A REVIEW OF AVAILABLE DESIGN TECHNIQUES AND NUMERICAL ANALYSIS OF PILED EMBANKMENT WITH GEOSYNTHETIC

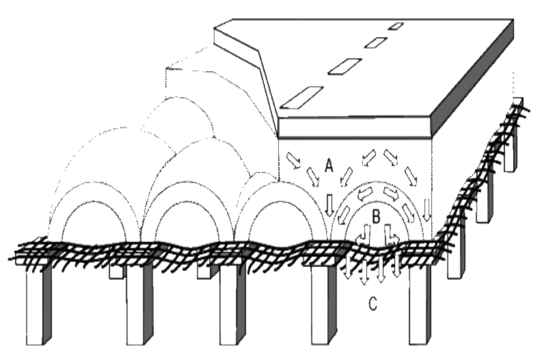
Tuan A. Pham[1,2] , Pascal Villard[1], Daniel Dias[1]

Abstract: *Piled embankment reinforced geosynthetics are used as integrated foundation systems for construction of embankment over soft ground. Several design guidelines are available in the literature for these embankments based on the soil arching and tensioned membrane theories. However, among design engineers, there is uncertainty regarding the applicability of these design methods. This paper investigates some practical aspects and identifies some inconsistencies in applying these design methods. Discrete element method with the most advanced code description currently used for analysis of problems and compared to the available design techniques from the case study. This comparison allows giving recommendations about selecting the most suitable design method corresponding to detailed items. According to results, methods of Van Eekelen and EBGEO are the design methods recommended highly for prediction of stress reduction ratio, while methods proposed by Abusharar et al. and EBGEO are more suitable for the design of geosynthetic reinforcement.*

Keywords: Piled embankment, geosynthetics, available design methods, discrete element method, deformation, critical height.

1. INTRODUCTION

Embankments constructed over soft soils induce a significant load over a large area. The technique of reinforcing soil with columns has proven to be an interesting solution that prevents failure or excessive deformations of embankments. A piled embankment reinforced geosynthetic is a complex system consisting of piles, generally arranged in a square or rectangular pattern and driven into the soft ground to a firm-bearing stratum, Figure 1. Geosynthetic reinforcement is installed over the pile caps at or close to the base of the embankment. Due to the significant difference in stiffness between the piles and soft soils, the stresses within the soil between piles are redistributed as the soil tries to establish equilibrium by transferring loads into stiffer elements and decrease loads on soft ground. As a result, different structural arrangements of the particles are created. Sometimes this arrangement and stress redistribution are such that the resistance provided by the soil is analogous to a structural arch. This is called soil arching.



[1] Lab 3SR, University of Grenoble Alpes, Grenoble, France
[2] University of Science and Technology, The University of Danang, Vietnam

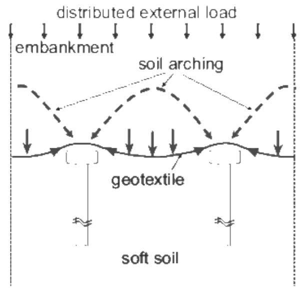


Figure 1. Load transfer mechanism in reinforced piled embankments (Van Eekelen et al.,2013)

A number of research studies have been carried out using experimental and numerical modelling to investigate the behaviour of piled embankment reinforced geosynthetic (PERG) (e.g. Low et al., 1994; Giroud, 1995; Abusharar et al., 2009; P. Villard, 2009; Van Eekelen et al., 2014; Joe A. Sloan, 2012). It has been found that the loads generated in the geosynthetic reinforcement in piled embankments are due to two mechanisms. Firstly, the reinforcement acts to transfer the vertical embankment load not supported by the embankment arch to the pile caps. Secondly, the geosynthetic reinforcement counteracts the horizontal outward thrust of the embankment fill. The load due to arching occurs both along the length and across the width of the embankment. The load due to horizontal outward thrust across the width of the embankment only.

While several methods currently exist for estimating the magnitude of arching (Terzaghi, 1943; Guido et al., 1987; BS8006, 2010; Collin, 2007; Hewett and Randolph, 1998; PWRC, 1997; Kempfert et al., 2004; Abusharar et al., 2009; Low et al., 1994; Van Eekelen et al., 2014) none yet captures the essential characteristics of these complex structures. Also, most of them have not considered the support of the soft ground in the load transfer mechanism. The shape of the arch and its evolution are not consistent with these guidelines.

This paper aims to investigate a valued design method for the analysis and design of the piled embankment reinforced geosynthetic. A review of existing design techniques (new and recently revised design methods), that will help engineers and designers access more comfortable in practical works. In addition, the discrete element method, an effective approach was used in numerical modelling program to support the comparison, which was not previously modeled. Moreover, the inconsistencies in results of the current hand's methods are identified and discussed in detail. While the debation and disagree continually between researchers on the selection of the best method of the available existing design techniques for design, there detailed discussions provide a great insight to clarify and answer three questions: What popular design methods are existing? What are the advantages and disadvantages of each method? Moreover, what methods should be chosen for the design?

2. NUMERICAL MODELLING BY DISCRETE ELEMENT METHOD (DEM)

2.1. Discrete element method

Discrete element methods comprise a set of computational modeling techniques suitable for the simulation of the dynamic behavior of a collection of multiple rigid or deformable, particles or domains of arbitrary shape, subject to continuously varying constraints. Bodies collide with one another, new contacts are established, while old contacts may be released, giving rise to changes in the contact status and contact interaction forces, which in turn influences the subsequent movements of bodies.

The discrete element method used is a three-dimensional software (SDEC) based on the dynamic molecular which apply the Newtonian approach for each particular particle, through using rigid bodies (Donze and Magnier, 1995, 1997). The basic element employed are spherical particles of various sizes which can interact together. The algorithm of calculation used consists in successively alternating the application of Newton's second law.

2.2. Discrete element modeling of the problem

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. 2. For a control case, pile spacing is installed 3m, the width of pile cap equals 0.6m, the embankment height is 3m.

2.3. Modeling of the soft ground

The compressible subsoil under the geosynthetic sheet is assumed to be very weak. And the action of underlying soil was modeled by using a Winkler's Spring Model (1867)(springs of rigidity k are positioned under the sheet). A compressive modulus of the soft soil is taken into account to simulate the reaction of the subgrade soil. For an element of the spring of a section S, the coefficient K is defined by $K=E_{oed}S/D$, with E_{oed} is the geometric modulus of the soft soil and D is the thickness of the compressible soil.

2.4. Modeling of the geosynthetics

The geosynthetic sheet is a non-woven geotextile (modeled by 16 directions of fibers) with an overall stiffness J = 3000kN/m reinforced in two perpendicular directions. The friction angle of the interface soil/geosynthetic is 26^0. The sheet is modeled by 1800 three node finite elements of a thickness e = 5mm.

2.5. Modeling of the embankment material

The embankment is modeled by discrete element (8000 particles per m^3). The particles shape is given in Fig. 2. The vertical interfaces between pile-soil-geosynthetics were modeled to take into account the friction between pile and embankment materials. The mechanical properties of interfaces have the similarity to mechanical properties of embankment clusters.

2.6. Modeling of the structure element

According to J. Han et al. (2002) showed that as the Young modulus (E_p) of the pile is higher than 1000Mpa corresponding to 1356Mpa/m, the stiffness of the pile will not have an effect on the settlement and load transfer. To eliminate the effect of pile stiffness, a value 2000Mpa/m was chosen for all cases.

2.7. Interface behavior and boundary condition

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.

The boundary conditions include four frictionless vertical rigid walls to fix the horizontal displacement because of the symmetric condition. A simulation image is shown in Figure 2.

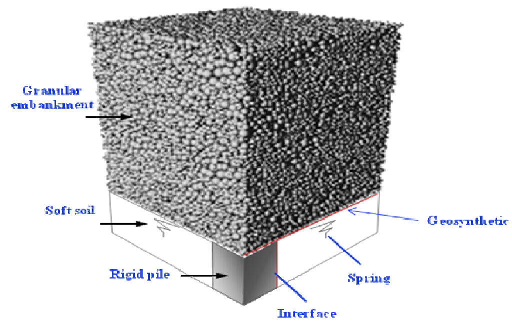


Figure 2. Numerical modeling of problem by discrete element method

All parameters of materials used in the analysis of a control case are listed in Table 1. where φ_p is the peak friction angle, n is the porosity, γ is the unit weight, r_g is the radius of grains, K_s is the subgrade reaction, K_p is the stiffness of pile, J is the tensile stiffness, e is the thickness, ν is the Poisson ratio.

Table 1. Material parameters for a control case

Embankment materials:	$\varphi_p = 40^0$, n = 0.4, γ =18kN/m^3, r_g =0.04m
Soft soil	K_s = 0.2Mpa,
Pile	E_p = 1500Mpa, ν =0.25
Geosynthetics	J = EA =3000kN/m, e = 5mm, ν =0.35

3. REVIEW OF CURRENTLY AVAILABLE DESIGN METHODS

There are various methods available for the design of GRPS embankments. Not all these methods were initially developed for designing embankments, but they were later adopted for this process. This section presents a description of currently available design methods.

3.1. Estimation of stress reduction ratio

3.1.1. Adapted Guido Method

The last expression for the stress reduction ratio included in Russell and Pierpoint (1977) is commonly referred as the adapted Guido Method.

$$S_{3D} = \sqrt{2}(s-a)/3H \quad (1)$$

In that, s - centerline pile spacing, a - width of pile cap, H - embankment height

3.1.2. Adapted Terzaghi Method

The arching theory developed by Terzaghi (1943) based on his classic trap door, is used by many authors to describe the load transfer mechanism in a pile-supported an embankment.

$$S_{3D} = \frac{\gamma(s^2-a^2)}{(\gamma H+q)4aK\tan\varphi}\left(1-e^{\frac{-4aHK\tan\varphi}{(s^2-a^2)}}\right)+\frac{q}{\gamma H+q}e^{\frac{-4aHK\tan\varphi}{(s^2-a^2)}} \quad (2)$$

where γ - unit weight of embankment fills, K - coefficient of earth pressure, φ – effective friction angle, q – surcharge or traffic load

3.1.3. British Standard BS 8006 (2010)

In this design code, two different arching conditions are defined: (i) the partial arching condition, where 0.7(s-a)≤H≤1.4(s-a) and (ii) the full arching reduction, where H >1.4(s-a). Equations for the stress reduction ratio can be derived for both conditions using the method adopted by Russell and Pierpoint (1997).

For partial arching:

$$S_{3D} = 2s[s^2 - a^2(P_c/\gamma H)]/[(s+a)(s^2-a^2)] \quad (3)$$

For full arching:

$$S_{3D} = 2.8s[s^2 - a^2(P_c/\gamma H)]/[(s+a)^2 H] \quad (4)$$

where P_c – vertical stress on pile cap, S_{3D} - stress reduction ratio

3.1.4. Hewlett and Randolph method (1998)

Hewlett and Randolph (1988) carried out model tests on a granular embankment fill material overlying a rectangular grid of pile caps to investigate the amount of load transferred to the piles and the foundation soil due to soil arching. The calculations based on the semi-spherical arches formed of the fill material.

$$S_{3D} = (1-a/s)^{2(K_p-1)}\left(1-\sqrt{2}s(K_p-1)/[H(2K_p-3]\right)+[2(s-a)(K_p-1)]/[\sqrt{2}H(2K_p-3)] \quad (5)$$

where K - coefficient of passive earth pressure, S_{3D} - stress reduction ratio

3.1.5. Japanese PWRC method (1997)

This method was proposed by Miki (1997) for embankments on deep mixing method columns. The total embankment volume is divided into the volume of the embankment that acts on the improved ground and the unimproved ground or geosynthetic. The expression of the vertical stress, p, on the unimproved ground is:

$$p = \gamma\frac{\frac{\pi}{96}(s-d_c)^2\tan\theta(5d_c+4s)+(4-\pi)\left(\frac{s}{2}\right)^2\left[\frac{s-d_c}{2}\tan\theta+\frac{s}{6}\right](\sqrt{2}-1)\tan\theta}{s^2-\frac{\pi d_c^2}{4}} \quad (6)$$

where d_c – diameter of the column, θ – arching angle ($\theta=45^0-\varphi/2$)

3.1.6. Kempfert et al. (EBGEO) method

The Kempfert et al. (2004) method is based on lower bound plasticity theory, pilot-scale tests, and numerical analyses. Like the Hewlett and Randolph (1998) method, this method considers a hemispherical domed arch between columns or piles caps. The stress reduction ratio for this method is shown as follows:

$$S_{3D} = \frac{1}{\gamma H}\left\{\lambda_1^X\left(\gamma+\frac{q}{H}\right)\left[H(\lambda_1+h_g^2\lambda_2)^{-X}+h_g\left(\left(\lambda_1+\frac{h_g^2\lambda_2}{4}\right)^{-X}-(\lambda_1+h_g^2\lambda_2)^{-X}\right)\right]\right\} \quad (7)$$

$$\lambda_1 = (s_d - d)^2 / 8; \qquad \lambda_2 = (s_d^2 + 2ds_d - d^2)/2s_d^2; \qquad X = d(K_p - 1)/\lambda_2 s_d$$

$$h_g = s_d / 2 \text{ for } H \geq s_d/2; \qquad h_g = H \text{ for } H \leq s_d/2$$

where s_d – diagonal pile spacing, d – pile diameter, K_p – passive lateral earth pressure, h_g – arching height, q – surcharge, H –embankment height, γ – unit weight of embankment fill

3.1.7 Low et al. method (1994)

Low et al. (1994) developed some equations and charts to evaluate the tension and mobilized strain in the geosynthetic reinforcement layer and the stress reduction over the foundation soil. The vertical stress acting on the foundation soil midway between piles, σ_s, is

$$\sigma_s = [0.5\gamma(s-a)(K_p - a)/(K_p - 2)] + [s-a)/s]^{K_p - 1}[\gamma H - 0.5\gamma s(1 + (K_p - 2)^{-1})] \quad (8)$$

The estimation of stress reduction ratio can be expressed by the following equation:

$$S_{3D} = (\sigma_s - tE_s / D)/\gamma H \quad (9)$$

where D – soft soil thickness, E_s – elastic modulus of soft soil, t – deflection of geosynthetic

3.1.8. Abusharar et al. method (2009)

Based on the approach of Low et al. (1994), theoretical analysis for pile embankment was developed by Abusharar et al., (2009). The main modification was taking into account the skin friction mechanism at the soil-geosynthetic interface. The stress reduction ratio can be calculated by Eq. (9). The following cubic equation with β = 4t/(s-a) can be obtained:

$$a\beta^3 + b\beta^2 + c\beta + d = 0 \quad (10)$$

a = 32DJ +4(s-a)^2E_s ; b = 2(s-a)^2$\lambda_3$$E_s$tanφ - 4(s-a)D$\sigma_s$;

c = 2(s-a) λ_3D$\sigma_s$tanφ + (s-a)^2E_s; d = -(s-a)Dσ_s

where σ_s – vertical stress acting on soft soil, J – tensile stiffness of geosynthetic, λ_3-interaction factor, φ – effective friction angle of the surrounding soils.

3.1.9. Van Eekelen et al. method (2014)

A new calculation model is derived and summarised by Van Eekelen et al. (2013, 2014). This model is a concentric arch model with the assumption that the load is transferred along the concentric 3D hemispheres towards the GR strips and then via the concentric 2D arches towards the pile caps. This method is applied to calculate soil arching as follows:

$$A = F_{pile} = (\gamma H + p).s_x.s_y - F_{GRsquare} - F_{GRstrip} \quad (11)$$

The total load resting on GR + subsoil is, therefore:

$$B + C = F_{GRsquare} + F_{GRstrip} \quad (12)$$

where, $F_{GRsquare}$ – total vertical load applied exerted by 3D hemispheres, $F_{GRstrip}$ – total vertical load on GR trips, s_x, s_y – center-to-center spacing in both directions.

3.2. Estimation of tension in geosynthetic

The tension in the geosynthetic, T, is calculated according to,

$$T = \frac{p.(s^2 - a^2)}{4a}\sqrt{1 + \frac{1}{6\varepsilon}} \quad (13)$$

where, p – pressure distributed on geosynthetic, ε – a strain of geosynthetic

This equation was used to calculate the reinforcement tension for the Hewlett and Randolph, Guido, Terzaghi, Van Eekelen and BS8006 methods. A design strain of 5% was used for the calculation, as recommended by BS8006 (2010).

McGuire and Filz (2008) present a solution which imposes stress-strain compatibility by substituting ε=T/J into Equation (13), resulting in the square column as follow:

$$96T^3 - 6K_g^2 T - K_g^2 J = 0$$

where $K_g = p(s^2 - a^2)/a \quad (14)$

According to Nordic guideline (2005), the tension in geosynthetic due to vertical load in three dimensional can be determined by

$$T_{rp3D} = \frac{1 + s/a}{2} \cdot \frac{(s-a)^2}{4.\tan 15^0} \cdot \gamma \cdot \sqrt{1 + \frac{1}{6\varepsilon}} \quad (15)$$

where s = pile center to center spacing (m), a = width of pile cap (m), γ = unit weight of embankment material (m); ε = maximum allowable strain in the reinforcement

Abusharar et al., (2009) provided a formular for prediction of tensile force in geosynthetic:

$$T = \left(\frac{1 + 4\beta^2}{8\beta} \right)(s - a)\left(\sigma_s - \frac{tE_s}{D} \right) \qquad (16)$$

where t - deflection of geosynthetic, σ_s – stress on geosynthetic and soft soils, β = 4t/(s-a)

3.3. Estimation of differential settlement

The maximum mid-pan deflection of the geosynthetic can be determined by

$$t = (s - a)\sqrt{\frac{3}{8}\varepsilon} \qquad (17)$$

Eq. (17) is presented in BS8006 (2010) and Nordic Guideline (2005) in order to calculate the deflection of the geosynthetic after obtaining strain value of reinforcement, ε.

4. ANALYSIS OF RESULTS

4.1. Comparison of results using stress reduction ratio

The variation in stress reduction ratio (S_{3D} or SRR) with embankment height is shown in Fig. 3. To avoid time-consuming, the embankment height is selected for comparison in this study because that it is one of the most critical factors which influence soil arching and tensioned membrane effect. Out of the nine design methods, the one proposed by Guido et al. considerably under-estimate the stress reduction ratio. Terzaghi's method, BS8006 modified, Hewlett & Randolph, Low et al. method, and method adapted by PWRC give overly conservative results for the stress reduction ratio, yielding uneconomical designs. The Abusharar et al. method highly underpredicts the S_{3D}. The variation in S_{3D}, obtained from this method shows an inverse variation compared to the other design methods and numerical results. This is because the tE_s/D term in calculation equation becomes larger when t is increased with embankment height.

The design methods proposed by Kempfert et al. that adopted into EBGEO guideline and Van Eekelen method produces a better match for numerical results. However, inconsistent results over the range of embankment height selected. It has been found that Van Eekelen et al., method give the most excellent agreement with numerical results compared to other remaining methods. The average difference between these methods with numerical analysis can be accepted, approximately 22.6% for EBGEO and only 1.97% for Van Eekelen method.

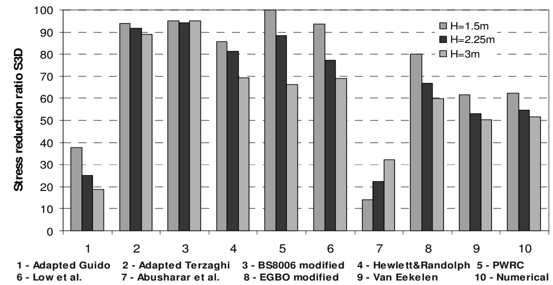


Figure 3. Stress reduction ratio with embankment height

It is better to recall that Van Eekelen method is one of the newest method currently, which based on a concentric arch model with the assumption that the load is transferred along the concentric 3D hemispheres towards the GR strips and then via the concentric 2D arches towards the pile caps. Therefore, this approach produces more realistic results in practice.

The Van Eekelen et al. method is therefore strongly recommended for estimation of stress reduction ratio in the design process. Kempfert et al. method that adopted into EBGEO can also be considered as the second selection to predict the stress reduction ratio.

4.2. Comparison of results using the differential settlement

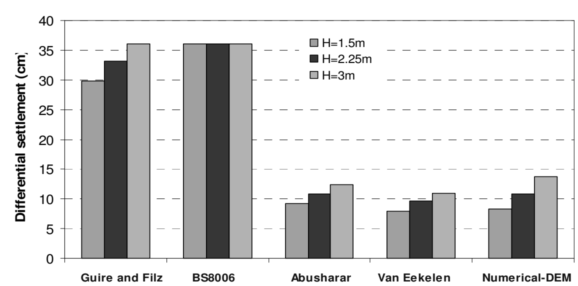


Figure 4. Differential settlement with embankment height

A comparison of the design methods for different embankment height using differential settlement is shown in Fig. 4 with the pile spacing equals 3m. The differential settlement is defined as the maximum difference in settlement between pile and soft ground. According to the results, the Guire & Filz method significantly over-predict the differential settlement. The similar trend can also be seen in the results of BS8006. The data show that the BS8006 and Guire & Filz methods are over conservative and uneconomical. It should also be noted that the method in BS8006 does not have the ability to assess the influence of embankment height.

In the meanwhile, a method of Van Eekelen et al. gave the results slightly under-predict compared to numerical results, up from 5% to 20%. The Abusharar et al. method provides good agreement with the numerical results for cases 1.5m and 2.25m. However, for the Abusharar et al. method, the estimation of differential settlement is smaller than the numerical results for the case 3m and this difference might increase when embankment height is increased. This can induce instability or uncertainty for embankment in reality.

4.3. Comparison of results using tension in geosynthetic

The geosynthetic tension results, obtained using the selected design techniques, are compared with the results from present method and three-dimension numerical model, with the results plotted in Figure 5. According to the results, the Guire & Filz method and Nordic guideline significantly over-predict for all three cases, it may be even higher when using BS8006 due to a safety used and adapted into BS8006, which yielding uneconomical design. The EBGEO gives an overestimation of the geosynthetic tension as compared to numerical analysis (about 48÷63%). At the meanwhile, Van Eekelen et al. method produces a significant under-prediction than the numerical results (about 38.6÷51.4%). The Abusharar et al. method slightly over-estimate (about 18.4÷38.7%) compared to the numerical method, but it still agrees better or equally well with the numerical results.

A similar pattern can be observed in Figure 6 which shows the variation in geosynthetic strain with different embankment heights for the selected design techniques. The Abusharar are in better agreement with the numerical results compared to the other methods. The Van Eekelen et al. method is under-prediction significantly, meanwhile, Guire &Filz and EBGEO is still overestimation of geosynthetic strain compared to numerical results.

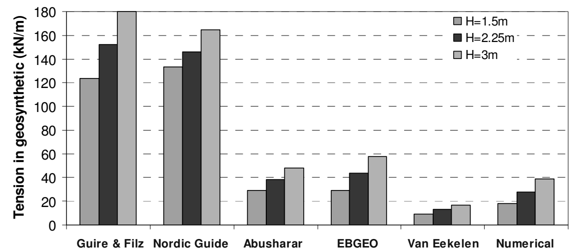


Figure 5. Maximum tension in geosynthetics with embankment height

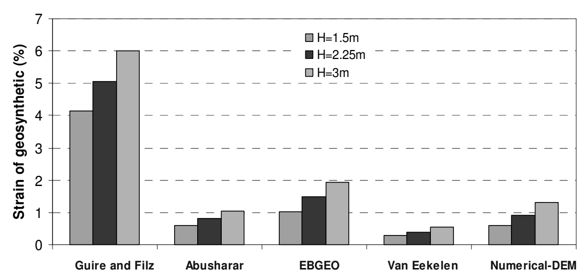


Figure 6. Maxium strain of geosynthetics with embankment height

5. CONCLUSIONS

The design techniques used for comparison in this paper are the most popular methods used in practice. According to the results, these methods differ significantly when predicting the stress reduction ratio, differential settlement, strain and tension in geosynthetic.

The methods proposed by Terzaghi, BS8006, Hewlett & Randolph, PWRC consistently overestimates the stress reduction ratio, the methods proposed by Guido, Abusharar, meanwhile, consistently underpredict the results. The results obtained from Guido et al.'s method cannot be relied upon because they only consider the pile spacing diameter and the embankment height and no other material parameters.

Van Eekelen et al. method is highly

recommended for selecting to compute stress reduction ratio. The method presented in EBGEO guideline might also be considered as the second choice in the estimation of S_{3D}. However, Van Eekelen et al. method is still the best agreement with numerical methods and is therefore applicable for use in practice.

The Van Eekelen et al. method could be in better agreement with the numerical results compared to the other methods in prediction of stress reduction ratio. However, this method provides significant underestimation for terms including differential settlement, strain, and tension in geosynthetic. It, therefore, is unrealistic as well as unsafe in the design of geosynthetic reinforcement.

The Abusharar et al. method gives a better match with a numerical method for prediction of differential settlement and strain of geosynthetic while there is significantly overestimation for tension in geosynthetic. However, the small strain and deflection of geosynthetic given by this method cannot be accepted because of the calculated strain based on the highly underpredicted stress reduction ratio. The EBGEO can also be considered the second choice for prediction of strain and tension in the geosynthetic.

The critical height of the embankments was inconsistently suggested overtimes by many different authors. The numerical results in this paper show that soil arching can develop maximum at the ratio 1.25(s-a) and might decrease after that.

Notation

a = width of pile cap
d_c = diameter of column cap
D = thickness of soft soil
E_{oed} = odometer modulus of soft soil
E_p = stiffness of pile
E_s = elastic modulus of soft soil
h_g = arching height
H = embankment height
J = tensile stiffness of geosynthetics
K_p = passive earth pressure coefficient
K_s = subgrade reaction coefficient
n = porosity of embankment fills
p = pressure distributed on geosynthetic
P_c = vertical stress on pile cap
q = surcharge or traffic load
r_g = radius of grains
s = center-to center pile spacing
s_d = diagonal pile spacing
S_{3D} = stress reduction ratio
t = deflection of geosynthetics
T = maximum tension in geosynthetics
φ = friction angle of embankment
γ = unit weight of embankment,
ν = poisson ratio
θ = arching angle
σ_s = vertical stress acting on soft soil
λ_3 = interaction factor
ε = maximum allowable strain

REFERENCES

Abusharar, S.W., Zheng, J.J., Chen, B.G., Yin, J.H., 2009. *A simplified method for analysis of a piled embankment reinforced with geosynthetics*. Geotext. Geomembr. 27 (1), 39–52.

Ariyarathne, P., Liyanapathirana, D.S., Leo, C.J., 2013. *A comparison of different two-dimensional idealizations for a geosynthetic reinforced pile- supported an embankment*. Int. J. Geomech. 13 (6), 754–768.

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

Collin, J.G. 2004. *Column-supported embankment design considerations*. In: Proceedings of the 52nd Annual Geotechnical Engineering Conference. University of Minnesota, Minneapolis, Minnesota, pp. 51–78.

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.

Filz, G.M., Smith, M.E., 2006. *Design of Bridging Layers in Geosynthetic- Reinforced Column-Supported Embankments.* Virginia Transportation Research Council, Charlottesville, Virginia, 46.

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.

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

Le Hello, B., & Villard, P. (2009). *Embankments reinforced by piles and geosynthetics-Numerical and experimental studies dealing with the transfer of load on the soil embankment*. Engineering Geology, 106(1–2), 78–91.

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

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

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.

Villard, P., Chevalier, B., Le Hello, B., & Combe, G. (2009). *Coupling between finite and discrete element methods for the modeling of earth structures reinforced by geosynthetic*. Computers and Geotechnics, 36(5), 709–717.

Abstract:

PHÂN TÍCH NỀN ĐẮP ĐƯỢC GIA CỐ HỆ CỌC VÀ LƯỚI ĐỊA KĨ THUẬT: TỔNG QUAN, PHÂN TÍCH SỐ VÀ TỐI ƯU THIẾT KẾ

Hệ cọc kết hợp gia cường lưới địa kỹ thuật là thường được sử dụng như một hệ móng tích hợp để gia cố cho nền đắp đi qua các khu vực đất yếu. Một vài phương pháp thiết kế cho kỹ thuật gia cố này đã được đề xuất bởi một vài tác giả dựa trên nguyên lý của hiệu ứng vòm và lý thuyết màng căng xảy ra trong nền đắp. Tuy nhiên, kết quả tính toán từ các phương pháp thiết kế cho đến giờ vẫn tồn tại những sự khác biệt đáng kể, bao gồm cả việc so sánh với kết quả phân tích số và thí nghiệm. Mục đích chính của bài báo này là để nghiên cứu các khía cạnh thực tế và xác định sự khác biệt giữa các phương pháp thiết kế tồn tại hiện thời. Mô hình số dựa trên phương pháp phần tử rời rạc (DEM) cũng được tiến hành trong bài báo này để hỗ trợ cho việc phân tích và so sánh. Kết quả so sánh giữa các phương pháp lý thuyết và phân tích số đã thể hiện rằng các kết quả từ phương pháp của Van Eekelen và EBGEO là nhiều hợp lý và phù hợp với kết quả phân tích số so với các phương pháp khác. Kết quả nghiên cứu cũng chỉ ra rằng hiệu ứng vòm chỉ xảy ra trong phạm vi chiều cao giới hạn, xấp xỉ bang 1.25 lần khoảng cách giữa hai cọc liên tiếp.

Từ khóa: Nền đắp, hệ cọc gia cường lưới địa kỹ thuật, phương pháp thiết kế, phân tích số, hiệu ứng vòm, chiều cao tới hạn

Ngày nhận bài: 15/3/2018

Ngày chấp nhận đăng: 28/3/2018