the reduction of surface settlements. Although this technique is widely used, the mechanisms involved are still poorly understood.

Soil layers and underlying geosynthetics are assumed initially to be resting on a firm foundation. At some point in time, a void of a certain size opens below the geosynthetic. Under the weight of soil layers and any applied loads, geosynthetics will be deflected. The deflection has two effects, bending of soil layers and stretching of the geosynthetics.

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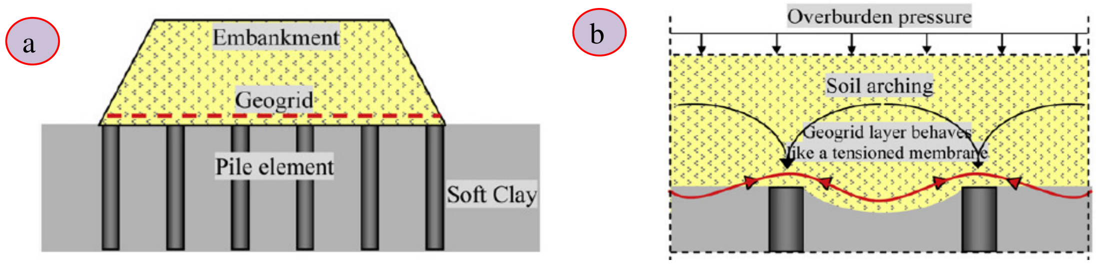

Fig. 1. Geosynthetic reinforced piled supported embankment (after Eskisar et al., 2013)

The bending of the soil layer generates the arching effect inside the soil due to the highly significant difference in stiffness of the piles relative to the surrounded soft soil. As a result, the vertical stresses are concentrated in the area over the piles, simultaneously the stresses over the soft soil reduce and is smaller than the average vertical stress of embankment.

The stretching of the geosynthetic mobilizes a portion of the geosynthetic strength. Consequently, the geosynthetic acts as a "tensioned membrane" and can carry a load applied normally to its surface. As a result of geosynthetic stretching, the load on the piles may be increased by the vertical components of the tension forces in the reinforcement. The redistribution mechanism of loads in the embankment depends generally on the geometry of system, the strength of embankment soil, the stiffness of piles and support of soft subsoil.

Several methods have proposed and currently used for estimating the soil arching effect (e.g.Terzaghi, 1943; Guido et al., 1987; Low et al., 1994; BS8006, 1995; Russell and Pierpoint, 1997; Hewett and Randolph, 1998; EBGEO, 2004; Kempfert et al., 2004; Van Eekelen et al., 2013). However, until now there is no analytical approach which describes precisely this complex behavior of a system consists of the embankment – reinforcement – piles – soft subsoil.

In spite of its limitation, this current study is an attempt to shed more light on the arching phenomenon using a mathematical model and numerical analysis. Simple expressions for the reduced load caused by arching are proposed. In addition, a design method of geosynthetic reinforcement has also proposed in this study by combining arching effect with tensioned membrane action and frictional resistance mechanism, thereby providing suitable and realistic design approach. This method can estimate the degree of arching in the embankment and calculate the required properties of the geosynthetic reinforcement that is a good match with experimental, numerical and field observations.

2. THEORETICAL ANALYSIS

The proposed method in this study is a new analytical model, and a two-step approach is therefore used. First, the behavior of the soil layer is analyzed using arching theory. This step gives the pressure at the base of the soil layer on the portion of the geosynthetic located above the soft ground. Second, tensioned membrane theory is used to establish a relationship between the pressure on the geosynthetic, the tension and strain in the geosynthetic, and the deflection of the geosynthetic. These coupled effects will be considered in a later section.

2.1. Definition

Three related terms are used to assess the degree of arching in an embankment. Firstly, efficacy, E, is the percentage by weight of the embankment fill carried by the pile caps. The stress reduction ratio, SRR, is the ratio of the actual average vertical stress on the soft ground to the value of overburden stress, γH. The stress concentration ratio is the ratio of stress on the pile cap to stress on soft ground. If there is no arching, efficacy is equal to $(A_p/A) \times 100\%$, and the stress reduction ratio equals to 1.0.

$$E = \frac{P_p}{A(\gamma H + q_o)} \times 100\% \quad (1);$$

$$SRR = \frac{P_s}{(A - A_p)(\gamma H + q_o)} = \frac{\sigma_s}{\gamma H + q_o} \quad (2);$$

$$n = \frac{\sigma_p}{\sigma_s} \quad (3)$$

where P_p is the load on a pile-cap; P_s is the load on soft-ground area; A is the tributary area of a pile-cap; A_p is the area of a pile-cap; γ is the unit weight of the fill; H is the embankment height; σ_p is the vertical stress applied on a pile-cap; σ_s is the vertical stress acting soft soil.

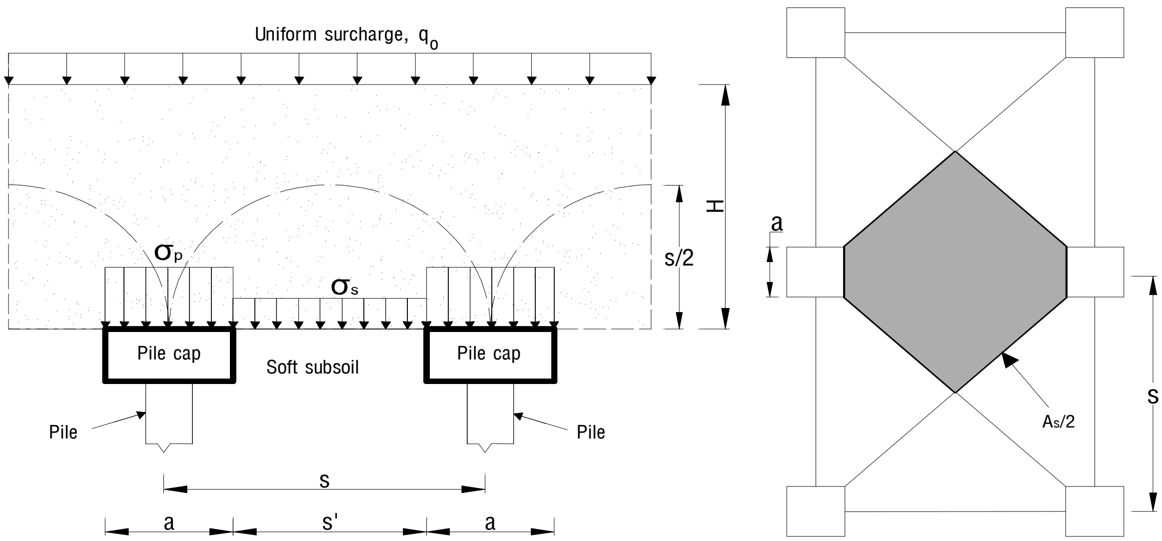

Fig.2. Stress distribution in an area of influence

Taking into account a model of square pattern as shown in Fig. 2. The total pile cap area per an area of influence is $A_p=a^2$ and the remaining area covered by soft ground is ($A_s = s^2-a^2$). The following relationships are easily established:

$$E = 1 - \frac{s^2}{s^2 - a^2} SRR \quad (4);$$

$$\sigma_p == \frac{E.(\gamma H + q_o)s^2}{a^2} \quad (5);$$

$$n = \frac{\sigma_p}{\sigma_s} = \frac{E}{1-E} \frac{s^2 - a^2}{a^2} = \frac{E}{SRR} . \frac{s^2}{a^2} \quad (6)$$

2.2. Step 1. Prediction method of soil arching

As a starting point, the shape of the stable physical arch is presumed to be a circular curve, as depicted in Fig. 3. Let θ be the inclination, concerning the horizontal of the tangent line through each end of the arch spanning width s' of the inclusion.

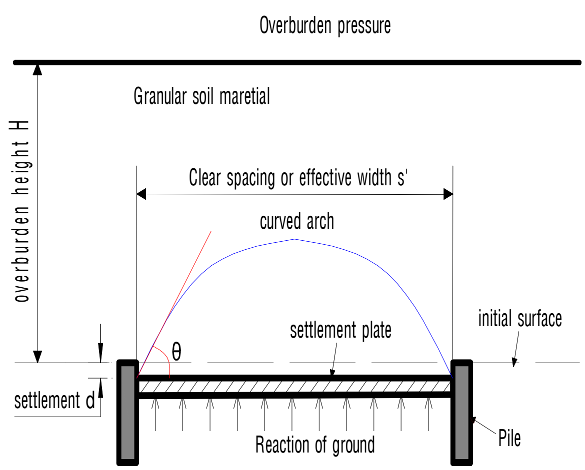

Fig.3. Assumed-shape of the stable physical arch

For a circular arch with an inclination angle θ at the sides with respect to the horizontal, this results in (7) and (8), (after Miki, 1997).

$$p_o = V_s.\gamma / [(s'+a)^2 - \pi d_c^{\,2} / 4] \quad (7)$$

$$V_s = 0.5s'(s'+d_c)^2 \tan\theta - \pi \tan\theta[(s'+d_c)^3 - a^3]/24 + (4-\pi)(\sqrt{2}-1)(s'+d_c)^3 \tan\theta/24 \quad (8a)$$

$$\theta = 90^0 - \phi \quad (8b)$$

$$SRR = P_s/\gamma H_t = \alpha\gamma H_s/\gamma H_t = \alpha V_s/V_t \quad (9)$$

where V_s is the embankment volume acting on the soft ground in-between; V_t is the total volume of embankment; p_o is the pressure acting on the soft ground in-between; α is the arching shape dimension ratio; a is the width of pile-cap; s'is the clear spacing and equal (s-a); SRR is the stress reduction ratio.

The arching shape ratio α based on the basis of empirical model and numerical results is:

$$\alpha = m^{H/s+0.5} \times H^{0.5} \times 1.1^k \quad (10)$$

$$m = 1.3+0.05(H/s) \quad (11)$$

$$k = E_s(1+\nu_p)(1-2\nu_p)/E_p(1+\nu_s)(1-2\nu_s) \quad (12)$$

where H is the embankment height, s is the center to center pile spacing, k is called modular ratio between the pile and soft ground. E_p is the elastic modulus of the pile, ν_p is the Poisson's ratio of the pile material, E_s is the elastic modulus of soft soil, ν_s is Poisson's ratio of soft soil.

Substitute (8), (10) back into (7) gives the vertical stress acting on the soft ground:

$$\sigma_s = \frac{m^{H/s+0.5} \times H^{0.5} \times 1.1^k.(\gamma + q_0/H)}{2(s^2 - \pi d_c^2/4)} \times \left\{ \frac{\pi}{96}(s_x - d_e)(s_y - d_c)\tan\theta(5d_c + 4s_x) + (4-\pi)\left(\frac{s_x \times s_y}{4}\right)\left[\frac{s_x + s_y - 2d_c}{4}\tan\theta + \frac{s_x + s_y}{12}\right](\sqrt{2}-1)\tan\theta \right\} \quad (13)$$

For a square arrangement of piles with $s_x = s_y = s$, the Eq. (13) becomes:

$$\sigma_s = \frac{\alpha(\gamma + q_0/H)}{2(s^2 - \pi d_c^2/4)} \times \left\{ \frac{\pi}{96}(s-d_c)^2 \tan\theta(5d_c + 4s) + (4-\pi)\left(\frac{s}{2}\right)^2 \left[\frac{s-d_c}{2}\tan\theta + \frac{s}{6}\right](\sqrt{2}-1)\tan\theta \right\} \quad (14)$$

where s_x, s_y - pile center-to-center spacing in directions of x and y; d_c – diameter of pile cap

$d_c = d$ for round pile cap; $d_c = 2a/\sqrt{\pi}$ for square pile cap (15)

2.3. Step 2. Analysis method of geosynthetic reinforcement

In the second step, the vertical stress, σ_s, is applied to the geosynthetic reinforcement as an external load. When one or more layers of geosynthetic are placed at the top of pile caps, a possible upwards counter-pressure, σ_{up}, from the partially compressed upper zone of soft subsoil between piles is assumed, which reduces the tension in reinforcement, as shown in Fig. 4. Eq. (16) had to be developed to reflect this interaction.

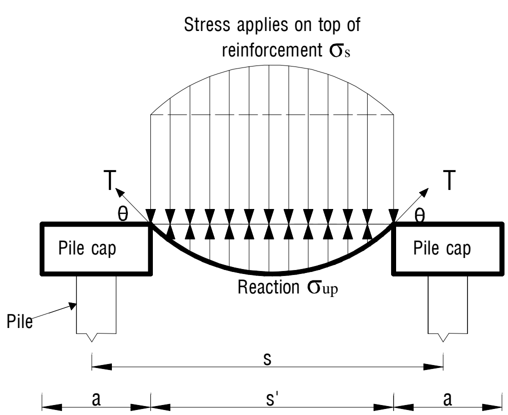

Fig. 4. Idealized stress distribution

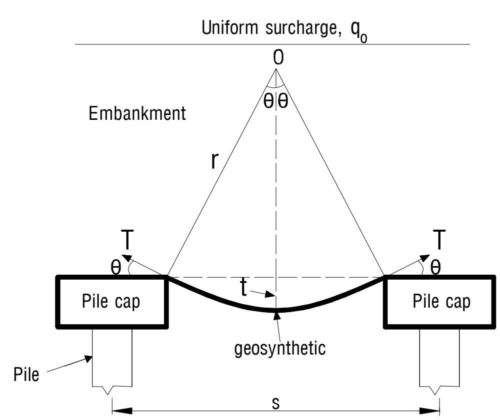

Fig. 5. Assumed-shape of geosynthetic

The influence of bearing effect of the soft subsoil between piles is taken into account by using a modulus of subgrade reaction.

$$\sigma_{GR} = \sigma_s - \sigma_{up} = \sigma_s - tE_s / D \quad (16)$$

where σ_{GR} is the stress induces the tension in geosynthetic; t is the deflection of geosynthetic or differential settlement; E_s is the average elastic modulus of multiple soft soil layers; D is the thickness of soft soil layer.

By considering a deformed length of geosynthetic, Δl_d, the tensile force in the geosynthetic reinforcement is a function of strain, and is approximately equal to:

$$\varepsilon_a = \frac{\Delta l_d}{s'} \approx \frac{8}{3}\left(\frac{t}{s'}\right)^2 \quad (17)$$

Assuming the strain is uniform, the tension in the geosynthetic is a function of the amount of strain in the geosynthetic. The tension in the geosynthetic is determined as follows:

$$T = \sigma_{GR}\Omega s' = [\sigma_s - tE_s / D].\Omega s' \quad (18)$$

In which, s' = span design for tensioned membrane (s'=s–a), Ω = dimensionless factor from tensioned membrane theory. The dimensionless factor Ω is defined by Giroud et al. (1990)

$$\Omega = \frac{1}{4}\left[\frac{2t}{s'} + \frac{s'}{2t}\right] \quad (19)$$

Substitute Eq. (19) into Eq. (18), the resulting of the tension in the geosynthetic is given: $T = \frac{1}{4}[\sigma_s - tE_s / D].\left[\frac{2t}{s'} + \frac{s'}{2t}\right]s'$ (20)

On the other hand, under the influence of fill weight, the embankment between pile caps has a tendency to move downward, due to the presence of soft foundation soil. This movement is partially restrained by shear resistance, τ, which reduces the pressure acting on the geosynthetic but increase the load transferred onto the column caps (Han and Garb, 2002).

The shear resistance is a result of skin friction at the top and bottom of soil-geosynthetic interfaces. The expression of shear resistance is

$$\tau = \sum_{i=1} \sigma_n \tan\delta_i = \left(\sigma_s\lambda_p \tan\phi_p + (tE_s / D)\lambda_s \tan\phi_s + \lambda_c\{c_p + c_s\}\right) \quad (21)$$

where σ_n is the normal stress on the interface; δ_i is the interface friction angle, □p is the effective friction angle of the platform soil layer; □s is the effective friction angle of the subsoil; c_p is the adhesion value of platform soil layer; c_i is the adhesion value of soils; λ_p and λ_c are interaction coefficients between the reinforcement material and the proposed soils. Due to the effect of underground water, value λ_c is relatively small and varies from 0.1 to 0.2.

The total tensile force in the geosynthetic is calculated by the following expression:

$$\int_0^{s'/2} T dx = \frac{J_{GR}\Delta l_d}{2} + \int_0^{s'/2} \tau x dx \quad (22)$$

where J_{GR} is the tensile stiffness of the geosynthetic(kN/m); T is a maximum tensile force.

By integrating and imposing (17), (21) into (22), this leads to:

$$T = \frac{8t^2 J_{GR}}{3s'^2} + \frac{s'}{4}\left(\sigma_s\lambda_p \tan\phi_p + (tE_s / D)\lambda_s \tan\phi_s + \lambda_c\{c_p + c_s\}\right) \quad (23)$$

2.4. Design solutions of a piled embankment reinforced geosynthetic

Combining (20) and (234) implies a relevant third-order equation as follows:

$$\alpha_1 t^3 + \alpha_2 t^2 + \alpha_3 t + \alpha_4 = 0 \quad (24)$$

in which,

$\alpha_1 = 12E_s s'^2 + 64DJ_{GR}$;

$\alpha_2 = 6E_s s'^3 \lambda_s \tan\phi_s - 12\sigma_s Ds'^2$;

$\alpha_3 = 6\sigma_s Ds'^3 \lambda_p \tan\phi_p + 3E_s s'^4 + 6D(c_p + c_s)\lambda_c s'^3$;

$\alpha_4 = -3\sigma_s Ds'^4$

The solution of (24) gives the deflection of geosynthetic, t. Then, the remaining design parameters can be derived. The maximum tensile force of geosynthetic is determined by (20), and the maximum strain of geosynthetic is easily calculated by relationship $\varepsilon = T/J_{GR}$.

The other parameters can be derived by using (4), (5), (6).

3. DISCRETE ELEMENT MODELLING OF PROBLEMS

In this study, a three-dimensional software (SDEC) based on the discrete element method has been used for analysis (Villard et al., 2004). The algorithm of calculation used consists in successively alternating the application of Newton's second law.

Geometry: Because of the symmetric condition, only a quarter mesh was modeled to reduce time-consuming calculation in this study. An illustrative example of piled embankment reinforced geosynthetic is shown in Fig. 6. For a control case, pile spacing installed is 3m; the width of pile-cap equals 0.6m.

Soft ground: The soft subsoil is modeled like as springs by using a Winkler's Spring Model. The reaction of the subgrade soil is taken into account by using a subgrade reaction coefficient.

Geosynthetic: The geosynthetic sheet in this study is a non-woven geosynthetic, which modeled by 16 directions of fibers. These elements allow describing the tensile and membrane behavior of the sheet well. The micromechanical parameters consist of a normal & tangential, rigidity of contact, a tensile stiffness of the geosynthetic, an interactive friction angle geosynthetic/soil

Embankment material: The embankment is modeled by discrete element (8000 particles per m^3). The particles shape is given in Fig. 7. The vertical interfaces between pile-soil-geosynthetic were modeled. The mechanical properties of interfaces have the similarity to mechanical properties of embankment clusters.

Pile element: The pile element is modeled as the embedded pile with a very large stiffness, in which piles are considered as beam elements, is used to define the properties of pile group.

Interface behavior: Specific interaction laws are used to characterize the interface behavior between the soil particles and the sheet elements. The main contact parameters are the normal rigidity, the tangential rigidity, and the friction angle. In order to rather than the absence of relative roughness between the sheet elements and the soil particles, the microscopic friction angle of contact between exactly to the macroscopic friction angle given by the model.

Boundary condition: The boundary conditions include four frictionless vertical rigid walls to fix the horizontal displacement because of the symmetric condition.

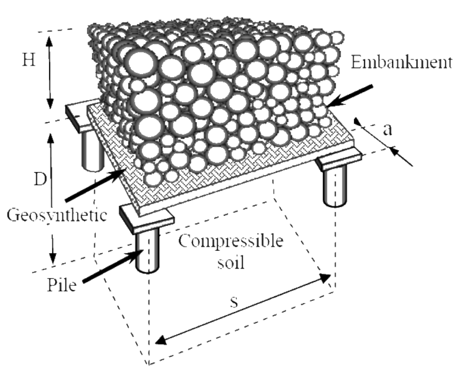

Fig. 6. Geometry of problem by SDEC

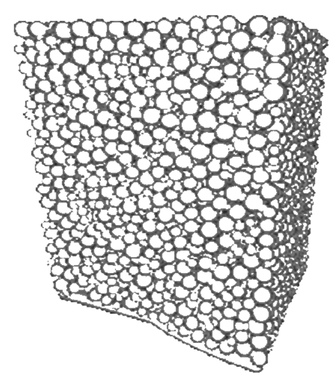

Fig. 7. Particle shape of the embankment

4. ANALYSIS OF RESULTS AND DISCUSSION

In order to investigate the validity of the presented method, the results are compared to the numerical analysis by DEM. One of the well-known design procedures currently is

EBGEO (2004) that also used for comparison in this section. The summary of the embankment geometry and design parameters used in this study case as follows:

Pile: cap width is 0.6m; pile spacing is 3m; Poisson ratio is 0.25; elastic modulus is $1.5x10^6$ kN/m^2. *Embankment fill:* Height is 0.75-3m; unit weight is $20kN/m^3$; peak friction angle is 40^0. *Platform fill (gravel):* Friction angle is 40^0. *Soft ground*: Thickness is 10m; elastic modulus is $1800kN/m^2$; friction angle is 10^0; cohesion is $12kN/m^2$; Poisson ratio is 0.33. *Geosynthetic:* tensile stiffness is 3000kN/m; λ_p equals to 1.0, λ_s equals to 0.62; λ_c equals to 0.1. No surcharge load. Piles are arranged in a square pattern.

4.1. Load transfer mechanis

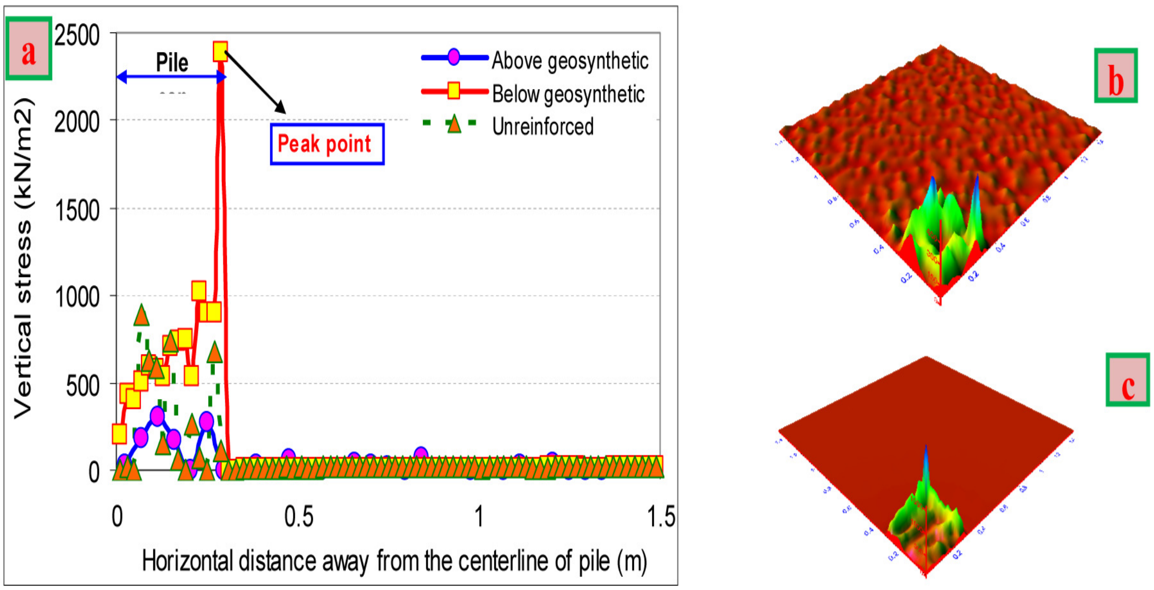

Fig. 8. Stress distribution from numerical modeling (a - 2D and b, c – surface of 3D)

Due to the inclusion of geosynthetic, the vertical stress distribution above and below the geosynthetic is different as shown in Fig. 8. The stress above the geosynthetic is mainly induced by soil arching while the stress below the geosynthetic is influenced not only by soil arching but also by the component of tension from the geosynthetic. As compared with the vertical stress magnitude for an unreinforced case, the vertical stress above the geosynthetic is less than that for the unreinforced case within the range of the pile cap. The benefit of reduced vertical stress above the geosynthetic and the pile is realized through minimizing the possibility of soil yielding or pile punching into the embankment (Fig. 8b). Meanwhile, the benefit of increased vertical stress below the geosynthetic is that membrane effect increased and more load transfer onto the piles (Fig. 8c)

4.2. Comparison of efficacy

A comparison of the design methods with numerical analysis for different height of embankment is shown in Fig. 9. According to the results, the proposed method produces an excellent agreement with the numerical results. Whereas, the EBGEO method significantly underpredict the efficacy of pile, which yields an over-conservative result.

The analysis also shows that the efficacy increases with an increase in the height of the embankment. With the increase in the height of the embankment, more shear resistance accumulates for enhancing the development of soil arching. It can be seen that the efficacy approaches a limiting value when the height of the embankment is increased.

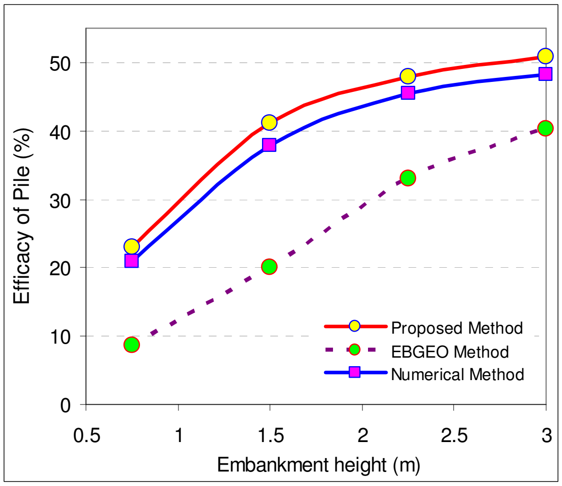

Fig. 9. Efficacy with fill height

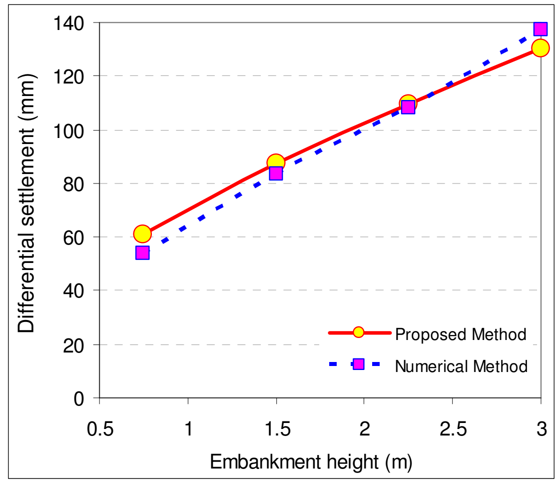

Fig. 10. Differential settlement with a fill height

4.3. Comparison of differential settlement

The differential settlement defined as the settlement difference between the center of the pile and the midpoint of the pile spacing. As presented in Fig. 10, the results show that the differential settlement increases with the height of the embankment fill. Such increase is estimated to be 155% when the embankment height is increased from 0.75m to 3m. It can be also seen that the results obtained from the proposed method agree well with the numerical results, only 2-6% in errors. This can be explained by the inclusion of membrane effect of geosynthetic into the proposed method. The EBGEO design procedures do not give the equation for prediction of differential settlement, hence it is not presented here.

4.4. Comparison of strain and tension in geosynthetic

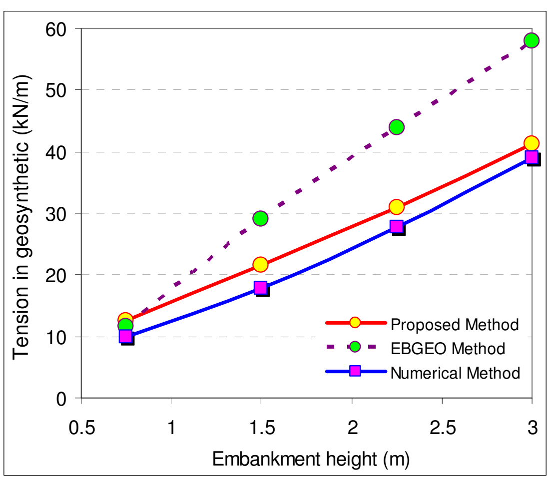

Fig. 11. Maximum tensile force with fill height

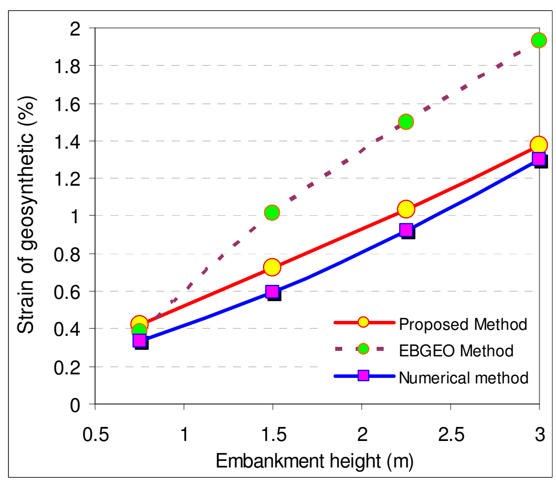

Fig. 12. Maximum strain with fill height

For the design purposes, the maximum strain and tension in geosynthetic are of more interest to geotechnical engineers, which is further investigated herein. As shown in Fig. 11, the maximum tension in the geosynthetic increases with the height of embankment fill. In addition, the proposed method gives a reasonable match to the numerical results while the EBGEO results are over-estimation highly compared to the numerical results. Similarly, the good match between the proposed method and numerical method is also seen in the term of a maximum strain of geosynthetic, which yields a suitable approach of the proposed method, as shown in Fig. 12. Therefore, the maximum value of geosynthetic strain obtained from the proposed method can be a good value to be used in design, which yields more economical and effective.

5. CONCLUSIONS

This study presents a proposed method for the design of geosynthetic reinforced pile supported embankment. The developed design method was established by combining tensioned membrane theory of geosynthetic materials with arching soil theory in the granular embankment, which allows considering the interaction behavior between pile-geosynthetic-soils, thereby providing a more suitable design method in practice.

The proposed method provides a simple equation to perform design analyses for a range of possible field situations. In addition, the three-dimensional approach with using a stress reduction ratio in the proposed method presented here. Therefore, the proposed method provides a useful tool to consider the role of geosynthetic, effect of soft soil and properties of fill materials, which have generally neglected in the currently available methods.

The results of comparison showed that the results obtained from the present method agree well with numerical results, and generally better than the one of EBGEO in this study.

The results indicate that the efficacy increases with an increase in the height of embankment, and approaches to a limiting value at a large value of embankment height. It is found that geosynthetic enhances the load transfer from the soil to the piles by tensioned membrane effect.

The study results also suggest that the vertical stress concentrated significantly at the edge of the pile cap. The vertical stress above the geosynthetic is less than that for the unreinforced case within the range of the pile cap.

REFERENCES

BS 8006, 2010. *Code of Practice for Strengthened/Reinforced Soils and Other Fills*. British Standard Institution, UK.

EBGEO, 2010. *Emfehlungen für den Entwurf und die Berechnung von Erdkorpern mit Bewehrungen aus Geokunststoffen – EBGEO,* 2. German Geotechnical Society, Auflage ISBN 978-3-433-02950-3.

Guido, V.A, Kneuppel, J.D., Sweeney, M.A., 1987. *Plate loading tests on geogrid reinforced earth slabs (New Orleans).* Proc. Geosynthetics 87, 216–225.

Giroud, J. P., Bonaparte, R., Beech, J. F., & Gross, B. A. (1990). *Design of soil layer-geosynthetic systems overlying voids.* Geotextiles and Geomembranes, 9(1), 11–50.

Han, J., Gabr, M.A., 2002. *Numerical analysis of geosynthetic-reinforced and pile-supported earth platforms over soft soil.* J. Geotech. Geoenviron. Eng. 128 (1), 44–53.

Han, J., & Alzamora, D. E. (Eds.). (2011, February). *Geo-Frontiers 2011: Advances in Geotechnical Engineering.* American Society of Civil Engineers.

Hewlett, W.J., Randolph, M.F., 1988. *Analysis of piled embankments*. Ground Eng. 21 (3), 12–18.

Jenck, O., Dias, D., & Kastner, R. (2005). *Soft ground improvement by vertical rigid piles two-dimensional physical modelling and comparision with current design methods*. Soils and Foundations, 45(6), 15-30.

Johnes, C. J. F. P., Lawson, C. R., & Ayres, D. J. (1990). *Geotextile reinforced piled embankment.* Geotextile, Geomembranes and Related Products, 155-160.

Low, B. K., Tang, S. K., & Choa, V. (1994). *Arching in piled embankments.* Journal of Geotechnical Engineering, 120(11), 1917-1938.

Miki, H. (1997). *Design of deep mixing method of stabilization with low improvement ratio.* In Proceedings of the first seminar on ground improvement in highways, Bangkok, Thailand, August (pp. 197-204).

Russell, D., & Pierpoint, N. (1997). *An assessment of design methods for piled embankments.* Ground Engineering, 30(10).

Sloan, J. (2011). *Column-supported embankments: full-scale tests and design recommendations.*

Terzaghi, K., 1943. *Theoretical Soil Mechanics.* Wiley, New York.

Villard, P., & Chareyre, B. (2004). *Design methods for geosynthetic anchor trenches on the basis of true scale experiments and discrete element modelling.* Canadian Geotechnical Journal, 41(6), 1193-1205.

Van Eekelen, S. J. M., Bezuijen, A., & Van Tol, A. F. (2013). *An analytical model for arching in piled embankments.* Geotextiles and Geomembranes, 39, 78-102.

Abstract:

MỘT PHƯƠNG PHÁP MỚI CHO VIỆC THIẾT KẾ NỀN ĐẮP ĐƯỢC XÂY DỰNG TRÊN HỆ CỌC KẾT HỢP VẬT LIỆU GIA CƯỜNG

Kỹ thuật gia cường đất với cọc đã được chứng minh là một giải pháp thú vị bởi khả năng đảm bảo sự ổn định của nền đắp. Một vài phương pháp lý thuyết dựa trên các mô hình đơn giản hóa đã được đề xuất nhưng kết quả từ các phương pháp này thường cao hơn đáng kể và quá an toàn, đưa đến thiết kế không đạt hiệu quả kinh tế tối ưu. Nghiên cứu này trình bày một phương pháp phân tích mới cho việc thiết kế nền đắp trên hệ cọc kết hợp vật liệu gia cường, dựa trên sự kết hợp của hiệu ứng vòm, hiệu ứng màng căng và cơ chế tương tác sức kháng cắt. Phương pháp trình bày có khả năng để mô tả ứng xử và những tương tác phức tạp giữa vật liệu gia cường-đất-cọc, và do đó cung cấp một phương pháp thiết kế nhiều thích hợp cho các kỹ sư. Thêm vào đó, mô hình số dựa trên phương pháp phần tử rời rạc (DEM) với các ngôn ngữ mã mới nhất cũng đã được sử dụng để nghiên cứu độ tin cậy của phương pháp đề xuất. Các kết quả nghiên cứu hướng đến việc cung cấp một số chỉ dẫn cho các nhà thiết kế, và làm sáng tỏ một vài cơ chế tương tác trong nền đắp có gia cường vật liệu địa kỹ thuật.

Từ khóa: Nền đắp, cọc, vật liệu gia cường, phương pháp đề xuất, thiết kế, phân tích số,

Ngày nhận bài: 10/5/2018

Ngày chấp nhận đăng: 07/8/2018