UDC 539.3

EVALUTION ON STRESS DISTRIBUTION, DEFORMATION RATE IN EMBANKMENT AND SOFT SOIL REINFORCED CONCRETE PILE COMBINED GEOTEXTILE BELOW THE EMBANKMENTS IN GEOLOGICAL CONDITIONS MEKONG DELTA

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The main content of the paper is evalution stress distribution, deformation rate in embankment and soft soil reinforced concrete pile combined geotextile below the embankments in geological conditions in Mekong delta by finite element method to Geotechnique-designer have to notice the correlation of rational pile-distance and embankment-depth when design weak foundation.
Keywords: Geosynthetic reinforced pile, soft soil, pile embankment, foundation, FEM.

1. Introduction

With the rapid growth of the economy and the trend of industrialization and mordenization of the country, the demand of developing buildings, factories and other infrastructures in Vietnam increases, especially in the Mekong Delta area. Because of the geological structure property of Mekong Delta is mainly soft soil, the foundation depth can vary from 30 to 40m. To meet the demand of infrastructure development, there some studies and researches done recently on this geological structure.

The divergent subsidence due to causeway, path way to the bridge, storage has caused difficulty for the transportation and facility in some areas in Hochiminh city. For instance, according to Hochiminh city Transportation Department, the sunsidence on Nguyen Huu Canh street varies from 0.5 to 1m. There are some proposal solutions to solve this issue such as concrete piles, sand piles, reinforced concrete, soil&cement mixture to reduce load, etc… These solutions takes a lot of time and are not efficient. Recent years, there is a solution for the foundation called “The embankment on the pile conbined with geotextile”. Hopefully this solution will solve the issue.

The evaluation and analysis on stress distribution, ground deformation and reinforced concrete and geotextile treatment on soft soil is extremely essential to find the new solution to improve the quality of foundation in Mekong Delta area.

2. Theoretical basis

2.1. Theory of soil arching

According Terzaghi (1943) arching effects have been described. Arching effects base on his experiment on the trap-door effects as shown in Fig. 1.

Stress Distribution Equation:

$$(\sigma_z + d\sigma_z)^* S - \sigma_z^* S + 2\tau_{xz} dz - dG = 0. \quad (1)$$

With σ_z is vertical effective Stress (Z direction), τ_{xz} is shear stress on xz plane, S is width of trap-door, G is weight of soil on trap-door, γ is the soil unit weight.

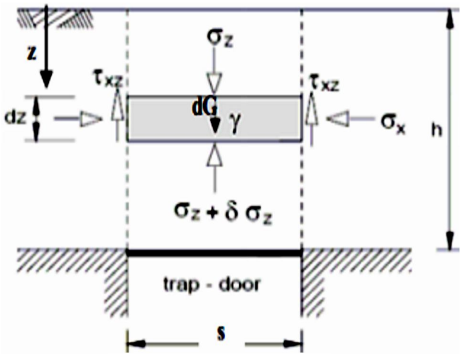


Fig. 1. Description of soil Arching analysis with Terzaghi's method

The equivalent equation:

$$d\sigma_z^* S = \gamma^* S dz - 2\tau_{xz} dz. \quad (2)$$

According to Mohr-Coulomb, the Shear Stress at failure can be expressed as:

$$\tau_{xz} = C' + \sigma_x \tan\varphi'. \quad (3)$$

With C' and φ' are the effective cohesion and friction angle of the soil. The effective horiziontal stress as afunction of verticle effective stress is $\sigma_x = \sigma_z^* K$, Terzaghi determined that K=1 based on his experimental results.

The equation is written as:

$$d\sigma_z^* S = \gamma^* S dz - 2(C' + \sigma_z K \tan\varphi') dz. \quad (4)$$

Dividing both side of Equation (4) with σ_z and s:

$$\left(\frac{d\sigma_z}{\sigma_z}\right) = \frac{\gamma}{\sigma_z} dz - \frac{2C}{s^* \sigma_z} dz - \frac{2K^* \tan\varphi}{s} dz. \quad (5)$$

The solution for the differential equation is as follows

$$\sigma_z = \frac{S^*(\gamma - 2C'/S)}{2^* K^* \tan\varphi'} \left\{1 - e^{-2K \tan\varphi' \frac{z}{s}}\right\} + p^* e^{-2K \tan\varphi' \frac{z}{s}}. \quad (6)$$

According to the result found by Terzaghi, K=1. Solve equation (6) gives an exponentially increasing vertical effective stress within the embankment fill between the two rigid foundations. Comparison between effective vertical stress distribution with the linearly increasing geostatic vertical stress is shown in Fig 2. Due to arching, the vertical stress acting on the ground surface below the embankment is much lower than the geostatic vertical stress.

Arching is difined by Mc. Nulty (1965) as "The ability of a material to transfer from one location to another in response to a relative displacement between the location. A system of shear stresses is the mechanism by which the loads are transfer".

The Nordic Guiline method helps analyze arching in soil is suggested by Carlsson (1987), this method shows the angle of arching is 30 degrees refer to Fig. 5.

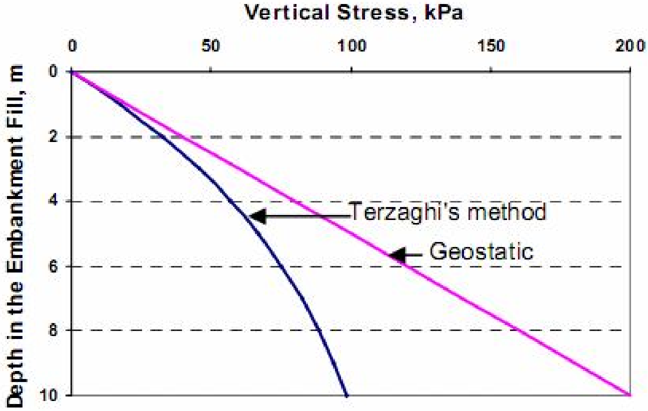


Fig. 2. Typical vertical stress distribution of embankment fill between trap-door of Terzaghi

Weight of the soil is calculated in 2D as below:

$$W = \frac{(b-a)^2}{4\tan 15^\circ}\gamma, \tag{7}$$

with a is the width of the pile, b is the distance between the centre of 2 piles, γ is unit weight of the embankment.

Svant et al. (2000) suggested the soil weight formula in 3D

$$W = \frac{\gamma}{2a}\left\{b^2 H - \frac{1}{6\tan\beta}\left[(a + H\tan\beta)^3 - a^3\right]\right\}. \tag{8}$$

With a is the width of the pile, b is the distance between the centre of 2 piles, γ is the soil unit weight, H is the height of soil layer.

Development by Jones et al. (1990) based on the past study by Marston and Anderson (1913) about the peak of the spherical dome between piles.

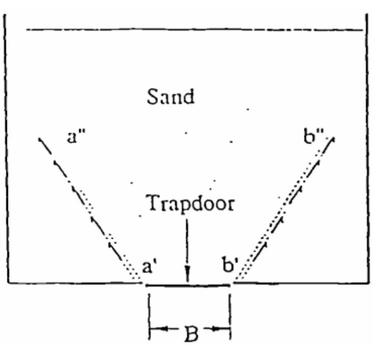


Fig. 3. The shear stress path when trap-door min displacement of Terzaghi

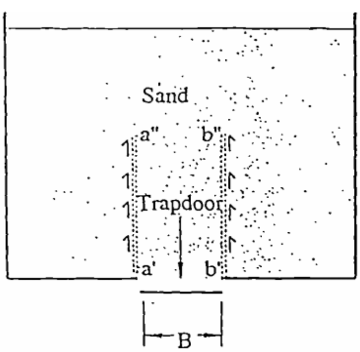
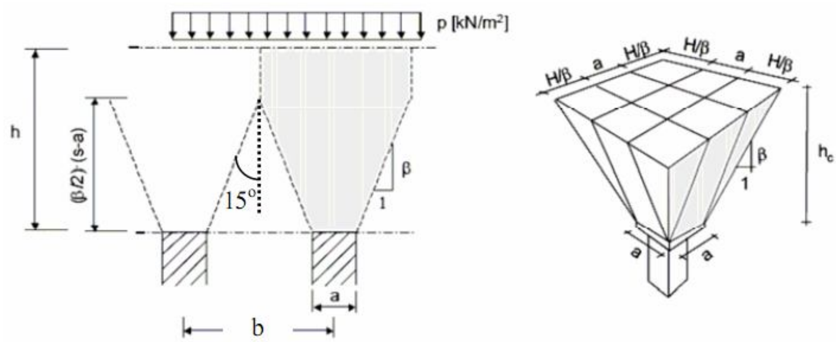


Fig. 4. The shear stress path when trap - door max displacement of Terzaghi



(a) (b)

Fig. 5. Soil wedge model (a) 2D; (b) 3D defined by Carlsson

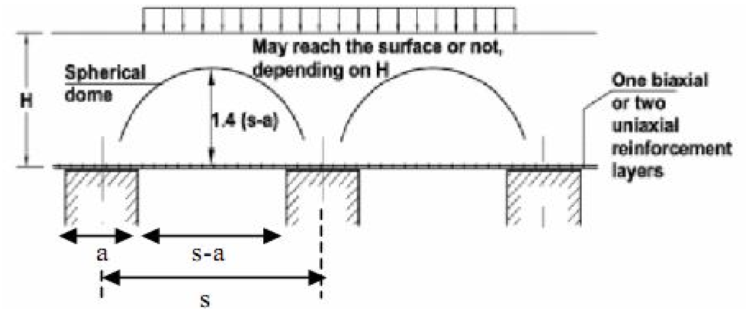


Fig. 6. Hemispheric in BS 8006

Analyze the spherical dome based on the ratio between pressure on the pile and vertical stress on the soft soil layer, P_c'/σ_v

$$P_c' = \delta_v' \left[\frac{C_c a}{H} \right]^2 . \tag{9}$$

With: C_c is soil arching coefficient (C_c =1.95(H/a) - 0.18 for end-bearing pile, C_c = 1.5(H/a) - 0.07 for friction and other pile), a is the size of the pile caps, H is the height of the embankment.

2.2. Load Transfer

McNulty (1965) and Kempton (1998) The ratio of the vertical stress on top of the cap:

$$\rho = \frac{P_b}{\gamma H + q_0} . \tag{10}$$

Where: p_p is average vertical pressure above geosynthetic, q_0 is uniform surcharge on the embankment, T is tension on geotextile, $\rho = 0$: Represents the Complete soil arching / $\rho = 1$: Represents no soil arching, γ is the soil unit weight.

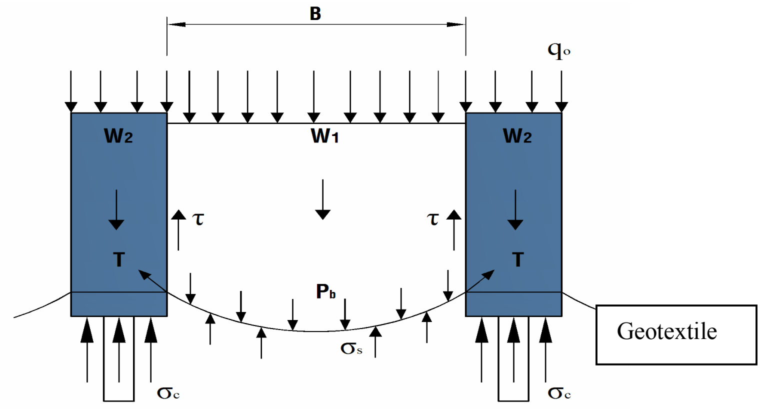


Fig. 7. Load transfer (cited in Li, 2002)

Han (2003) The ratio of the vertical stress on top of the cap:

$$n = \frac{\sigma_c}{\sigma_s}. \quad (11)$$

With σ_c is vertical stress on pile, σ_s is vertical stress between piles.

Schimidt (2004) The ratio of the vertical stress on top of the cap:

$$\mathrm{LKF} = \frac{Q}{Q_s} = \frac{\sigma_c^* A_c}{\gamma^* H^* A_c} \quad (12)$$

Γ is the soil unit weight, H is height of embankment, A_c is Cross sectional area of pile.

2.3. Factor that determines arching

Ratio that determines arching

CSR is the column stress ratio

SRR is the stress reduction ratio

N is the ratio of the vertical stress on top of the cap and E The piled embankment efficacy.

$$\mathrm{CSR} = \frac{\sigma_c}{\sigma} = \frac{\sigma_c}{(\gamma H + q)}, \quad (13)$$

$$\mathrm{SRR} = \frac{\sigma_s}{\sigma} = \frac{\sigma_s}{(\gamma H + q)}, \quad (14)$$

$$n = \frac{\sigma_c}{\sigma_s}, \quad (15)$$

$$E = \frac{\sigma_c x a_s}{\sigma}. \quad (16)$$

With: γ is the soil unit weight, H is height of soil layer, q is surcharge load,

$$a_s = \frac{A_c}{A_c + A_s},$$

A_c is pile cross sectional area, A_s is area of the soil associated with the column.

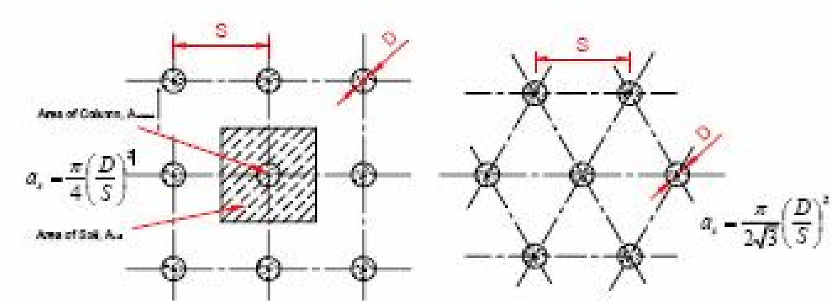


Fig. 8. Area replacement ratio

According to BS8006:1995 and some researchers, SRR is calculated as Table 1.

Table 1

The stress reduction ratio

No.		The stress reduction ratio	No. Equa
1	BS 8006 1995	$SRR = \frac{2*S*(\gamma H+q)(S-a)}{(S^2-a^2)^2*\gamma H}*\left[S^2-a^2\left(\frac{P'_c}{\gamma H}\right)\right]$ With H ≤ 1.4(S-a) $SRR = \frac{2.8*S}{(S+a)^2*H}*\left[S^2-a^2\left(\frac{P'_c}{\gamma H}\right)\right]$ With H > 1.4(S-a) With $\frac{P'_c}{\gamma H} = \left(\frac{C_c a}{H}\right)^2$	(17) (18) (19)
2	Terzaghi	$SRR = \frac{(S^2-a^2)}{4*H*a*K*\tan\phi}\left\{1-e^{\frac{-4HaK\tan\phi}{s^2-a^2}}\right\}$	(20)
3	Randolph 1988	$SRR = \frac{1}{\left[\frac{2K_p}{K_p+1}\right]\left[\left(1-\frac{a}{s}\right)^{(1-K_p)}-\left(1-\frac{a}{s}\right)\left(1+\frac{a}{s}K_p\right)\right]+\left(1-\frac{a^2}{s^2}\right)}$	(21)
4	Guido	$SRR = \frac{(s-a)}{3\sqrt{2}.H}$	(22)
5	Low 1994	$SRR = \frac{(K_p-1)(1-\delta)S}{2H*(K_p-2)}+(1-\delta)^{(K_p-1)}\left[1-\frac{S}{2H}-\frac{S}{2H(K_p-2)}\right]$	(23)
6	Carlsson	$SRR = \frac{\text{s - a}}{4.H.\tan 15^0}$	(24)
7	Kivilo 1998	$CSR = \frac{1}{a_s+\frac{E_{soil}}{E_{col}}(1-a_s)}$ $SRR = \frac{E_{soil}}{E_{col}a_s+E_{soil}(1-a_s)}$	(25) (26)

In which: H is height of the embankment, q is external load, s is distance between pile center, a is Area replacement ratio, C_c is arching coefficient (C_c =1.95(H/a) - 0.18 for end-bearing pile, C_c = 1.5(H/a) - 0.07 for friction and other pile), b' is angle of friction of the embankment fill, K is coefficient of later earth pressure (K = 1)

$$K = \frac{1+\sin\varphi}{1-\sin\varphi}$$

is Rankine coefficient of passive earth pressure, E_{col} is Modulus of elasticity of the column, E_{soil} is Modulus of elasticity of the unstabilized soil surrounding the column.

3. The embankment on the pile conbined with geotextile

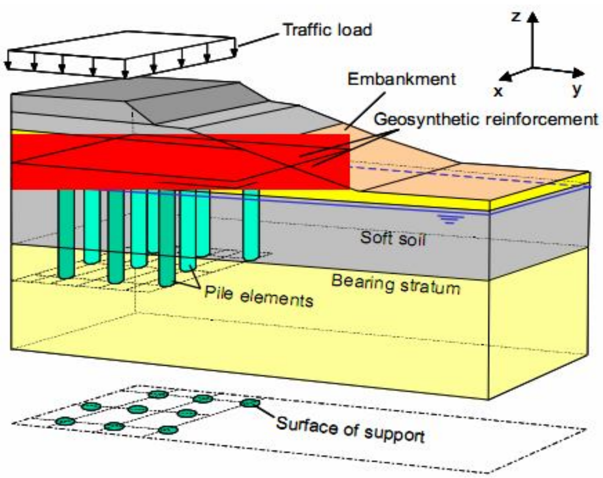


Fig. 9. Geosynthetic reinforced pile supported embankment [4],[5]

Arching in embankment.

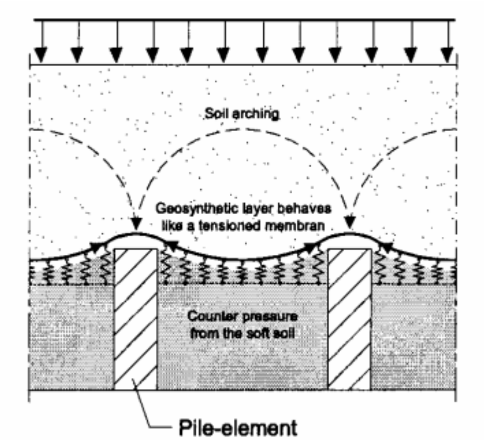


Fig. 10. Arching in embankment [4], [5]

3.1. Geosynthetic reinforcement

3.1.1. BS 8006 (1995)

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

$$T_{rp} = \frac{W_T(s-a)}{2a}\sqrt{1+\frac{1}{6\varepsilon}}\,, \tag{27}$$

where T_{rp} is the tensile force per meter geosynthetics, W_T is distributed vertical load acting on the geosynthetic between the piles, ε is the strain in the geosynthetics (%), a is the pile cap size and s is the center-to-center spacing.

3.1.2. Zaeske (2001) and Kempfer (2002)

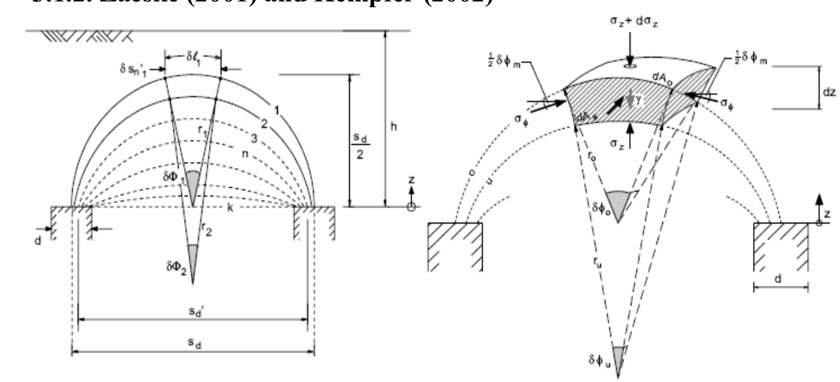


Fig 11. Multi aching model [2]

The equation is developed:

$$-\sigma_z \mathrm{d}A_u + (\sigma_z + \mathrm{d}\sigma_z)\mathrm{d}A_0 - 4\sigma_\varphi \mathrm{d}A_s \sin\left(\frac{\delta_{\varphi m}}{2}\right) + \gamma \mathrm{d}V = 0. \tag{28}$$

Where:

$$\mathrm{d}A_u = (r\delta_\varphi)^2, \tag{29}$$

$$\mathrm{d}A_0 = (r+\mathrm{d}r)^2\cdot(\delta_\varphi+\mathrm{d}\delta_\varphi)^2 = 2\mathrm{d}\delta_\varphi\cdot r^2\cdot\delta_\varphi + 2\mathrm{d}r\cdot r\cdot\delta_\varphi{}^2 + r^2\cdot\delta_\varphi{}^2, \tag{30}$$

$$\mathrm{d}A_s = (r+\tfrac{1}{2}\mathrm{d}r)\cdot(\delta_\varphi+\tfrac{1}{2}\mathrm{d}\delta_\varphi)\cdot\mathrm{d}z = \mathrm{d}z\cdot r\cdot\delta_\varphi, \tag{31}$$

$$\mathrm{d}V = (r+\frac{1}{2}\mathrm{d}r)^2\cdot(\delta_\varphi+\frac{1}{2}\mathrm{d}\delta_\varphi)^2\cdot\mathrm{d}z = \mathrm{d}z\cdot r^2\cdot\mathrm{d}\delta_\varphi{}^2. \tag{32}$$

The equation is developed the tension force in the geosynthetic:

$$\frac{\mathrm{d}^2 z}{\mathrm{d}x^2} = \frac{q_2}{H} + \frac{C-x}{H}, \tag{33}$$

$$H = \frac{2\cdot\int_0^i\sqrt{1+(z_w^1)^2}\cdot\mathrm{d}x + 2\cdot\int_i^j\sqrt{1+(z_p^1)^2}\cdot\mathrm{d}x - l_0}{2\cdot\int_0^i\left(1+(z_w^1)^2\right)\cdot\mathrm{d}x + 2\cdot\int_i^j\left(1+(z_p^1)^2\right)\cdot\mathrm{d}x}. \tag{34}$$

Where

$$z_W(x) = A_{1,W} \cdot e^{\alpha_W \cdot x} + A_{2,W} \cdot e^{-\alpha_W \cdot x} - \frac{\beta_W}{\alpha_W^2}, \quad 0 \le x \le i,$$
$$z'_W(x) = \alpha_W \cdot \left(A_{1,W} \cdot e^{\alpha_W \cdot x} - A_{2,W} \cdot e^{-\alpha_W \cdot x} \right). \tag{35}$$

The tensile force per meter geosynthetic:

$$S(x) = \varepsilon(x) / J = H \cdot \sqrt{1 + z'^2(x)}. \tag{36}$$

3.2. Result of model

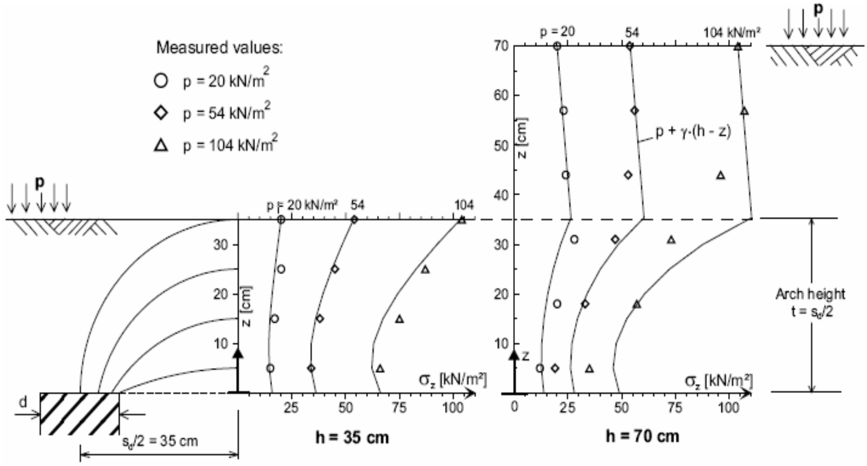


Fig. 12. The result of model Zaeske (2001) [2]

According to the experiment by Zaeske (2001), it is proven that the ratio of the arching in soil with the real dimension is 1/3. This includes 4 piles in soft soil. On the top of each pile is covered by geotextile with the earth pressure cells.

The experiment result is recorded as below:

Case 1: distance between 2 piles s =70cm, sand layer's thickness of 35cm, applied loads of 20kN/m^2, 5420kN/m^2, 10420kN/m^2. Vertical stress is measured at distance of 5cm, 15cm, 25cm between and above the top of 2 piles.

Table 2

Case 1

p (kPa)	**σ** (kN/m^2)	***h*** (cm)
20	15-16-19	5-15-25
54	33-42-45	5-15-25
104	65-75-87	5-15-25

Case 2: distance between 2 piles s = 70cm. sand layer's thickness of 70cm, applied loads of 20 kPa, 54 kPa, 104 kPa. Vertical stress is measured at distance of 5cm, 20cm, 30cm, 45cm, 55cm between and above the top of 2 piles.

Table 3

Case 2

p (kPa)	**σ** (kN/m^2)	***h*** (cm)
20	15-20-29-25-22	5-20-30-45-55
54	20-33-46-54-57	5-20-30-45-55
104	35-57-73-95-107	5-20-30-45-55

4. Design of ingenieurgesellschaft geotecgnik walz (igw) used for Hung Loi metro in Can Tho City, Vietnam

The model uses cylindrical piles with diameter *D* = 300mm, spacing between piles *S*=4000mm, reinforced concrete dimension of 1500x1500x300mm, above is geotextile with the height of 500mm for big sand particles. Concrete layer 10x20 B.15 thickness of 250mm, rock layer 0x40mm thickness of 350mm.

The model is ilustrated in Fig.13.

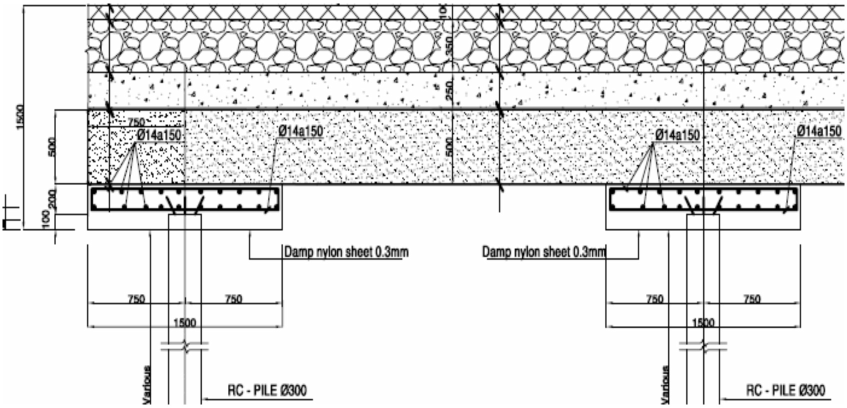


Fig. 13. Design of (IGW) used for Hung Loi Metro in Can Tho City

Deformation of the structure after completion of project in Fig. 14.

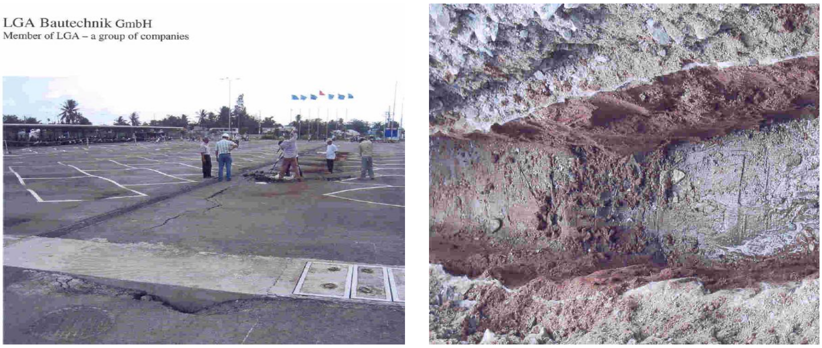


Fig. 14. Differential settlement of Hung Loi Metro project

5. The development of the new model

Redesign the model using reinforced concrete piles with B.20, cross section area of 300x300mm, spacing between piles varies from 1.0m, 1.5m, 2.0m, 2.5m, using Mac 40 geotextile to put on the top of each pile. Sand layer is 1m high, reinforced concrete thickness of 150mm. Using Plaxis 3D Tunnel and Mohr-Coulomb to model with the following parameters.

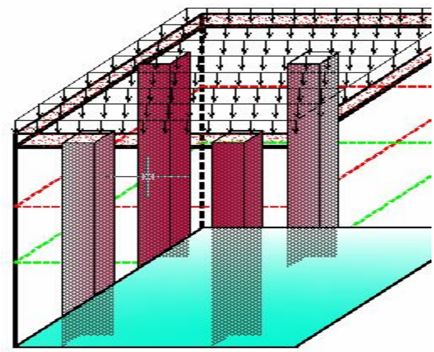


Fig. 15. The development of the new model to repair Hung Loi Metro in Can Tho city

Table 4

Properties of soil layers

Properties	Index	Layer 1	Layer 2	Layer 3	Sand layer	Unit
		Soft soil	stiff clay	sand	Sand embank ment	
Horizontal coefficient ratio	k_x	0.214*10-6	1.2*10-4	2*10-2	3*10-2	cm/s
Vertical coefficient ratio	k_y	0.12*10-6	0.6*10-4	1*10-2	1*10-2	cm/s
Modulus of elasticity of the unstabilized soil surrounding	E_{oed}	1252	14900	28860	30000	kN/m^2
Poisson ratio	ν	0.35	0.33	0.3	0.3	-
Cohesion	C’	8	71	1	1	kN/m^2
Angle of friction of soil	φ’	$18^0$16’	$26^0$58’	30^0	30^0	degree

Table 5

Properties of reinforced concrete pile

Properties	Index		Unit
Modulus of elasticity reinforced concrete	E	$2.9*10^7$	kN/m^2
Area of section	A	0.3*0.3	m2
Poision ratio	ν	0.15	-
Base thickness	h	0.15	m

5.1. The analysis of model used Plaxis 3d Tunnel software

5.1.1. Pile subsidence

To create the spherical dome, there should not be any subsidence on the piples, the limit subsidence is S<= 10mm

5.1.2. The effect of geotextile

Geotextile with high expansion, there should not be any damange on geotextile under load.

5.1.3. Sand layer

Sand particles are big with *c'* = 1kN/m2, φ*'* = 30 degrees, the height of sand layer should be corresponding to the distance between piles.

5.1.4. Reinforced concrete layer

There needs to be reinforced concrete layer for the load distribution to avoid stress concentration on critical points.

5.2. The analysis result of the model

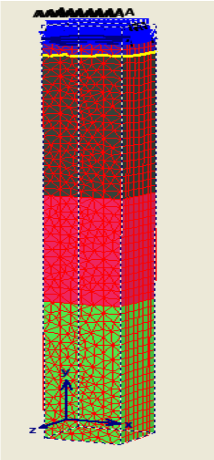


Fig. 16. Model in Plaxis

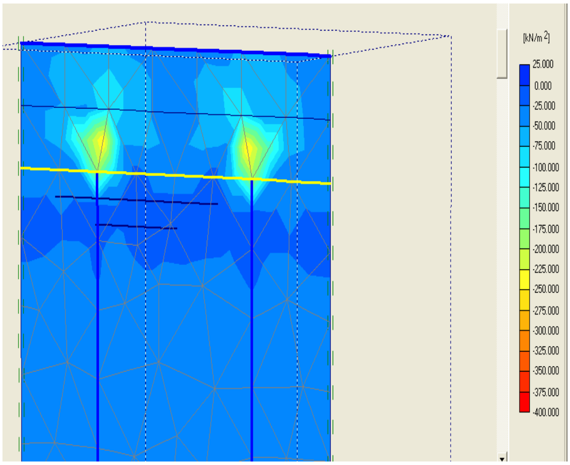


Fig. 17. The vertical stress on top of the top pile

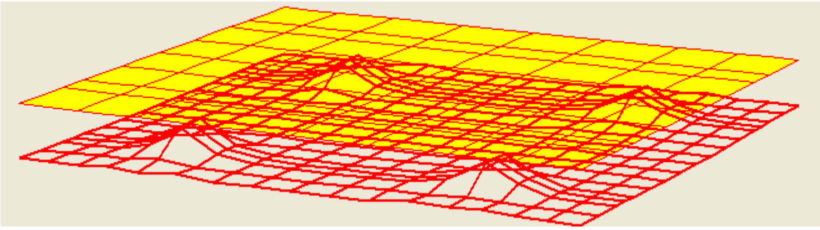


Fig. 18. The tensile force per meter geosynthetic

5.2.1. Stress Distribution

Below are the graphs of the relationship between stress distribution and the pile spacing.

With the pile spacing S = 1m and S = 1.5m, the vertical stress value at the top of the pile is maximum, in the higher up than the top of the pile, the vertical stress tends to decrease and distributed equally near the armoured concrete slab.

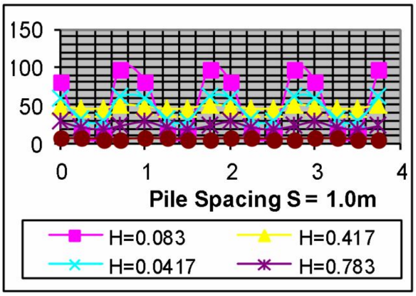


Fig. 19. The vertical stress on top of the top pile with S = 1.0m

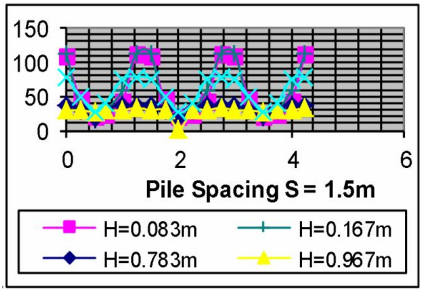


Fig. 20. The vertical stress on top of the top pile with S = 1.5m

When the pile spacing is farther S = 2,0m and S =2,5m, the maximum vertical stress value at the top of the pile is 1.5 times higher than the pile spacing S = 2.0m and S = 2.5m, in the higher up than the pile head, the vertical stress tends to decrease but not distributed equally near the armoured concrete slab.

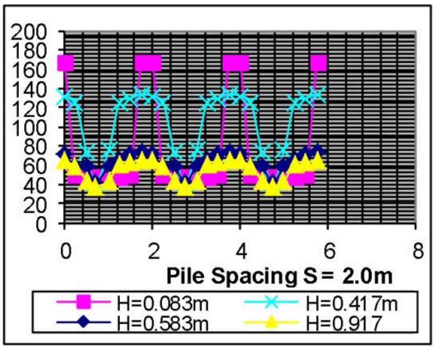


Fig. 21. The vertical stress on top of the top pile with S = 2.0m

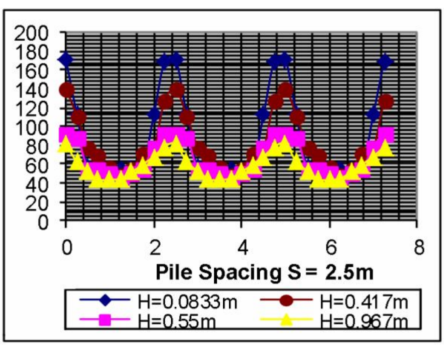


Fig. 22. The vertical stress on top of the top pile with S = 2.5m

5.2.2. The vertical stress on top of the top pile ratio

The vertical stress on top of the top pile ratio $n = \sigma_c/\sigma_s$

- Spacing between piles and hight embankment S=1m, H = 1m

When S = 1m, H = 1m. The stress concentration factor at the top of the pile is n = 6.8, in the higher up than the pile head, the vertical stress tends to decrease and distributed equally near the armoured concrete slab. The stress concentration factor n=1.12.

- Spacing between piles and spacing between pile S=1.5m, H=1m.

When S = 1.5m, H = 1m. The stress concentration factor at the top of the pile is decreased n = 4.93, in the higher up than the pile head, the

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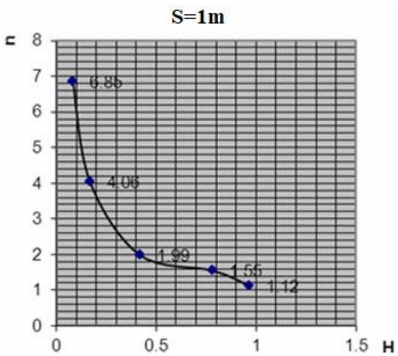
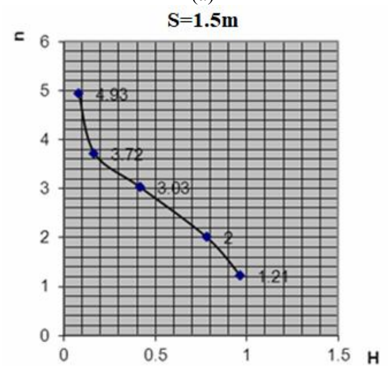
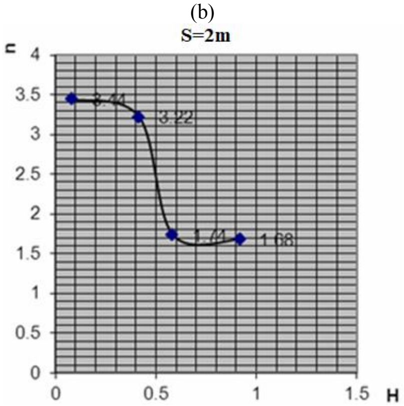


(a)



(b)



(c)

Fig. 23. Relation between the vertical stress on top of the top pile ratio and hight embankment with spacing between piles is 1m (a), 1.5m (b), 2.0m (c)

vertical stress tends to decrease and distributed equally near the armoured concrete slab. The stress concentration factor n=1.12.

- Spacing between piles and spacing between pile S=2m, H = 1m.

When S = 2m, H = 1m. The stress concentration factor at the top of the pile is decreased n = 3.44, in the higher up than the pile head, the vertical stress tends to decrease and not distributed equally near the armoured concrete slab. The stress concentration factor n=1.68.

6. Conclusion and recommendation

6.1. Conclusion

- The coefficient of stress concentration n or the inverse n* will depend on the distance between piles. The further the distance is, the more n decreases and the more n* increases.

- The height $h_{dap} \geq S$ will make the spherical dome become clearer. When $h_{dap} \geq S/2$ then hg=S/2. When $h_{dap} < S/2$ then arching height hg= h_{dap}.

- When the height cao $h_{dap} < S/2$ the deformation is not uniform, and the other way around. When choosing the sand layer, we should choose $h_{dap} > S/2$ and depend on the distance between the piles.

6.2. Recommendation

- Structure of the project is not reasonable. We should choose Structure of the project bearing capacity distribution.

- When applying the new model, notice that the distance between piles and the height should be carefully considered to improve and increase the efficiency of the arching..

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EVALUTION ON STRESS DISTRIBUTION, DEFORMATION RATE IN EMBANKMENT AND SOFT SOIL REINFORCED CONCRETE PILE COMBINED GEOTEXTILE BELOW THE EMBANKMENTS IN GEOLOGICAL CONDITIONS MEKONG DELTA

With the rapid growth of the economy and the trend of industrialization and mordenization of the country, the demand of developing buildings, factories and other infrastructures in Vietnam increases, especially in the Mekong Delta area. Because of the geological structure property of Mekong Delta is mainly soft soil, the foundation depth can vary from 30 to 40m. To meet the demand of infrastructure development, there some studies and researches done recently on this geological structure.

The evaluation and analysis on stress distribution, ground deformation and reinforced concrete and geotextile treatment on soft soil is extremely essential to find the new solution to improve the quality of foundation in Mekong Delta area. The main content of the paper is evalution stress distribution, deformation rate in embankment and soft soil reinforced concrete pile combined geotextile below the embankments in geological conditions in Mekong delta by finite element method to Geotechnique-designer have to notice the correlation of rational pile-distance and embankment-depth when design weak foundation.

Keywords: Geosynthetic reinforced pile, soft soil, pile embankment, foundation, FEM.

Нгуен Ань Туан, Нгуен Туан Фыонг

ОЦЕНКА РАСПРЕДЕЛЕНИЯ НАПРЯЖЕНИЙ, СКОРОСТИ ДЕФОРМАЦИИ НА НАСЫПИ И ГЕОТЕКСТИЛЕ КОМБИНИРОВАННОГО БЕТОНА ИЗ МЯГКОГО ГРУНТА В ГЕОЛОГИЧЕСКИХ УСЛОВИЯХ ДЕЛЬТЫ РЕКИ МЕКОНГ

С быстрым ростом экономики и тенденцией к индустриализации и модернизации страны, спрос на строительство зданий, фабрик и других видов инфраструктур во Вьетнаме увеличивается, особенно в области дельты Меконга. Из-за особенностей геологической структуры в дельте реки Меконга преобладают мягкие грунты. Глубина заложения фундаментов сооружений может варьироваться от 30 до 40 метров. Чтобы удовлетворить потребность в развитии инфраструктуры, недавно были проведены некоторые исследования этой геологической структуры.

Были выполнены оценка и анализ распределения напряжений, деформации грунта, железобетонных конструкций и геоткани в мягком грунте. Такая оценка чрезвычайно важна для поиска новых подходов, направленных на улучшение характеристик фундаментов сооружений в районе дельты реки Меконг. Основным методом исследования является метод конечных элементов.

Ключевые слова: армированный геосинтетический материал, мягкий грунт, насыпной грунт, фундамент, метод конечных элементов.

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Fig. 23. Ref. 14.

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