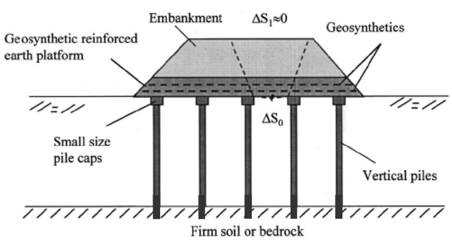
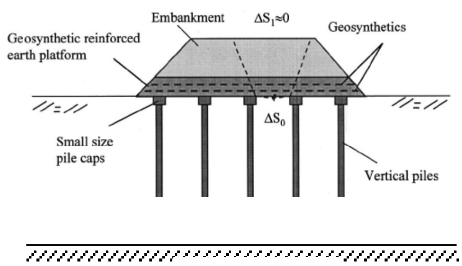
Universität Stuttgart

PILED EMBANKMENTS:

LITERATURE REVIEW AND REQUIRED FURTHER RESEARCH USING NUMERICAL ANALYSIS





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Foreword

This report contains literature review on the current analytical and numerical methods of the design of piled embankments. In this report, the current design methods are analysed and discussed. At the end of the report, required further research, which is focused on the numerical analysis of piled embankments, are proposed. This report is done in the framework of research on Advanced Modelling of Ground Improvement on Soft Soil (AMGISS) conducted at the Institute of Geotechnical Engineering, University of Stuttgart.

SUMMARY

The increasing need for infrastructure development has often forced engineers to deal with building on soft soils. The soft soil cannot take external load without having large deformations. Thus, soil improvement is needed. One of the soil improvement techniques is a “piled embankment”. Several analytical methods exist for the design of piled embankments. However, there are uncertainties with the methods as the piled embankment consists of complex soil-structure interaction. Nowadays, numerical methods such as finite element analysis are available for analysing complex soil-structure interaction problems. Nevertheless, clear and uniform procedures or guidelines on piled embankment design with finite element method are not available.

This research is aimed to establish reliable calculation procedures of piled embankment design using finite element analysis. For this reason, literature about piled embankments is analysed and a comprehensive research proposal on numerical analyses of piled embankments is established in order to improve the technique in the future design guidelines.

From the analysed literature on piled embankments, there are several uncertainties in the design of piled embankments as well as in the procedure of numerical analysis of piled embankments. In the framework of this topic, several research topics have been established to reduce the uncertainties. The research topics include the investigation of the influence of soil constitutive models, the number of geosynthetic layers, the effects of the pile installation process, the pile penetration depth, the effects of consolidation and creep and the possible local failure on the soil arching development and differential settlement of a piled embankment. In addition to that, research on the correlation between 2-dimensional and 3-dimensional numerical analyses and the determination of geosynthetic tension are also considered.

The research items are required to obtain a better understanding of the design of piled embankments especially in the case of floating piles using finite element analysis. Furthermore, a proper procedure for piled embankment design using finite element method with reliable parameter assumptions need to be found.

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1. Introduction

The increasing need for infrastructure development has often forced engineers to deal with building on soft soils. The soft soil cannot take external load without having large deformations. One soil improvement technique is a "piled embankment". Several analytical methods exist for the design of piled embankments. However, there are uncertainties with the methods as the piled embankment consists of complex soil-structure interaction. Nowadays, numerical methods such as finite element analysis are available for analysing complex soil-structure interaction problems. Nevertheless, clear and uniform procedures or guidelines on piled embankment design with finite element method are not available.

This proposal is aimed to improve the design method of piled embankments using finite element analysis. For this reason, literature about piled embankments is analysed and a comprehensive research proposal on numerical analyses of piled embankments is established in order to improve the technique in the future design guidelines.

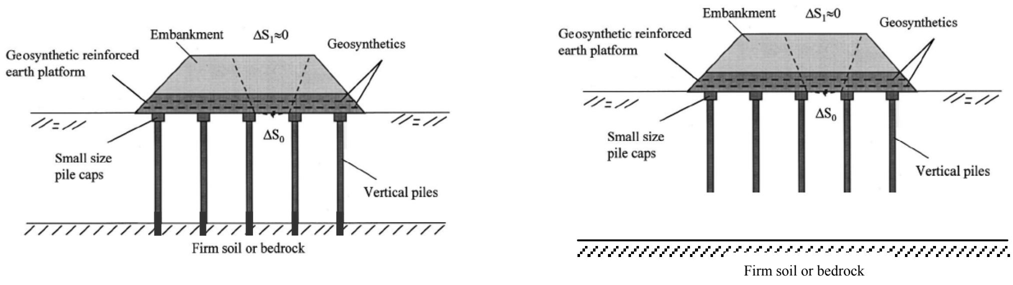
1.1 Problem Definition

Several analytical design methods have been proposed and some of them have been included in the guidelines. Nevertheless, the methods have uncertainties. In addition to that, a uniform design procedure using advanced numerical method such as finite element analysis is not available.

1.2 Aims

This report is aimed to analyze literature on the design of piled embankments and to propose research topics on possible improvements of piled embankment design using numerical methods.

2. Description of Piled Embankments



(a) (b)

Figure 2.1: Piles-supported embankment (a) on end bearing piles (b) on floating piles

A piled embankment is an embankment, which is supported by piles embedded in the soft soil. Geosynthetic layers are often included for the embankment reinforcement. Figure 2.1 shows typical structures for piled embankments. A piled embankment is constructed by installing piles with a certain grid formation in a soft soil up to a certain depth, which is generally reaching a competent stratum such as firm soil or bedrock. If a geosynthetic reinforcement is included, the geosynthetic material is laid on top of a thin layer of embankment material. It is not usually laid directly on top of pile caps. After constructing the geosynthetic layers, the embankment fill is raised up to the required height. Finally, the construction such as railway or road pavement is built on top of the embankment.

2.1 Soil

When an embankment needs to be built on soft soil, large soil deformation will take place if no measure is taken. A piled embankment can be applied as ground improvement when the ground condition at the construction site is soft clay. The soft clay is typically very compressible and has a low strength. For example, in South East Asia the undrained shear strength of the soft clay could be about 10 kPa. The thickness of the soft clay could reach about 50 m below ground surface.

2.2 Piles

The soft soil cannot take the external loads from the traffic and embankment without having large deformations. Hence, in a piled embankment, the loads are transferred to the much stiffer piles. The piles used for this purpose are generally prefabricated (driven) or cast in place displacement piles (jacked in or screwed piles). The piles are commonly concrete or timber piles with diameter ranging from 10 to 30 cm. However, larger diameter piles up to 60 cm diameter have also been used. The piles are preferably embedded to a competent stratum (end bearing piles). However, when the soft clay layer is thick, it is often that the piles cannot reach the competent stratum. These are called floating piles. For example in South East Asia, Bakau timber piles with maximum length of 4.5 to 6 m are often used. To increase the load transfer to the piles, pile caps are used. The pile cap size is usually determined as area-covering ratio, which is the relative area, covered by the cap to the total embankment area. Cap sizes with area covering ratio of 4% to 22% have been reported.

2.2.1 End Bearing and Floating Piles

The piles supporting the embankment as can be seen in Figure 2.1 are preferably embedded to a competent stratum (end bearing piles) such as firm soil or bedrock. However, when the soft clay layer is thick, it is often that the piles cannot reach the competent stratum. These are called floating piles. For example in South East Asia, Bakau timber piles with maximum length of 4.5 to 6 m are often used as floating piles.

2.2.2 Head-Settling and Non Head-Settling Piles

In relation to the design of embankment and geosynthetic tensile strength, the consideration of head-settling and non head-settling piles is more relevant. Head-settling piles settle, due to some settlement of the pile base at the bearing stratum and some pile shortening. This causes smaller differential settlement of the geosynthetic layer as shown in Figure 2.2. End bearing piles can be head-settling or non head-settling piles depending on the stiffness of the pile and the strength of the bearing stratum. Similarly, floating piles can also behave like head-settling or non head-settling piles depending on the pile penetration depth and pile stiffness. In practice, it seems that end bearing piles will not settle much and can be considered as non head-settling piles whereas floating piles can be both head-settling or non head-settling piles. In this report, the literature will be distinguished between embankment on end bearing and floating piles. This is because the terms are more general and easier to distinguish the studies.

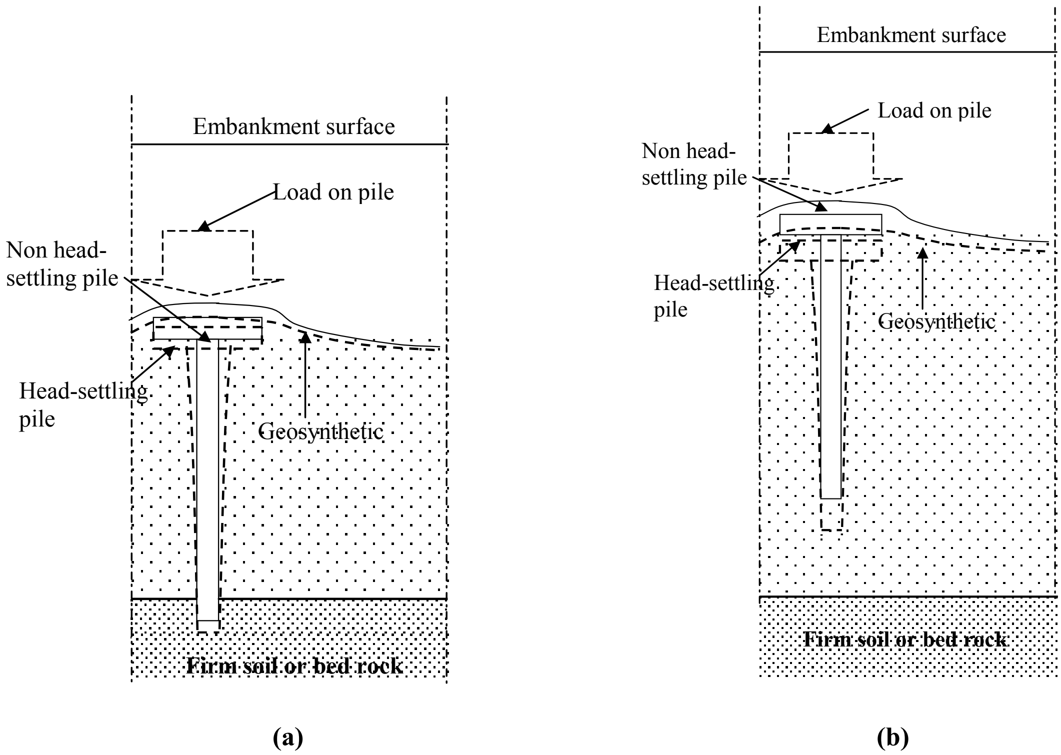


Figure 2.2: Non head-settling piles vs. head-settling pile (a) with firm end bearing stratum (b) without firm end bearing stratum (floating pile)

2.3 Embankment

To built infrastructure such as roads, a stiff layer or platform is needed. For this reason, embankments are built. This embankment needs to be stiff and strong. In order to reach this, it is required that the embankment must consist of good quality materials (sand and gravel or crushed stones) with wide grain size distribution and the internal friction angle should be higher than 30°. The minimum height of the embankment is determined to gain a full development of soil arching.

2.4 Geosynthetics

Embankments on piles are often reinforced with geosynthetics. The advantages of utilising geosynthetic layers such as reducing the need of large pile caps and the need of raking piles have been shown by Reid and Buchanan (1984). Based on the practical experience, the geosynthetic type used varies. The geosynthetics can have uni- or biaxial tension with the tensile strength varying between 20 to 1100 kN/m. Often up to 3 layers of geosynthetics are applied.

3. Current Empirical Design Methods

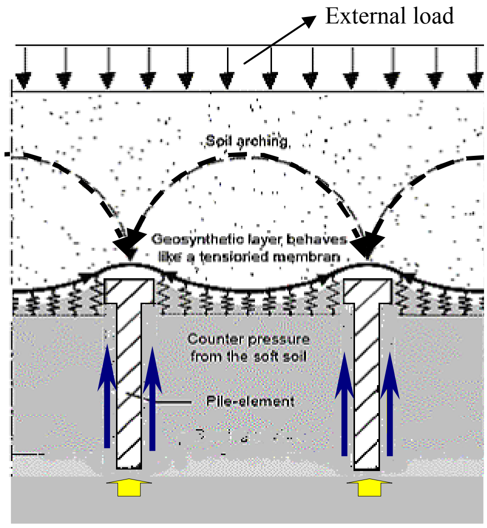


Figure 3.1: The Idea of Piled Embankment

The way of piled embankments work is described in Figure 3.1. The external load, for example from the traffic and the embankment above the soil arch is transferred to the piles via the soil arching mechanism. The embankment load below the soil arch will be bared by the geosynthetic and will be directed to the piles via geosynthetic tension. The piles transfer the load to the deeper and stiffer soil stratum. Thus, the soft soil experiences little force and therefore compaction because of the force are transferred through the geosynthetic and the piles.

The design of a piled embankment includes the design of embankment, geosynthetic and piles. There are several methods for designing piled embankment. Some of them are included in the existing design guidelines of piled embankment design such as Nordic Guidelines, British Standard 8006 and German method (EBGEO).

3.1 Design of Embankments

The design of embankment consists of the design of its geometry, stability and load transfer mechanism via soil arching. The geometry is chosen so that it satisfies the construction requirements, embankment stability and soil arching development. The embankment stability can be assessed using the common slope stability analyses such as Bishop's method, Fellenius's method or using numerical analysis such as finite element method. Since the design of geometry and embankment stability is relatively clear and definite, most uncertainties of the embankment design lie on the load transfer mechanism, which occurs via soil arching. Therefore, when improving the design methods of piled embankments, most attention should be addressed to this aspect. Soil arching is the typical stress propagation in the embankment body, which allows the transfer of external and large part of embankment loads to the piles. In order to have more understanding of soil arching, it will be briefly explained from an analytical point of view but also by the guidelines.

Arching is defined by McNulty (1965) as "the ability of a material to transfer loads from one location to another in response to a relative displacement between the locations. A system of shear stresses is the mechanism by which the loads are transferred". When a soil mass is placed on a rigid base, no tendency to differential movement exists so that soil arching does not develop. At the moment the soil loses a local support, an arch is formed (McKelvey III, 1994). Several different empirical approaches have been proposed to model the soil arching. In following sections five different methods for designing soil arching are presented.

3.1.1 Terzaghi's Method

Arching effects have been described by Terzaghi (1943). He describes the arching effects based on his experiment on the trap door effects. As shown in Figure 3.2, based on the vertical equilibrium of a soil element, one can write:

$$(\sigma_z + d\sigma_z) \cdot s - \sigma_z \cdot s + 2\tau_{xz}\, dz - dG = 0 \tag{3.1}$$

where:

- σ_z is the vertical effective stress
- τ_{xz} is the shear stress on the xz plane of the soil element
- s trap-door's width
- G is soil weight

It can be simplified as

$$d\sigma_z \cdot s = \gamma \cdot s\, dz - 2\tau_{xz}\, dz \text{ , with } \gamma \text{ is the soil unit weight} \tag{3.2}$$

According to the Mohr-Coulomb failure criterion, shear stress at failure can be expressed as

$$\tau_{xz} = c' + \sigma_x \cdot \tan\varphi' \tag{3.3}$$

with c' and φ' are the effective cohesion and friction angle of the soil. The effective horizontal stress as a function of vertical effective stress is $\sigma_x = \sigma_z \cdot K$. Hence Equation 3.2 can be written as follows:

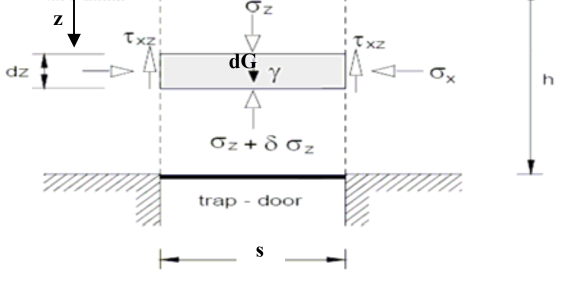


Figure 3.2: Description of soil Arching analysis with Terzaghi's method

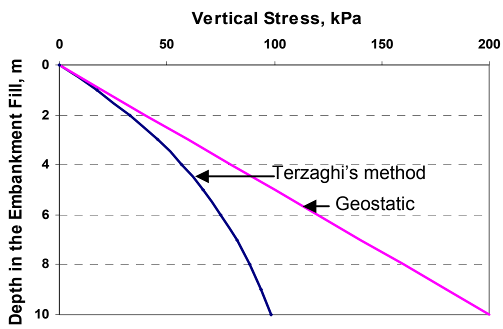


Figure 3.3: Typical vertical stress distribution of embankment fill between trap-door with Terzaghi's method

$$d\sigma_z \cdot s = \gamma \cdot s\,dz - 2\left(c' + \sigma_z \cdot K \cdot \tan\varphi'\right)dz \tag{3.4}$$

Dividing both sides of Equation 3.4 with σ_z and s we get a differential equation as below

$$\left(\frac{d\sigma_z}{\sigma_z}\right) = \frac{\gamma}{\sigma_z}dz - \frac{2c'}{s \cdot \sigma_z}dz - \frac{2K \cdot \tan\varphi'}{s}dz \tag{3.5}$$

The solution for the differential equation is as follows:

$$\sigma_z = \frac{s \cdot \left(\gamma - 2 \cdot c'/s\right)}{2 \cdot K \cdot \tan\varphi'} \cdot \left[1 - e^{-2K\tan\varphi' \cdot \frac{z}{s}}\right] + p \cdot e^{-2K\tan\varphi' \cdot \frac{z}{s}} \tag{3.6}$$

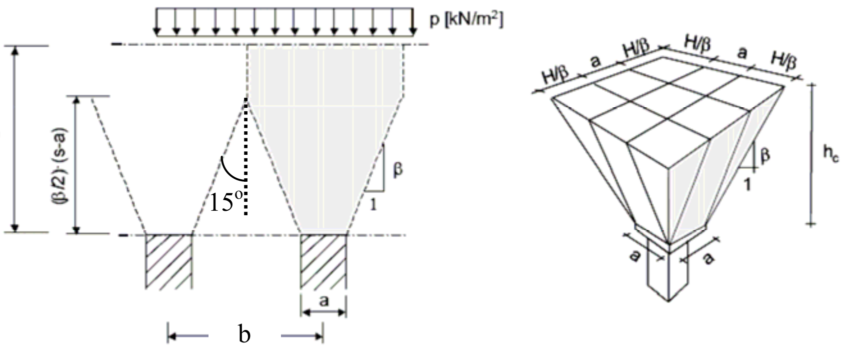
Based on his experimental results, Terzaghi determined that the K value is 1. The solution in Equation 3.6 gives an exponentially increasing vertical effective stress within the embankment fill between the two rigid foundations. A comparison between effective vertical stress distribution with the linearly increasing geostatic vertical stress is shown in Figure 3.2. Due to arching, the vertical stress acting on the ground surface below the embankment is much lower than the geostatic vertical stress.

3.1.2 Nordic Guidelines Method

This method uses a wedge shape soil arching suggested by Carlsson (1987). The method considers a wedge of soil whose cross-sectional area under the arching soil can be approximated by a wedge with an internal angle at the apex of the wedge equal to 30º (Figure 3.4).

The Method adopts a critical height approach such that any additional overburden above the top of the wedge is transferred directly to the columns. For a 2D approach, with the height of the embankment above the triangular area, the weight of the soil wedge per unit length can be calculated as:

$$w = \frac{\left(b - a\right)^2}{4\tan 15°}\gamma \tag{3.8}$$



(a) (b)

Figure 3.4: Soil wedge model: (a) 2D and (b) 3D

where:

- w is the weight per unit length (out of plane direction of Figure 3.5a)
- a is the width of square pile caps
- b is the centerline spacing of piles
- γ is unit weight of the embankment

The calculated soil wedge weight is the load on the geosynthetic layer. The rest of the embankment load is carried by the piles. Svanø et al. (2000) proposed a method considering the 3-D effect as shown in Figure 3.4b. The weight soil mass per pile cap side that will be transferred to the geosynthetics can be calculated as follows:

$$w_s = \frac{\gamma}{2a}\left\{b^2H - \frac{1}{6\tan\beta}\left[(a + H\tan\beta)^3 - a^3\right]\right\} \tag{3.9}$$

where:

- w_s is the weight of soil per pile cap side (half pyramid)
- a is the width of square pile caps
- b is the centerline spacing of piles
- γ is unit weight of the embankment
- H is the height of embankment
- β is the slope depicted in Figure 3.4b

This method that considers a pyramid type arching is popular mainly in the Scandinavian countries.

3.1.3 BS 8006 Method

The British Standard "Code of practice for strengthened / reinforced soils and other fills" (British Standards Institution 1995) has adopted an empirical method developed by Jones et al. (1990), which is based on a formula proposed by Marston and Anderson (1913) for soil arching on top of a buried pipe. In the BS 8006 Method, The arching is assume to be semi-spherical dome and it is independent of the type and strength properties of the embankment

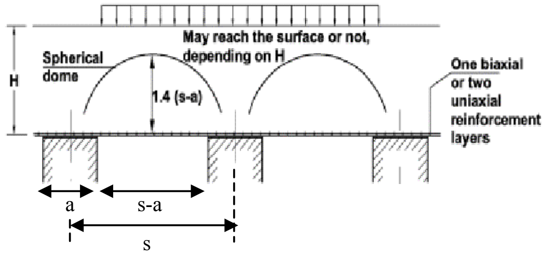


Figure 3.5 Arching dome according to British Standard (Alexiew, 2004)

fill (Figure 3.5). The ratio of the vertical stress on top of the caps to the average vertical stress at the base of the embankment, P'_c/σ'_v may be estimated as follows:

$$\frac{P'_c}{\sigma'_v} = \left[\frac{C_c\, a}{H}\right]^2 \tag{3.10}$$

where:

C_c soil arching coefficient defined as:

$1.95\,H/a - 0.18$ for end-bearing piles

$1.5\,H/a - 0.07$ for friction and other piles

a is the size of the pile caps

H is the height of the embankment

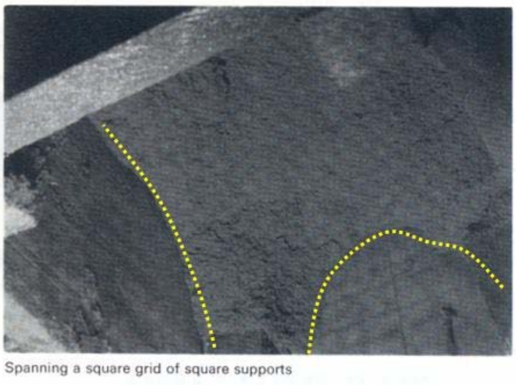
The majority of piled embankments designs follow this method although the load transfer (soil arching) design is still controversial. This might be because the BS 8006 is simple and familiar to engineering practice.

3.1.4 Old German Method

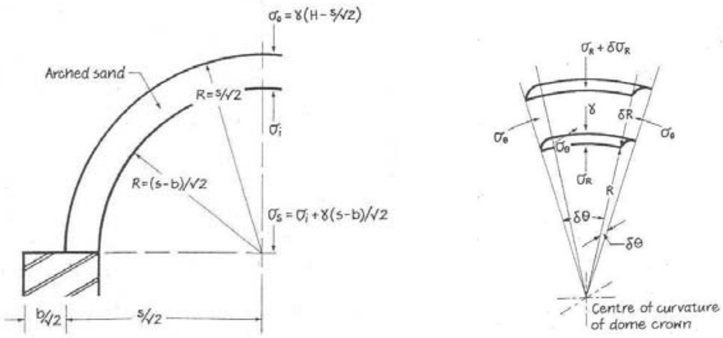
This method uses soil arching concept suggested by Hewlett and Randolph (1988). The method assumes that the arching in an embankment forms an arching shell with a shape of hemispherical dome as shown in Figure 3.6b and 3.6c. The thickness of the arching shell is $b/\sqrt{2}$ in the section of diagonal spacing of the squared pile grid. b is the width of a squared pile. Due to the soil arching, soil stresses are assume to be redistributed in this arching shell only. Outside the shell, the stress distribution is similar to initial stress distribution. This means that above and below the arching shell, the stress is increasing linearly with depth. Failure of the arching is assume to occur only at crown of the arch or at the pile cap. Equilibrium analyses at the two positions lead to two equations for the stress acting on the surface of the sub soil, σ_s as presented in the following:

a) Equilibrium analysis at the crown of arch

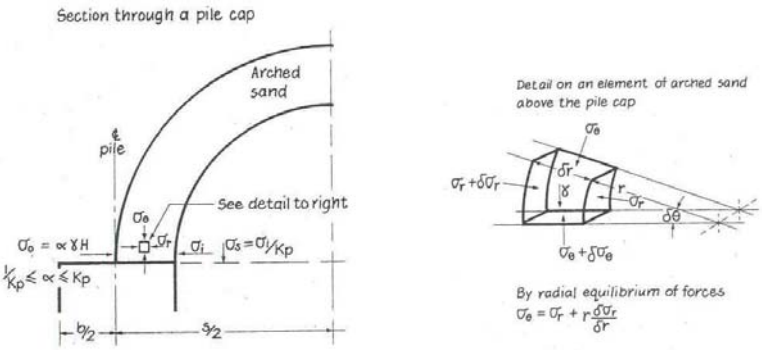
The analysis is in plane strain of arching shell with spherical geometry. Vertical equilibrium of soil at the crown of the arch as shown in Figure 3.6b requires that:



(a)



(b)



(c)

Figures 3.6: Soil arching (a) Experimental evidence (b) Equilibrium analysis at crown of arch (c) Equilibrium analysis at just above pile cap

$$\frac{d\sigma_r}{dr} + \frac{2(\sigma_r - \sigma_\theta)}{r} = -\gamma \qquad (3.11)$$

where $\sigma_\theta = K_p \cdot \sigma_r$, and K_p is the Rankine passive earth pressure coefficient which is equal to (1+ sin φ′) / (1- sin φ′). The inner radius is R = (s - b) / $\sqrt{2}$ and the outer radius is R = s / $\sqrt{2}$ with s is the centre-to-centre pile spacing. The corresponding vertical stresses are σ_i and $\gamma(H - s/\sqrt{2})$ as shown in Figure 3.6b, where H is the height of the embankment. Solving the Equation 3.11 subjected to the boundary conditions of inner and outer radius gives:

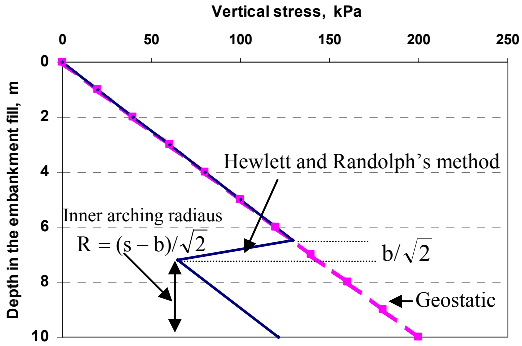


Figure 3.7: Typical vertical stress distribution of embankment fill along the centre of pile spacing with Randolph and Hewlett arching formula

$$\sigma_i = \left[\gamma(1-\delta)^{2(K_p-1)}\right]\cdot\left[H-\frac{s}{\sqrt{2}}\left(\frac{K_p-2}{2K_p-3}\right)\right]+\gamma\frac{s-b}{\sqrt{2}(2K_p-3)} \tag{3.12}$$

with δ is b/s. The total pressure acting on the subsoil σ_s is:

$$\sigma_s = \sigma_i + \gamma(s-b)/\sqrt{2} \tag{3.13}$$

b) Equilibrium analysis at the pile cap

At the pile cap, the vault comprises four plane strain arches, each occupying a quadrant of the cap. The equilibrium analysis is in plane strain at the pile cap section as shown in Figure 3.6c. The pressure acting on the subsoil, σ_s due to this equilibrium analysis is as below:

$$\sigma_s = \frac{\gamma\cdot H}{\left(\frac{2\cdot K_p}{K_p+1}\right)\cdot\left[(1-\delta)^{(1-K_p)}-(1-\delta)\cdot(1-\delta\cdot K_p)\right]+(1+\delta^2)} \tag{3.14}$$

The larger stress acting on the surface of subsoil is determined by the larger value between the result from Equation 3.13 and Equation 3.14. The Hewlett and Randolph formula will give a typical vertical stress distribution of the embankment fill along the centre of the arching dome as shown in Figure 3.6. This method gives a better approach of soil arching than the BS 8006 method. It takes into account the strength of the embankment fill. In this old German method, some support from the soft soil counter pressure can also be taken into account.

3.1.5 New German Method

The new German method adopts the multi shell arching theory proposed by Kempfert et al. (1997). The method uses the idea of the Hewlett and Randolph (1988) approach, with a modification for low-height embankments using multi shell arching theory. In the new approach, Kempfert et al. considers domed arches spanning between columns or pile caps. As shown in Figure 3.8, according to the vertical equilibrium, the following governing equation can be obtained.

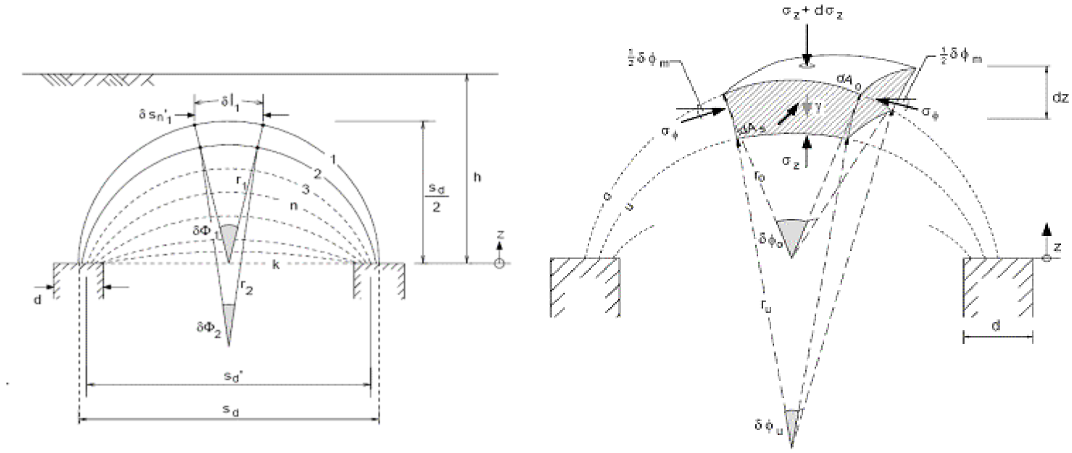


Figure 3.8: Multi-Arching model

$$-\sigma_z \cdot dA_u + \left(\sigma_z + d\sigma_z\right) \cdot dA_o - 4\sigma_\varphi \cdot dA_s \cdot \sin\left(\frac{\delta\varphi_m}{2}\right) + \gamma dV = 0 \tag{3.15}$$

where

$$dA_u = (r \cdot \delta\varphi)^2$$

$$dA_O = (r + dr)^2 \cdot (\delta\varphi + d\delta\varphi)^2 \approx 2d\delta\varphi \cdot r^2 \cdot \delta\varphi + 2dr \cdot r \cdot \delta\varphi^2 + r^2 \cdot \delta\varphi^2$$

$$dA_S = (r + \frac{1}{2}dr) \cdot (\delta\varphi + \frac{1}{2}d\delta\varphi) \cdot dz \approx dz \cdot r \cdot \delta\varphi$$

$$dV = (r + \frac{1}{2}dr)^2 \cdot (\delta\varphi + \frac{1}{2}d\delta\varphi)^2 \cdot dz \approx dz \cdot r^2 d\delta\varphi^2$$

$$d\delta\varphi_m = \delta\varphi + \frac{\delta\varphi}{2}$$

$$\sigma_\varphi = K \cdot \sigma_z$$

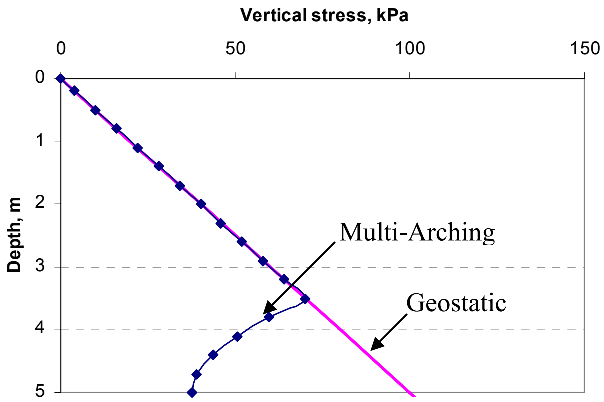


Figure 3.9: Typical vertical stress distribution of embankment fill along the centre of pile spacing with Multi-Arching model

The solution of vetical stress of soil between caps is as below:

$$\sigma_z[z] \approx (\lambda_1 + t^2 \cdot \lambda_2)^{-x} \cdot (\lambda_1 + z^2 \cdot \lambda_2)^{-x} \cdot \left\{ \begin{array}{l} (h-t)\cdot\gamma + \dfrac{(t-z)\cdot\gamma\cdot(\lambda_1 + t^2\cdot\lambda_2)^{x}\cdot(\lambda_1 + \dfrac{t^2\cdot\lambda_2}{4})^{-x}}{s_d\cdot(4\cdot\lambda_1 + t^2\cdot\lambda_2)} \cdot \\ \dfrac{\left[4\cdot s_d\cdot\lambda_1 + t\cdot(-2\cdot d\cdot(K_p-1)z) + s_d\cdot t\cdot\lambda_2\right]}{s_d\cdot(4\cdot\lambda_1 + t^2\cdot\lambda_2)} \end{array} \right\} \tag{3.16}$$

where:

$$k_p = \tan(45° + \frac{\varphi'}{2})$$

$$t = \begin{cases} h & \text{when} \quad h < s_d/2 \\ s_d/2 & \text{when} \quad h \geq s_d/2 \end{cases}$$

γ is unit weight of the embankment

h is height of embankment

$$x = \frac{d(k_p - 1)}{\lambda_2 \cdot s_d}; \quad \lambda_1 = \frac{1}{8}(s_d - d)^2; \quad \lambda_2 = \frac{s_d^2 + 2d\cdot s_d - d^2}{2\cdot s_d^2}$$

Figure 3.9 shows the vertical stress distribution in the embankment body along the centre of pile spacing. The multi-arching model is developed to improve stress redistribution model in the embankment body and to find a way for a reasonable consideration of a possible upward soft soil counter pressure between the piles (Alexiew, 2005).

3.2 Geosynthetic Reinforcement

Through the soil arching mechanism, the load that is acting on the soft soil and the load transferred to the piles can be determined. The load that is acting on the soft soil will be conveyed to the piles by tension in geosynthetics as shown in Figure 3.10. The tension of geosynthetics is evaluated using cable or membrane theory. All the guidelines for piled embankment design use the theories to determine the required tensile strength of the geosynthetics. However, the formulations are different depending on the method of determining the load on the geosynthetic.

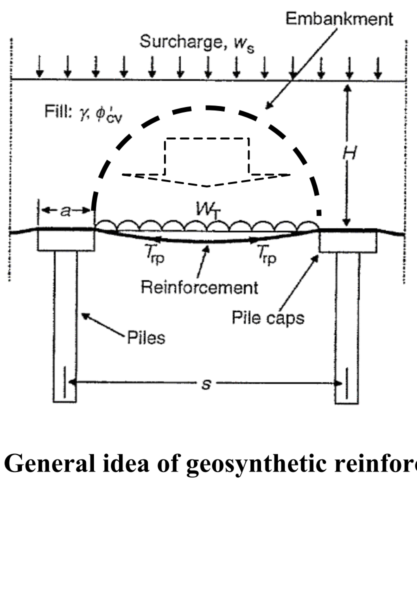


Figure 3.10: General idea of geosynthetic reinforcement (BS 8006)

One of the formulas to calculate the tension force in the geosynthetics based on the BS 8006 is as follows:

$$T_r = \frac{W_t(s \cdot a)}{2a}\sqrt{1+\frac{1}{6\varepsilon}} \tag{3.17}$$

where:

T_r	is the tensile force per meter geosynthetics
W_t	is distributed vertical load acting on the geosynthetics between the piles (Figure 3.10)
s	is the centre-to-centre spacing
a	is the pile cap size
ε	is the strain in the geosynthetics

In general, the distributed vertical load on the geosynthetics, W_t depends on the method of designing soil arching. If the method of designing soil arching is not realistic, the distributed vertical load is also not realistic which then leads to conservative or unsafe design of the geosynthetics.

The tensile strain of the geosynthetics is given and limited to certain value. According to BS 8006 and Nordic guidelines, the maximum allowable strain of 6% should be considered to ensure that the load of the entire embankment is transferred to the piles. In the New German method geosynthetic strength is evaluated based on several specific factors, however the maximum possible strain is also 6%. In addition to that all guidelines suggest to allow a maximum of 2% creep strain over the design life of the reinforcement.

In term of the number of geosynthetic layer, one strong biaxial tension geosynthetic layer is suggested by the guidelines.

3.3 On the Design of Displacement Piles

A displacement pile is a pile that due to its installation process displaces the soil to accommodate its volume. This type of piles is often used for the construction piled embankment. The best way to determine the pile's carrying capacity is by doing pile load tests. However, pile load tests at several representative locations are too costly. Several empirical approaches have been suggested to determine pile's capacity, which consist of determining the skin resistance and the base resistance of the pile. The approaches are the same as in the design of pile foundation.

Pile skin resistance can be estimated using empirical methods such as the total stress approach (α method), effective stress approach (β method), mixed method (λ method) or using empirical relation to the in-situ test data (CPT or SPT data). Similarly, pile base resistance can be estimated using several empirical approaches such as Vesic method, Meyerhof method, Salgado et al. method or using empirical relations to in-situ test data. The piles can be considered as end-bearing or floating piles. When considering end-bearing pile, the pile transfers the load directly to the hard stratum via end bearing. Pile capacity may be limited to the structural strength of the pile. On the other hand, if the piles are considered as floating piles, the capacity is determined mainly by the capacity of its skin resistance. Therefore, proper estimation of skin resistance is important. In addition to the empirical relations, it is also possible estimate the pile carrying capacity using finite element load test simulation. For example as suggested by Satibi et al. (2007) using K pressure method.

Once the pile capacity is known, the number of piles required, pile spacing, and pile cap size can be determined. The design of pile spacing and pile caps should also follow the rule that relates to the embankment height. For example, the BS 8006 method suggests that the piles' spacing should fulfil the rule of $h \geq 0.7 \cdot (s-a)$, whereas the new German method uses the relation of $h \geq 0.7s$. h is the embankment height, s is the centre-to-centre piles spacing and a is the size of pile cap side. After that, the effects of lateral loading to the piles and the adequacy of group capacity of piles are evaluated.

4. Literature Review on Numerical Analysis of Piled Embankment

The current analytical procedures for assessing the load transfer (soil arching effect) are conservative. Jones et al. (1990) stated that the reasons for this are twofold. Firstly, simplified analytical procedures rely on empirical relationships to quantify the arching mechanism across the adjacent pile caps. Secondly, the simplified procedures cannot accurately take into account partial foundation support beneath the geosynthetic reinforcement.

Therefore a more accurate method such as numerical analysis is needed to analyse complex soil-structure interactions like piled embankments. The influencing parameters on the performance of piled embankments, which are not considered in the analytical procedure, can be taken into account.

In the numerical analyses of piled embankments, authors have used several terms to describe the load transfer, geosynthetic tension and settlements behaviour of piled embankments. The following section explains the definitions of the terms used for piled embankment analysis.

4.1 Definitions

In order to evaluate the performance of a piled embankment construction, some definitions have been used by different researchers. The following terms and their definitions will be often referred in the next sections.

4.1.1 Efficacy

The efficacy E of the pile support is defined as the proportion of embankment weight carried by the pile caps. This may be expressed as (Hewlett and Randolph, 1988):

$$E = \frac{P}{s^2 \gamma H} \tag{4.1}$$

where :

- P is the total force carried by the pile caps
- γ is unit weight of the embankment
- H is height of embankment; and
- s is pile centreline spacing

Apart from the above definition, Sovulj (2005) used the definition of efficacy, which is the ratio of load carried by the pile that is composed of skin friction and base resistance, to the load of one cell embankment.

$$E = \frac{Q}{(s/2)^2 \cdot \pi \cdot \gamma \cdot H} \qquad \text{with} \quad Q = Q_s + Q_b \tag{4.2}$$

where :

- Q is the total force carried by the pile

Q_s is the load carried by skin friction

Q_b is the load carried by base resistance

γ is unit weight of the embankment

H is height of embankment

s is pile centreline spacing

4.1.2 Soil Arching Ratio or Stress Reduction Ratio

The definition of the term "soil arching ratio" is the same as "stress reduction ratio". The term soil arching ratio was used by McNulty (1965) to describe the degree of soil arching. For the same purpose, Kempton et al. (1998) used the term stress reduction ratio instead of soil arching ratio. The soil arching ratio or stress reduction ratio is defined as follows:

$$\rho = \frac{p_b}{\gamma H + q_0} \tag{4.3}$$

where :

ρ is soil arching ratio or stress reduction ratio, where:

$\rho = 0$ represents the complete soil arching

$\rho = 1$ represents no soil arching

γ is unit weight of the embankment

H is Height of embankment

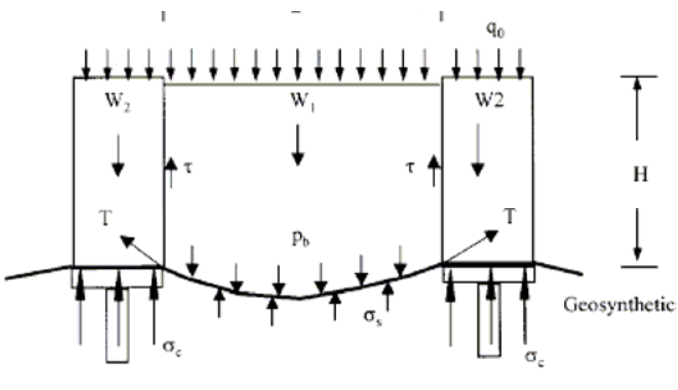


Figure 4.1: Load transfer mechanisms of piled embankments

q_0 is uniform surcharge on the embankment

p_b is average vertical pressure above geosynthetic

The description of the symbols is depicted in Figure 4.1.

4.1.3 Stress Concentration Ratio

The degree of the load transfer is quantified by Han (2003) using an index-stress concentration ratio, n, which is defined as the stress on the pile caps to that on the soil.

$$n = \frac{\sigma_c}{\sigma_s} \tag{4.4}$$

where

n is stress concentration

σ_c represents the vertical stress on the pile caps

σ_s represents the vertical stress on the soil

The load concentration factor can also reflect the load transfer degree, which is defined by Schmidt (2004) as:

$$LKF = \frac{Q}{Q_s} = \frac{\sigma_c \cdot A_c}{\gamma \cdot H \cdot A_c} \tag{4.5}$$

where :

LKF is load concentration factor

σ_c is the vertical stress on the pile caps

γ is unit weight of the embankment

H is Height of embankment

A_c is area of pile caps

4.1.4 Maximum and Differential Settlement

The term maximum and differential settlements are generally referred at the level of pile head (soft ground surface). The differential settlement is defined as the settlement difference between the centre of the pile head and the middle the pile spacing at the pile head level. For other position of observed settlements, such as on the embankment surface, the term will be mentioned specifically.

4.1.5 Geosynthetic Tension

Numerical studies indicate that tension is not uniform along the geosynthetics and the maximum tension generally occurs at the edge of the pile. For design purposes, the maximum tension in geosynthetics is of more interest.

4.2 Parameter Studies

Numerical analysis is capable to analyse complex soil-structure interactions. Almost all influencing parameters related the performance of a piled embankment can be accounted for in the analysis. Parametric studies using numerical analyses have been performed by several researchers. In this literature review, the parameter studies are distinguished into embankment on end bearing piles and on floating piles. This seems to be more general and simple rather than the consideration of embankment on head-settling and non head-settling piles. The later consideration is not particularly discussed in the referred literature except in term of influence of pile stiffness.

4.2.1 Embankment on End Bearing Piles

4.2.1.1 Embankment Height

It is obvious that increasing the embankment height will increase the load transferred to the piles. As a consequence, stress concentration ratio increases with the increase of embankment height. This has been shown by Han and Gabr (2002) and Ganggakhedar (2004) through their studies using 2-D axisymmetric finite difference analyses and 2-D axisymmetric finite element analyses of a piled embankment (a cell) respectively. Figure 4.2 shows the influence of embankment height to stress concentration ratio. Similar results are also shown by Suleiman et al. (2003) from 2-D plane strain analyses.

Some authors use the definition of soil arching ratio ρ to evaluate the load transfer mechanism. Han and Gabr (2002) show that soil arching ratio decreases asymptotically to a certain value with increasing embankment height. Suleiman et al. (2003) show also similar results from 2-D plane strain analysis. 3-D analyses confirm the tendency although the amount is higher (Kempton et al., 1998).

Another way to express load transfer is using the term efficacy as determined in Chapter 3. The results from 2-D axisymmetric analyses by Van der Stoel et al. (2006) and plane strain analyses by Jenck et al. (2007) show that efficacy increases with the increase of embankment height. Van der Stoel et al. (2006) also show the comparison of numerical calculations of efficacy with the calculation results from various empirical methods. The analytical results of empirical methods show lower efficacy compared to the one obtained by numerical calculation. Furthermore the results from different methods show large scatter.

Sovulj (2005) used the term of efficacy based on the study of Hewlett and Randolph (1988) to describe the load transfer. However, when the piles are with caps, she considered only the load carried by piles (excluding the cap contribution). With that definition, for a relatively low shear strength embankment fill material, it is shown that efficacy is increasing up to a certain maximum value with the increase of embankment height.

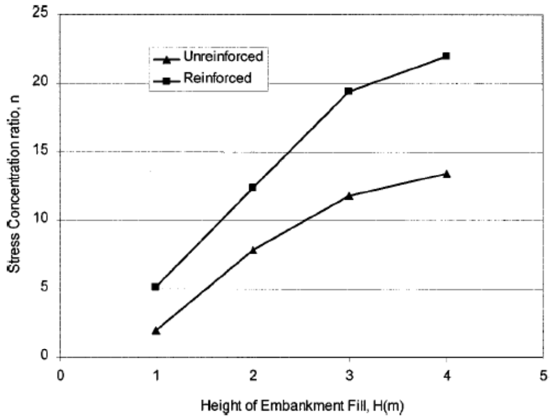


Figure 4.2: Influence of embankment height to stress concentration ratio (Han and Gabr, 2002)

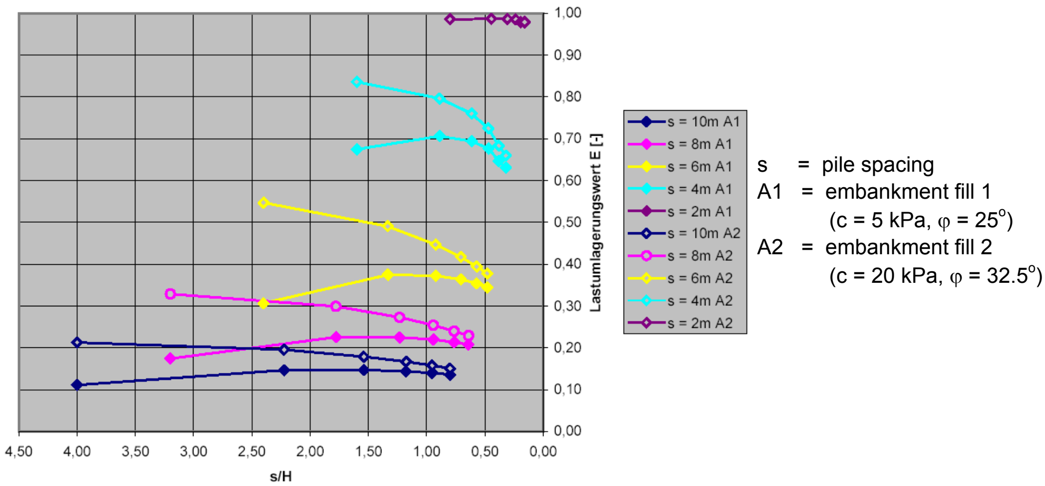


Figure 4.3: The change of efficacy to the ratio of spacing to embankment height (Sovulj, 2005)

Above that value, the efficacy decreases with the increase of embankment height. As shown in figure 4.3, the results from her 2-D axisymmetric analyses suggest that there is an optimum value of embankment height for specific pile spacing. However, it can also be seen that this optimum value does not exist if the shear strength of the embankment fill material is relatively high.

The maximum and differential settlements of piled embankment as well as the geosynthetic tension will increase as the embankment height increases. This is confirmed by all referred studies. In addition to that, Russell and Pierpoint (1997) based on their study on 3-D finite difference analyses showed that the construction process of the embankment up to the final height plays an important role. It is shown that higher maximum settlements, stress on the pile head and geosynthetic tensions were found when the embankment fill is placed in one step compared to the results from placing the embankment fill stepwise.

4.2.1.2 Pile Spacing

Increasing pile spacing obviously decreases the embankment load transferred to the piles. This leads to the decrease of efficacy, geosynthetic tension and the increase of maximum and differential settlements and soil arching ratio of a piled embankment. These conditions have been shown by numerical studies.

Instead of using spacing some authors use dimensionless unit as the ratio of spacing to embankment height (s/H) such as Sovulj (2005) or the ratio of embankment height to cap to cap distance (H/(s-a)), where a is the diameter of the pile cap. The later dimensionless unit has been used for example by Van der Stoel (2006). The suggestion from BS 8006 that the effect of arching starts if $H \geq 1.4\,(s - a)$ is confirmed by the 2-D axisymmetric numerical analyses carried by Van der Stoel (2006). On the other hand, Cortlever and Gutter (2006) based on their 2-D axisymmetric numerical analyses stated that the complete arching does not exist if $H \geq 1.4\,(s - a)$.

4.2.1.3 Pile Stiffness

The existing empirical design methods for calculating load transfer assumes that the pile is rigid. Therefore the effect of pile stiffness to the load transfer is not considered. Using numerical analysis, the influence of pile stiffness can be evaluated. Numerical studies about the influence of pile stiffness or the ratio of pile stiffness to the soft soil stiffness on the embankment load transfer have been done by Han and Gabr (2002), Suleiman et al. (2003) and Ganggakhedar (2004).

The studies show that increasing pile stiffness leads to the increase of stress concentration ratio n, geosynthetic tension T and differential settlements Δs asymptotically to a certain values as shown in Figures 4.4. Maximum settlements at the embankment surface as well as at the pile cap level and soil-arching ratio are decreasing with the increase of pile stiffness. The influences of pile stiffness as mentioned above, seem to be vanished (almost zero) if the pile elastic stiffness is higher than 1000 MPa and the ratio of pile-soil stiffness is higher than 200. For this particular case, it implies that the pile with that stiffness is considered as non head-settling pile. It is worth noting that almost all parametric studies using numerical analyses referred in this reports except the studies from Sovulj (2005) placed the fixity for the bottom boundary condition directly at the pile tip or the pile side and top as fixed boundary (pile is perfectly rigid).

For parametric studies, the assumptions of the above boundary conditions are good to observe the influence of specific parameters. However, it is questionable how well the model simulates reality. It is not always a very hard stratum or rock layer occurs at a required depth. Therefore the stiffness of the soil at the bearing stratum can be of as much important as other parameters. This consideration leads to concern whether the embankment should be designed as head-settling pile or non head-settling pile. This has consequences on the prediction of embankment settlements and the choice of geosynthetic strength.

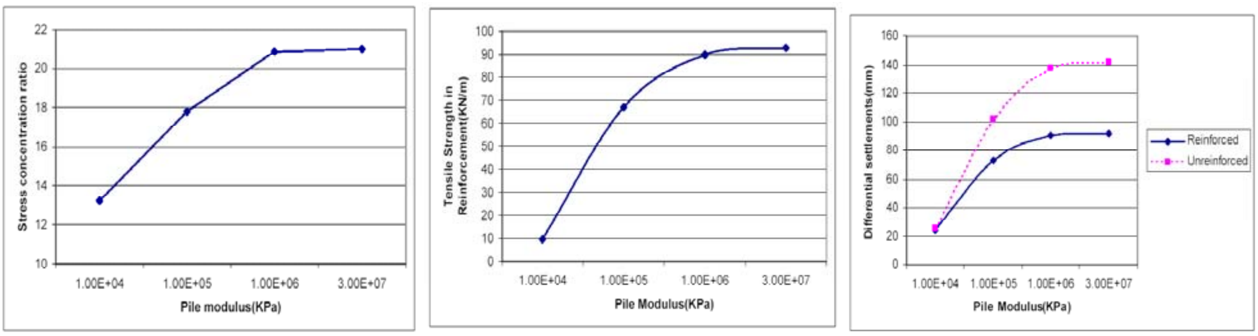


Figure 4.4: The influence of pile stiffness on the stress concentration ratio, geosynthetic tension and differential settlements (Ganggakhedar, 2004)

4.2.1.4 Geosynthetic Stiffness

The choice of geosynthetic stiffness influences performance of a piled embankment. 2-D axisymmetric numerical analyses by Han and Gabr (2002), Ganggakhedar (2004) have shown that the stiffer the geosynthetics used the lower the maximum and differential settlements of the embankment and the higher the stress concentration ratio. Increasing geosynthetic tensions with the increase of geosynthetic stiffness used are also observed.

In contrast to the advantage of using the geosynthetic, Sovulj (2005) found that the effect of using geosynthetic membrane on the embankment load transferred to the piles is not significant compared to without using geosynthetic membrane. Furthermore, almost no effect of using geosynthetic membrane observed if the shear strength of the embankment fill is relatively high. Similar results have also been found by Suleiman et al. (2003) using 2-D plane strain finite element analyses. They found that there is almost no different in maximum and differential settlements between with and without geosynthetic membrane. Moreover, the geosynthetic membrane does not seem to transfer significant load to the piles, hence there is no significant different on the stress concentration ratio observed. In addition to that, similar studies by Sa et al. (2001) concluded that the number of geosynthetic layers had more influence on the settlements behaviour than the value of the geosynthetic stiffness as long as the tensile stiffness was greater than about 1000 KN/m.

Concerning the number of geosynthetics used, Arwanitaki and Triantafyllidis (2006) from plane strain FE-analyses shows that if more layers of geosynthetics with the same stiffness are applied, the amount of maximum tension in the lowest geosynthetics is not reduced significantly compared to when using one layer of geosynthetics with equivalent stiffness. Similar finding is also shown by Heitz (2006) that the maximum tension in the lowest layer of the three geosynthethic layers applied is almost the same as if one layer of geosynthetics is used.

4.2.1.5 Soil Models and Parameters

Different soil models have been used for parametric studies of piled embankments. Mohr-Coulomb model for the embankment material has been very often used such as by Russell and Pierpoint (1997), Suleiman et al. (2003), Ganggakhedar (2004), Van der Stoel (2006), among others. Jenck et al. (2006) and (2007) adopted a modified Mohr-Coulomb model with stress dependent stiffness. Jones et al. (1990) and Han and Gabr (2002) used non-linear hyperbolic soil model by Duncan and Chang.

Although different soil models have been applied for the numerical analyses, there are no studies reported about the influence of soil models to the performance of a piled embankment. Intuitively, there should be a difference to load transfer mechanism as different soil models use different assumption for the stiffness and yielding behaviour of the soil. For example, the Mohr-Coulomb model, which incorporates one stiffness for the entire soil, will produce different stress field due to arching compared to advanced soil models, which consider stress level dependency. Hence the embankment load transfer should be different as well. To what extent that they are different still needs to be studied.

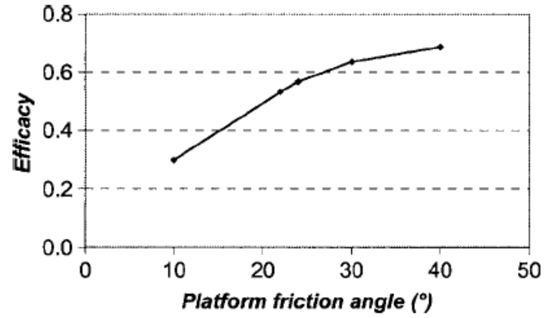


Figure 4.5: The influence of embankment fill friction angle on the efficacy (Jenck et al., 2007)

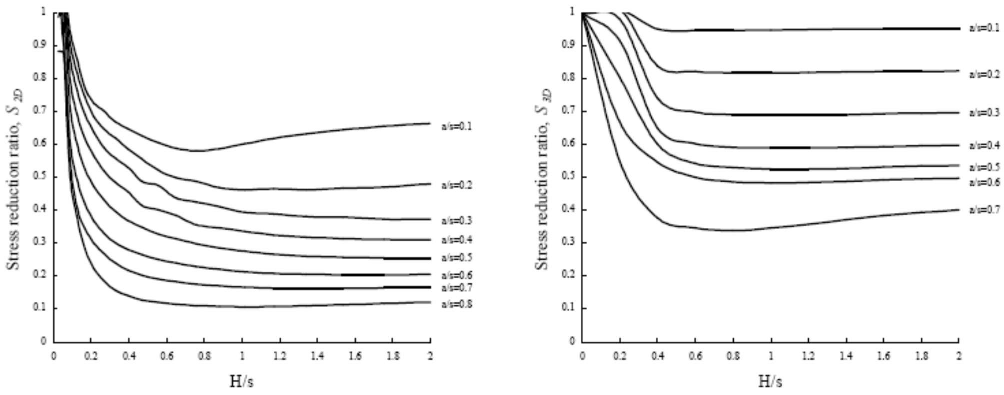
Unlike soil models, there are several studies about the influence of soil parameters on the performance of a piled embankment. Jenck et al. (2006) studied the influence of embankment soil stiffness and dilation angle to the load transfer mechanism using 3-D finite difference analyses. It was found that lower soil stiffness and low dilatancy angle of an embankment material lead to a lower efficacy. In addition to that, they also observed that as the pile is rigid, the compressibility of the soft soil has no influence on the load transfer.

As soil arching is driven by shearing mechanism, the shear strength soil parameters determine the amount of load transfer. Sovulj (2005) investigated the influence of embankment soil's effective shear strength (c′ and φ′) to the embankment load transferred to the piles. Higher embankment load transferred to the piles is observed if stronger soil is used for the embankment fill. Similarly, Jenck et al. (2007) showed also that the higher the friction angle of the embankment fill material the higher the efficacy as depicted in Figure 3.5. Nevertheless, the increase of efficacy is not linearly proportional to the increase of friction angle. Less considerable increase of efficacy observed if the friction angle is more than 30^{o}. This finding supports the suggestion by Hewlett and Randolph (1988), which states that the friction angle of material to be used as embankment fill should be at least 30^{o}. This suggestion is based on their experimental results.

The influence of embankment soil parameters on the settlements is consistent with the influence on the load transfer. The higher the soil strength parameters and soil stiffness the lower the maximum and differential settlement at the embankment surface.

4.2.1.6 On the Comparison between 2-D and 3-D Analysis

Complex pile embankment system is a three-dimensional problem. Thus, the true behaviour of the system can only be represented in 3-D analyses. As shown by Hewlett and Randolph (1988), arching mechanism in piled embankment appears to be hemispherical dome leaning on four piles. Therefore, neither plane strain nor axisymmetric 2-D analyses can accurately reproduce the behaviour. Plane strain will produce half tube type arching and axisymmetric will reproduce "umbrella shape arching" resting on a single central pile cap (Kempton et al., 1998: Noughton et al., 2005).



(a) 2-D (b) 3-D

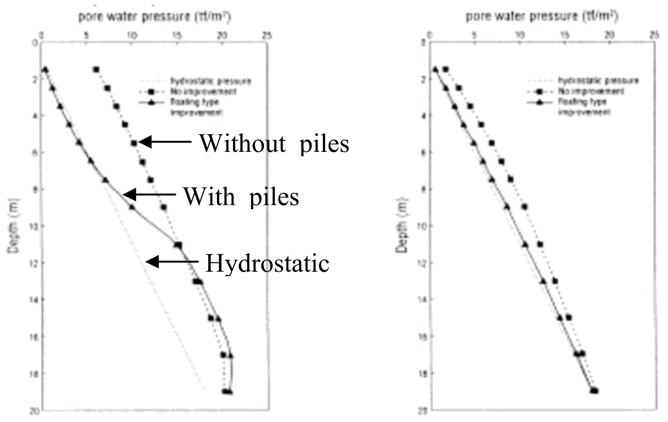
Figure 4.6: The comparison between 2-D plane strain and 3-D analyses (Kempton et al., 1998)

Several studies have been performed to show the different behaviour of soil arching effect from 2-D and 3-D analyses. Kempton et al. (1998) compare the load transfer and settlements behaviour between 2-D plane strain and 3-D analyses of a quarter cell embankment. The results show that although the tendency is the same, the magnitudes are considerably different. The soil arching ratio (stress reduction ratio) with respect to H/s are much lower in 3-D analyses compared to 2-D. Higher maximum and differential settlements are observed in 3-D analyses. Figure 4.6 shows the results of soil arching ratio from 2-D and 3-D analyses. The results seem to be accurate as the proportion of piles is lower than in 2-D plane strain. Similar findings have also been shown by Sovulj (2005), where comparison between 2-D axisymmetric and 3-D analyses is shown. The calculated maximum settlements of the embankment surface and differential settlements are higher in 3-D analyses. Furthermore, Jenck et al. (2006) showed that using 3-D half slice of embankment model, it is possible to observe the lateral pressure and movement of the embankment. On the other hand, Zaeske (2001) shows that the 3-D analysis can be well approached using 2-D plane strain analysis with conversion method suggested by Bergado and Long.

3-D analyses need a high computational and storage resources availability. Therefore, these analyses have been considered only recently. Nowadays, the use of 3-D numerical analysis for piled embankment design is recommended as it represents the reality more accurately (Russell and Pierpoint, 1997: Kempton et al., 1998: Jenck et al., 2006).

4.2.2 Embankment on Floating Piles

Publication about parameter studies on embankment on floating piles can hardly be found. It may be because most of the parametric studies mentioned previously try to compare or to improve the design from the existing empirical methods with the results from numerical analyses. As most of the existing empirical methods consider only end bearing (non settling) piles, therefore the numerical analyses are conducted for embankments on end bearing piles. Moreover, it may also because there is not enough experience and confidence on designing embankment on floating piles, despite there are embankments on floating piles have been constructed.



(a) **(b)**

Figure 4.7: Distribution of pore water pressure in clay at the centre of piled embankment (a) immediately after construction (b) after 2 years

Nevertheless, a parametric study on embankment on floating column type foundations (deep mixing columns) has been shown by Miki and Nozu (2004). The study was done using finite element plain strain analysis. The piles are embedded to 12 m below the surface of a Bangkok soft clay which thickness is 20 m. the study is focused on the influence of using floating piles on the settlement of soft ground surface and pore water pressure change. Thus, the analysis is consolidation analysis. Geosynthetics is not applied and creep is not accounted for in this study. The calculation results show significant improvement of soft ground surface settlement (more than 60%) after one month from the construction completion when using floating piles compared to without piles (settlement without piles is about 1.1 m).

The percentage of improvement between the two cases increases with time. In addition to that the excess pore pressure occurs only in the clay below the floating piles and it diminishes after 2 years from construction completion (Figure 4.7). It seems that the embankment load has no influence on the soft soil surrounding the floating piles.

4.3 Piled Embankment Design and Back Analysis

Designs of piled embankments and back analysis on the measured data using numerical analyses have been shown by several authors. The studies show the possibility and the method of using numerical analysis for the purpose of design and prediction of the piled embankment performance.

4.3.1 End Bearing Piles

Pleomteux and Porbaha (2004) show designs of embankment on end bearing columns for roadway construction using numerical analysis. The design is aimed to check on the preliminary designs and to have information about the short and long term settlements of the embankment. Two designs have been considered with different embankment height, pile diameter and thickness of soft soil layer. The spacing of the columns is 1 m, which are about 2 to 3 times the column diameters. The settlement performance is evaluated using finite element axisymmetric analyses and the stability of the embankment slope is calculated using a separate slope stability program. The design shows that the change of embankment settlement of only 4 mm within 5 to 10 years from the construction completion.

On the other case, finite element axisymmetric and plain strain analyses have been performed by Van der Stoel et al. (2006) to design a pile embankment for Barendrecht railway construction. The design is focused on the evaluation of soil arching and geosynthetic tension. The results from numerical analyses are compared to the calculation results from existing analytical methods. It is obtained that the calculation results from different methods are inconsistent with the finite element calculations. For safety reason, it was decided that the design follows the BS 8006 method. Later on, the verification with measured data shows that the measured tension in geosynthetics is higher than the finite element prediction and much lower than the calculation result from BS 8006. This fact implies that further research is required to find a better design procedure to have better confidence in the design as well as economical optimisation.

Numerical back analyses of completed piled embankment projects have been performed such as by Han et al. (2005) and Ganggakhedar (2004). Han et al. (2005) perform numerical analyses of embankment on deep mixing column constructed in Finland. The study is focused on the back analysis of maximum settlement and maximum goesynthetic tension from the measured data. The columns are assumed to be walls with width of 0.7 m in plain strain finite difference analysis. Linear elastic perfectly plastic soil model is used for the soft soil and embankment fill. One strong layer of geosynthetics with tensile strength of 1700 kN/m is applied. For the contact between the between the geosynthetics and sand, interface is used with shear strength of 85000 kN/m/m. From the analyses, it is shown that the maximum settlements and geosynthetic tension can be predicted reasonably well. Very small differential settlements on the surface of the embankment are mentioned to show that arching is developed in the embankment.

On the other study, Ganggakhedar (2004) performs finite element plain strain modelling of five completed piled embankment projects. In the study, focus is put on the evaluation of lateral movements of embankment toe, bending moment of the piles and geosynthetic tensile force. However, instead of comparing to measured data, the results are compared to the results from analytical method. As for modelling the piles, beam walls represented by beam elements in combination with interfaces are used. Soft soil model (Cam-clay) is used for the soft soil material model. Significant different results are obtained between numerical analyses and analytical results. It appears to be rather difficult to judge which method give a better approximation. However, from the point of view of a researcher, the result from numerical analysis results is considered more reliable as it takes into account the influence of many factors such as soft soil counter support, pile stiffness, pile and geosynthetic resistance to horizontal load whereas those factors are not accounted for in the analytical methods.

In addition to that, it is known that there are much data on the Kyotoweg project and might probably the project have been simulated numerically. However, the publication about the numerical studies is not found yet.

4.3.2 Floating Piles

Although publication on numerical design of embankment on floating piles is rare, quite recently Poulos (2007) has proposed design charts and procedure for embankment on floating piles. The charts are for preliminary design purpose of piled embankment in a typical Malaysian soft marine clay deposit, which is considered typical for South East Asia region. The design charts has been developed based on numerical analyses of pile and pile groups settlements. However, the insights of the performed numerical analyses are not available. In the design chart, attention is focused on the geotechnical capacity and settlements of the piles. However, consideration on the lateral response and structural capacity of the piles are also included. In this design procedure, some design issues such as soil arching, geosynthetic tensile strength and creep settlement are not covered. It would seem that the design charts are good tools for preliminary design of embankment on floating piles.

On the other case, The (2006) performs creep analyses of trial embankments on floating piles. The trial embankments are located in Bereng Bengkel, Kalimantan, Indonesia. A plain strain finite element analysis has been performed with the Soft Soil Creep model for the soft soil material. The soft soil consists of peat and soft clay up to 18 m depth. No geosynthetics is used. The piles are modelled as beam elements. Monitoring data shows settlements of the trial embankments of 1 to 2 m. therefore the finite element calculations are performed using creep and updated mesh analyses. Reasonably well approximations with the measured data of embankment settlement have been shown.

5. Discussion

Piled embankment design is a complex soil-structure interaction problem. All design methods especially empirical design methods use some assumptions to simplify the design. Therefore, some influencing factors are not taken into account. Numerical analysis of piled embankment design can be an adequate way to model the complex soil-structure interaction problems. However, there are uncertainties on the use of numerical analysis. The next section discusses the concerns or the uncertainties about design with empirical methods and numerical analysis of piled embankments.

5.1 On the Empirical Design Method

In the empirical methods for designing piled embankments, it is assumed that the piles are rigid. In the reality, it can happen that a very hard stratum or rock layer does not occur at a required depth. Therefore when considering embankment on floating piles, the methods must be reconsidered. This leads to the question of the degree of soil arching related to the settlement of piles (head-settling piles) especially when using timber piles. It is worth to note that particularly in the South East Asian region, this pile can be a very important option. The consequence of considering the piles as head-settling piles can be that the load on the geosynthetics due to soil arching is less. This is due to the slight settlement of the pile head, which leads to smaller differential settlement between the pile head and the surrounding soil. Thus, there will be more counter support from the sub soil to the geosynthetic layer. Finally, it leads to the choice of geosynthetic tensile strength requirement. However, by which pile head settlement relative to the soft soil settlement that the pile is considered as head-settling pile or not and to what extent the counter support available in time have not yet been investigated. As mentioned in Section 2.2.2, end bearing and floating piles can be head-settling or non head-settling piles. This depends on the load on the pile, stiffness of the pile, the strength and stiffness of the bearing stratum and the pile penetration depth in the case of floating pile. Practically, it seems that end bearing piles can be considered as non head-settling piles whereas floating piles can both non head-settling and head settling piles. These considerations are rather complex and numerical methods are the best tool to clarify this concern.

Design results from different empirical methods show large difference with each other (Van der Stoel et al., 2006; Van Eekelen and Alexiew, 2007). This is mainly due to different assumptions taken when developing the formulation for the soil arching. When looking at the five different methods discussed previously, all methods suggest a different arching mechanism.

The BS 8006 method, which assumes a semi-spherical dome arch independent of the type and strength properties of the embankment fill, has been used extensively despite some controversies on the assumption. Scarino (2003) states that Clarke (1968) refers the Marston and Anderson (1913) as “historical”, which to some could imply that it is not sufficient for current design requirements. Scarino (2003) mentions that the Marston’s formula may be inadequate and re-examination of the Marston model and the procedures used in the testing is needed. Love and Milligan (2003) observed that the BS 8006 Method does not satisfy vertical equilibrium. For high values of the ratio of pile cap width to pile spacing, the BS 8006 Method can give negative values of soil arching ratio. These imply that the method is unreliable. Nevertheless, the design results using BS 8006 generally lead to conservative design of pile embankment (Jones et al. 1990) and many projects have been completed successfully with this method.

The old German method, which is based on the Hewlett and Randolph method, appears to be a more realistic method in the way that the soil arching formulation takes into account the strength properties of the embankment fill. This method is improved in the new German method, which is based on the multi-shell arching developed by Kempfert et al. (2004). The method improves the stress distribution model in the embankment fill and suggests a reasonable consideration of the possible counter pressure from the soft subsoil. Nevertheless, the method is new and still needs to be proven in practice. Moreover, it is also questionable whether the sub soil counter pressure will remain unchanged under the arch in the case of consolidation and creep. To what extent and under which conditions the sub soil counter pressure remains should be investigated.

The calculated tension in the geosynthetics depends on the reliability of the soil arching design. To propagate tension in the geosynthetics, some allowable strain is required. All guidelines suggest of allowable strain of maximum 6%. However, in the new German method, the determination of geosynthetic strain is more specific. In practice, it is often to design with geosynthetic strain of 3 to 4%. The use of geosynthetic strain of about the half of the maximum allowed strain shows the level of uncertainties in the design of geosynthetic.

There are also uncertainties in calculating the tension in the geosynthetics. The BS 8006 suggests that designed tension in the geosynthetics should be the total of the geosynthetic tension due to soil arching calculation and the load due to the lateral earth pressure of the embankment. On the other hand, Love and Milligan (2003) suggest that the designed tension in the geosynthetics should be only the maximum of the two components. Heitz (2006) shows based on his model test piled embankment that for embankment up to 3 m, the Love and Milligan (2003) assumption is a realistic assumption whereas the BS 8006 assumption overestimates the test data. Nevertheless, this finding need to be further verified especially with field measurements. Moreover, the number of geosynthetic layers, which is used in practice, is often higher than suggested in the guidelines. The reason for this may probably because it gives more confidence in using a low embankment or larger pile spacing. It can also be because the tensile strength requirement becomes lower. This will then lead to economic optimisation. Nevertheless, the adequacy of this practice should be verified.

Another important aspect that is not covered by the empirical design methods is the settlement behaviour and horizontal movement of the piled embankment. This is very important considering the serviceability of the piled embankment. Especially when using floating piles, short and long term settlement behaviour will be of most importance.

Another concern is the consequence of local failure of for example, one or more piles or the snap of the geosynthetics. To what extent it will influence the performance of piled embankment system and what possible measures can be taken to mend the problem needs to be investigated. As this aspect is not accounted for in the empirical methods, numerical analysis should able to suggest some insight into this problem.

5.2 On the Numerical Analysis of Piled Embankment

Most of the numerical analyses of piled embankments referred to in this literature review assume that the piles are “wished in place” piles (non displacement piles) although in reality the piles are often displacement piles. This procedure is used because it is not simple to include effect of pile installation in the numerical analyses. This analysis is commonly based on small strain assumption whereas pile penetration is complex problem, which involve large strain. However, there is a possible way to account for the effect of installation process of a displacement pile using small strain finite element analysis which is based on the effective stress (β) method and cylindrical cavity expansion as shown by Satiric et al. (2007). This approach, which is commonly used in the engineering practice, can be included in the piled embankment analysis using available small strain numerical methods especially for embankments on floating piles.

Regarding soil arching and geosynthetic design, Van der Stoel et al. (2006) shows for a particular analysed case, the numerical prediction of geosynthetic tension as well as prediction from analitical methods deviate from the measurements. This implies that further research is required to find a better procedure for piled embankment design. Research on numerical analysis of piled embankments needs to be done to obtain a proper design procedure for modelling piled embankments including less but reliable assumptions.

As mentioned before, most of the guidelines suggest to use one strong biaxial geosynthetics for the embankment reinforcement, whereas in practice, it is often common to use three layers of geosynthetics. The reason for this may probably due the need of using lower embankment height, larger pile spacing and lower geosynthetic tensile strength, which leads to economic optimisation. However the adequacy of this practice should be verified. A study by Arwanitaki and Triantafyllidis (2006) from plane strain FE-analyses shows that if more layers of geosynthetics with the same stiffness are applied, the amount of maximum tension in the lowest geosynthetics is not reduced significantly compared to when using one layer of geosynthetics with equivalent stiffness. Thus, further study is needed to clarify this practice.

In contrary to the use of geosynthetics in piled embankment, several 2D numerical studies found that the effect of a geosynthetic layer is not significant. This implies that it is not necessary to use the geosynthetics. This finding needs to be further clarified with good calculation procedure and the 3D numerical analysis. In addition to that, there is a consideration that geosynthetic layer causes a lower arching height, hence the embankment critical height becomes lower. This consideration allows the use of low embankment fill with geosynthetics. Nevertheless, as far as can be found, there is no study specifically showing this fact yet. Therefore, this is another important aspect to be investigated.

In numerical analysis of piled embankments, different soil models have been applied for the numerical analyses, however, there are no studies reported about the influence of soil models to the performance of a piled embankment. Intuitively, there should be a difference in the load transfer mechanism as different soil models use different assumption for the stiffness and yielding behaviour of the soil. For example, the Mohr-Coulomb model, which incorporates one stiffness for the entire soil, will produce different stress field due to arching compared to advanced soil models, which considers stress level dependency. Hence the embankment load transfer should be different as well. To what extent that they are different still needs to be studied.

Furthermore, Kempton et al. (1998) and Noughton et al. (2005) show that a reliable numerical analysis should be done using 3D modeling. However, 3D analysis needs much computer power and is time consuming. 2D axisymmetric analysis for assessing soil arching, load on geosynthetics due to soil arching and settlements can reasonably be used for the calculation. This is reasonable at least for the preliminary design and provided that a reliable correlation between 2D and 3D analysis results is known. Similarly, proper 2D plain strain can be used for assessing the embankment horizontal deformations and load on piles and geosynthetics. Thus, research to find the correlation is required.

In the area where the thickness of soft soil is large such as in large parts of South East Asia, piled embankment design with floating piles is almost a must. Therefore, the design of the pile penetration depth with respect to the soft soil thickness is important concerning the capacity of the piles and serviceability of the piled embankment. As there is no guideline for piled embankments on floating piles, further studies need to be performed to understand the influence of the ratio of pile penetration depth to the soft soil thickness on the soil arching, geosynthetic tension and settlement behaviour of the piled embankment. Moreover, when considering settlement behaviour, consolidation and creep need to be considered.

Poulos (2007) proposes some design charts, which are based on numerical analyses to predict the settlements of piled embankment. The charts are addressed for preliminary design. No detailed insight about the numerical modelling of piled embankments is available and creep is not accounted for. For detailed design, further analysis, especially with numerical method such as finite element is required. Moreover, numerical analysis seems to be most detailed option for settlement prediction, which takes into account long term time effects (consolidation and creep) since there is no guideline for this. Thus, further research is needed to find a proper and reliable procedure of modelling piled embankment using numerical analysis.

Since timber piles can be the most economical and environmental friendly option for piled embankment construction (Van Eekelen and Alexiew, 2007), Therefore, attention should be addressed to these piles. Timber piles can be very flexible, thus generally considered as head-settling piles. Most of the guidelines do not consider these piles, however, this could be easily accounted for using numerical method.

6. Conclusions and Proposed Research Topics

From literature, it is known that design results from different empirical methods of piled embankment are inconsistent and they frequently deviate from the measured data. Besides this, the design practice does not always follow the guidelines consistently. In particular, it can be concluded that first, the design of soil arching needs to be improved and thoroughly verified with field measurements. Secondly, in the design of geosynthetic tension, there is a concern whether the geosynthetic tension should be design as equal to the total of the geosynthetic tension due to soil arching calculation and the load due to the lateral earth pressure of the embankment or only the maximum of the two components. This needs to be verified with measurements and numerical studies.

Another concern regarding of the geosynthetics is that the sufficiency and possible advantages of using several layer of geosynthetics in practice needs further confirmation from research in order to optimise design. Furthermore, the consideration of including the sub soil counter support in the design of geosynthetics needs to be clarified in more detail in consideration of consolidation and creep settlement.

In addition to that, empirical methods do not cover the prediction of short and long term settlements and horizontal deformations of piled embankments, which are very important especially for embankment on floating piles. Thus, it can be stated that there are no guidelines for embankments on floating piles.

Piled embankments are complex soil-structure interaction problems. Numerical methods are considered a powerful tool to reduce the uncertainties and have been used for the piled embankment design. Nevertheless, the design result is not always followed. Further research on numerical analysis of piled embankments is needed to obtain a proper design procedure for modelling piled embankments including less but reliable assumptions.

In particular, research is required to improve calculation procedures, which include or verify several items such as:

- Influence of soil constitutive models on soil arching development in an Embankment.
- Influence of number of geosynthetic layers on the soil arching development
- Determination of geosynthetic tension (Love and Milligan vs. BS 8006)
- Determining of correlation between the results of 2D and 3D analyses of soil arching and differential settlement
- Effects of pile installation process on soil arching development
- Influence of pile penetration depth on soil arching development (in consideration to head-settling and non head-settling piles)
- Influence of long term time effects (consolidation and creep) on the differential settlement and embankment surface settlement
- Influence of possible local failure on the surface settlement of embankment

Following the conclusions above, several research topics are established. The research focuses on the numerical analysis of the design of piled embankments. This is because numerical method can include almost all influencing parameters in the design of piled embankments, which is not possible to account for using analytical methods. Furthermore, it is known that there are also uncertainties in numerical analysis of pile embankments and there are no guidelines for it yet. In order to reduce the uncertainties in the design of piled embankments, the following research topics are proposed:

1. **Influence of Soil Constitutive Models on Soil Arching Development in an Embankment**

   This research evaluates soil arching development and horizontal load distribution of an embankment with different soil constitutive models. The calculation results will be compared to measured data. Embankments on end bearing piles will be considered for this research. The research will be performed with 2D plane strain and axisymmetric FE-analysis.

2. **Influence of Number of Geosynthetic Layers on Soil Arching Development**

   This research is aimed to observe the influence of the number of geosynthetic layers (e.g. 0, 1 and 3 layers) and their distances to the development of arching in the embankment. Geosynthetic tensile forces, embankment critical height and embankment settlements will also be evaluated. In this research, an embankment on end bearing piles is considered. The research will be performed with 2D axisymmetric FE analyses and 3D FE-analyses will be used when necessary to compare some specific results.

3. **Determination of Geosynthetic Tension**

   This research investigates the maximum geosynthetic tension underneath the embankment. It is aimed to resolve the contradiction in the design of geosynthetic tension between the suggestion by Love and Milligan (2003) and BS 8006 especially for embankments, which are higher than 3m and with large pile spacing. In this research an embankment on end bearing piles is considered. The research can be performed with 2D plane strain FE-analyses and 3D FE-analyses to compare some specific results.

4. **Determining of Correlation Between 2D and 3D Analyses of Soil Arching and Differential Settlement**

   This research determines correlations between 2D FE analysis and 3D FE analysis results regarding soil arching and differential settlement. The soil arching will be evaluated from efficacy and the differential settlement will be observed at the pile head level. In this analysis, embankments on end bearing piles are considered.

5. **Effects of Pile Installation Process on Soil Arching Development**

   This research is aimed to investigate the influence of the installation of displacement piles on differential settlement between the pile head and soft soil surface as well as the development of soil arching in the embankment. In addition to that, the rate of improvement of soft soil due to soil compaction from the displacement pile installation will also be studied. This research is addressed to both embankments on end bearing pile as well as on floating piles. Head-settling and non head-settling pile issue is important to be studied in this case. The analyses will be performed using 2D axisymmetric FE-analysis and 3D FE-analyses to compare some specific results.

6. **Influence of Pile Penetration Depth on Soil Arching Development**

   This research evaluates the influence of pile penetration depth to the differential settlement and soil arching development. In this research topic, an embankment on floating piles is considered. Head-settling and non head-settling piles issue is also important in this research. The research will be performed with 2D axisymmetric and 3D numerical analyses.

7. **The Influence of Consolidation and Creep on the Differential Settlement and Embankment Surface Settlement**

   This research investigates the influence of consolidation and creep on the differential settlement of the pile head relative to the soft soil in between the piles. Head-settling and non head-settling piles is an important issue in this research topic. The behaviour of embankment surface settlement and counter support from soft soil will be studied to understand the long term tensile strength requirement of the geosynthetics. Embankments on floating piles are considered. The research will be done with 2D axisymmetric FE analysis.

8. **Influence of Possible Local Failure on the Surface Settlement of Embankment**

   This research topic investigates the consequence of possible local failure of one pile or geosynthetic yielding to the surface settlements of the embankment and to suggest possible measures for mending the failure. The research can be done with 2D plane strain and axisymmetric FE analysis.

8. References

Alexiew, D. (2005). Piled Embankments in soft soils for railroads: Methods and Significant Case Studies. Proc. Of the 6th International Conference on Ground Improvement Techniques, July, Coimbra, Portugal, pp. 87-94.

Arwanitaki, A. and Triantafyllidis, T. (2006). Mehrlagig mit Geogittern bewehrte Erdkörper über pfahlartigen Gründungselementen. Bautechnik, Vol. 83, No.10, pp. 695-707.

BS8006 (1995). Code of Practice for Strengthened/Reinforced Soils and Other Fills. British Standard Institution.

Carlsson, B. (1987). Reinforced soil, principles for calculation. Terratema AB, Linköping.

Clarke, N. W. B. (1968). Buried pipelines - A manual of structural design and installation. Maclaren and Sons, London.

Cortlever, N. G. and Gutter, H. H. (2006). Design of double track railway Bidor-Rawang on AuGeo piling system according to BS8006 and PLAXIS numerical analysis. Cofra B.V., Amsterdam, The Netherlands.

Gangakhedar, R. (2004). Geosynthetic reinforced pile-supported embankments. Master thesis, University of Florida.

Han, J. (2003). Geosynthetic-reinforced and pile-supported embankments. Geo-WCS, 2003, pp. 308-317

Huang, J., Collin, J.G., Han, J. (2005). 3D numerical modeling for a eosyntheticreinforced pile-supported embankment–displacement analysis. Proc. of the16th Inter. Conf. on Soil Mech. and Geotechnical Eng., Osaka, Japan, Sept. 12-16, 913-916.

Han, J. and Gabr, M.A. (2002). A numerical study of load transfer mechanisms in geosynthetic reinforced and pile supported embankments over soft soil. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 128(1), pp. 44-53

Heitz, C. (2006). Bodengewölbe unter ruhender und nichtruhender Belastung bei Berücksichtigung von Bewehrungseinlagen aus Geogittern. Universität Kassel

Hewlett, W. J. and Randolph, M. F. (1988). Analysis of piled embankments. Ground Engineering, 21(3), pp.12-18.

Jenck, O., Dias, D., Kastner, R. (2006). Three-dimensional finite difference numerical modelling of a piled embankment section. SYMPOSIUM Rigid Inclusions in Difficult Subsoil Conditions, Mexico City , 11-12 May

Jenck, O., Dias, D., Kastner, R. (2007). Two-dimensional physical and numerical modeling of a pile-supported earth platform over soft soil. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 133, No. 3, pp. 295-305

Jones, C. J. F. P., Lawson, C. R., and Ayres, D. J. (1990). Geotextile reinforced piled embankments. Proceedings, 4th International Conference on Geotextiles, Geomembranes, and Related Products, Vol. 1, G. Den Hoedt (ed), The Hague, May - June 1990, pp. 155-160.

Kempfert, H. - G., Stadel, M. and Zaeske, D. (1997). Design of geosynthetic-reinforced bearing layers over piles. Bautechnik, Vol. 74, No. 12, pp. 818-825.

Kempfert, H.-G., Gobel, C., Alexiew, D. and Heitz, C. (2004). German recommendations for reinforced embankments on pile-similar elements. EuroGeo3 - Third European Geosynthetics Conference, Geotechnical Engineering with Geosynthetics, pp. 279-284.

Kempton, G., Russell, D., Pierpoint, N. D., and Jones, C. J. F. P. (1998). Two- and three-dimensional numerical analysis of the performance of piled embankment. Proceedings, 6th International Conference on Geosynthetics, pp. 767-772.

Love, J., and Milligan, G. (2003). Design methods for basally reinforced pile-supported embankments over soft ground. Ground Engineering, Vol. 36, No. 3, March, pp. 39-43.

Marston, A. and Anderson, A. O. (1913). The theory of loads on pipes in ditches and tests of cement and clay drain tile and sewer pip. Bulletin No. 31, Iowa Engineering Experiment Station, Ames, Iowa.

McKelvey III, J. A. (1994). The anatomy of soil arching. Geotextiles and Geomembranes, 13, 317-329.

McNulty, J. W. (1965). An Experimental study of arching in sand." Technical Report No. I-674, U.S. Army Engineer Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi, 170.

Miki, H. and Nozu, M. (2004). Design and Numerical Analysis of Road Embankment with Low Improvement Ratio Deep Mixing Method, Geo-Trans 2004, ASCE, No.126, Vol.2, pp.1395-1402.

Noughton, P. J., Kempton, G. T. (2005). Comparison of analytical and numerical analysis design methods for piled embankment. Proc. of Geo-Frontiers, Austin, USA.

Pleomteux, C. and Porbaha, A. (2004). CMC Foundation System for Embankment Support – A Case History, Geo Support.

Poulos, H. G. (2007). Design Charts for Piles Supporting Embankments n Soft Clay. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 133, No. 5, pp. 493-501.

Russell, D. and Pierpoint N. (1997). An assessment of design methods for piled embankments. Ground Engineering, November, pp. 39-44.

Russell, D. and Pierpoint N. (1998). An assessment of design methods for piled embankments - author's response to discussion. Ground Engineering, March, pp. 34-36.

Russell, D., Naughton, P.J. and G. Kempton, G. (2003). A new design procedure for piled embankments. Proceedings of the 56th Canadian Geotechnical Conference and 2003 NAGS Conference, Vol. 1, Winnipeg, MB, pp. 858-865.

Satibi et al. (2007) FE-simulation of installation and loading of a tube-installed pile, Institutsbericht no.29, IGS, Universität Stuttgart.

Schmidt, H. H. (2004). Lastkonzentration über steifen Gründungselementen infolge von Überschüttungen, Interne Arbeit, Universität Stuttgart: Institut für Geotechnik.

Scarino, H. J. (2003). Reappraisal of Marston's formula. Journal of Transportation Engineering, ASCE, December, pp 703-712.

Sovulj, D. (2005). Lastkonzentration über steifen Gründungselementen infolge von Überschüttungen. Diplomarbeit 158, Institut Für Geotechnik, Stuttgart.

Suleiman, M., Pham, H., and White, D. (2003). Numerical analyses of geosynthetic-reinforced rammed aggregate pier-supported embankments. Report No. ISU-ERI-03598, Department of Civil, Construction and Environmental Engineering, Iowa State University

Svan∅, G., Ilstad, T., Eiksund, G. and Want, A. (2000). Alternative calculation principles for design of piled embankment with base reinforcement. Proc. 4th Inter. Conf. On Ground Improvement Geosystems, Helsinki.

Terzaghi, K. (1943). Theoretical Soil Mechanics, John Wiley & Sons, New York, 66.

Terzaghi, K. (1936). Stress distribution in dry and in saturated sand above a yielding trap-door. Proceeding of the International Conference on Soil Mechanics and Foundation Engineering Bd.1. Cambrigde, 307-311.

Van der Stoel, A.E.C., Klaver, J.M., Balder A.T., de Lange, A.P. (2006). Numerical design, installation and monitoring of a load transfer platform (LTP) for a railway embankment near Rotterdam. NUMGE, Schweiger (ed.), Taylor & Francis Group, London.

Van Eekelen, S. J. M. and Alexiew, D (2007) Die Kyoto-Straße: Ein Großversuch, Messungen und Berechnungen, CUR Publication.