EVALUATION 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 evaluation 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 modernization 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 subsidence 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 take a lot of time and are not efficient. Recent years, there is a solution for the foundation called “The embankment on the pile combined 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:

\[(\sigma_z + d\sigma_z)^* S - \sigma_z^* S + 2\tau_{xz} dz - dG = 0.\]  \hspace{1cm} (1)

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

The equivalent equation:

\[d\sigma_z^* S = \gamma^* S dz - 2\tau_{xz} dz.\]  \hspace{1cm} (2)

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

\[\tau_{xz} = C' + \sigma_x \tan \phi'.\]  \hspace{1cm} (3)

With \(C'\) and \(\phi'\) are the effective cohesion and friction angle of the soil. The effective horizontal stress as a function of vertical effective stress is

\[\sigma_x = \sigma_z^* K,\] Terzaghi determined that \(K=1\) based on his experimental results.

The equation is written as:

\[d\sigma_z^* S = \gamma^* S dz - 2(C' + \sigma_z K \tan \phi') dz.\]  \hspace{1cm} (4)

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

\[\left(\frac{d\sigma_z}{\sigma_z}\right) = \frac{\gamma}{\sigma_z} dz - \frac{2C}{s^* \sigma_z} dz - \frac{2K^* \tan \phi}{s} dz.\]  \hspace{1cm} (5)

The solution for the differential equation is as follows

\[\sigma_z = \frac{S^* (\gamma - 2C'/S)}{2^* K^* \tan \phi'} \left\{ 1 - e^{-2K \tan \phi' \frac{z}{s}} \right\} + p^* e^{-2K \tan \phi' \frac{z}{s}}.\]  \hspace{1cm} (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 defined 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”.

Fig. 1. Description of soil Arching analysis with Terzaghi’s method
The Nordic Guilin 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.

![Vertical Stress, kPa](image1.png)

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

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

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

with \(a\) is the width of the pile, \(b\) is the distance between the centre of 2 piles, \(\gamma\) is unit weight of the embankment.

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

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

With \(a\) is the width of the pile, \(b\) is the distance between the centre of 2 piles, \(\gamma\) 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](image2.png)

![Fig. 4. The shear stress path when trap - door max displacement of Terzaghi](image3.png)
Analyze the spherical dome based on the ratio between pressure on the pile and vertical stress on the soft soil layer, \( \frac{P'_c}{\sigma_v} \)

\[
P'_c = 8\sqrt{\frac{C_c a}{H}}.
\]  

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:

\[
\rho = \frac{P_h}{\gamma H + q_0}.
\]

Where: \( P_h \) 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, \( \gamma \) is the soil unit weight.
Han (2003) The ratio of the vertical stress on top of the cap:

\[ n = \frac{\sigma_c}{\sigma_s}. \]  \hspace{1cm} (11)

With \( \sigma_c \) is vertical stress on pile, \( \sigma_s \) is vertical stress between piles.

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

\[ \text{LKF} = \frac{Q}{Q_s} = \frac{\sigma_c A_c}{\gamma H A_c} \] \hspace{1cm} (12)

\( \Gamma \) 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.

\[ \text{CSR} = \frac{\sigma_c}{\sigma} = \frac{\sigma_c}{(\gamma H + q)}, \] \hspace{1cm} (13)

\[ \text{SRR} = \frac{\sigma_s}{\sigma} = \frac{\sigma_s}{(\gamma H + q)}, \] \hspace{1cm} (14)

\[ n = \frac{\sigma_c}{\sigma_s}, \] \hspace{1cm} (15)

\[ E = \frac{\sigma_c x a_s}{\sigma}. \] \hspace{1cm} (16)

With: \( \gamma \) 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.
According to BS8006:1995 and some researchers, SRR is calculated as Table 1.

**The stress reduction ratio**

| No. | BS 8006 1995                  | The stress reduction ratio | No. Equa |
|-----|-------------------------------|----------------------------|----------|
| 1   | $(S - a) \times (\gamma H + q)$ | $SRR = \frac{2 \times S \times (\gamma H + q) \times (S - a)}{(S^2 - a^2)^2 \times \gamma H} \left( S^2 - a^2 \left( \frac{P_c}{\gamma H} \right) \right)$ | (17)     |
|     | $H \leq 1.4(S-a)$             | $SRR = \frac{2.8 \times S}{(S + a)^2 \times H} \times \left( S^2 - a^2 \left( \frac{P_c}{\gamma H} \right) \right)$ | (18)     |
|     | $H > 1.4(S-a)$               | $SRR = \left( \frac{C \times a}{H} \right)^2$ | (19)     |
| 2   | Terzaghi                      | $SRR = \frac{(S^2 - a^2)}{4 \times H \times a \times K \times \tan \phi} \left\{ 1 - e^{-\frac{\pi}{2} \times \tan \phi} \right\}$ | (20)     |
| 3   | Randolph 1988                 | $SRR = \frac{1}{2K_p \times [\left(1 - a^2/s^2\right)^{0.5}] - \left(1 - a^2/s^2\right)^{0.5} K_p - 2]}$ | (21)     |
| 4   | Guido                         | $SRR = \frac{(s - a)}{3 \sqrt{2} \times H}$ | (22)     |
| 5   | Low 1994                      | $SRR = \frac{(K_p - 1)(1 - \delta)S}{2H \times (K_p - 2)} + (1 - \delta)^{(K_p - 1)\left[1 - \frac{S}{2H} \times \frac{S}{2H (K_p - 2)}\right]}$ | (23)     |
| 6   | Carlsson                      | $SRR = \frac{s - a}{4 \times H \times \tan \phi}$ | (24)     |
| 7   | Kivilo 1998                   | $CSR = \frac{1}{\alpha_s + \frac{E_{soul}}{E_{col}} \times (1 - \alpha_s)}$ | (25)     |
|     |                               | $SRR = \frac{E_{soul}}{E_{col} \times \alpha_s + E_{soul} \times (1 - \alpha_s)}$ | (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), $\phi$ is angle of friction of the embankment fill, $K$ is coefficient of later earth pressure ($K = 1$)

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

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 combined with geotextile

![Geosynthetic reinforced pile supported embankment](image1)

Arching in embankment.

![Arching in embankment](image2)
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.

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

(27)

where \(T_{rp}\) is the tensile force per meter geosynthetics, \(W_T\) is distributed vertical load acting on the geosynthetics between the piles, \(\varepsilon\) 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)

The equation is developed:

\[
-\sigma_z dA_u + (\sigma_z + d\sigma_z) dA_0 - 4\sigma_\varphi dA_s \sin \left(\frac{\delta_{\varphi m}}{2}\right) + \gamma dV = 0.
\]

(28)

Where:

\[
dA_u = (r\delta_\varphi)^2,
\]

(29)

\[
dA_0 = (r + dr)^2 \cdot \delta_\varphi + d\delta_\varphi)^2 = 2d\delta_\varphi \cdot r^2 \cdot \delta_\varphi + 2dr \cdot r \cdot \delta_\varphi^2 + r^2 \cdot \delta_\varphi^2,
\]

(30)

\[
dA_s = (r + \frac{1}{2}dr) \cdot \delta_\varphi + \frac{1}{2} d\delta_\varphi) \cdot dz = dz \cdot r \cdot \delta_\varphi,
\]

(31)

\[
dV = (r + \frac{1}{2}dr)^2 \cdot \delta_\varphi + \frac{1}{2} d\delta_\varphi)^2 \cdot dz = dz \cdot r^2 \cdot d\delta_\varphi^2.
\]

(32)

The equation is developed the tension force in the geosynthetic:

\[
\frac{d^2z}{dx^2} = \frac{q_2}{H} + \frac{C-x}{H},
\]

(33)

\[
H = \frac{2 \cdot \int_0^i \sqrt{1 + (z_w^1)^2} \cdot dx + 2 \cdot \int_0^i \sqrt{1 + (z_p^1)^2} \cdot dx - l_0}{2 \cdot \int_0^i \left(1 + (z_w^1)^2\right) \cdot dx + 2 \cdot \int_0^i \left(1 + (z_p^1)^2\right) \cdot dx}.
\]

(34)
Where
\[
z_W(x) = A_{1W} e^{\alpha_{1W} x} + A_{2W} e^{\alpha_{2W} x} - \frac{\beta_W}{\alpha_{1W}}, \quad 0 \leq x \leq i,
\]
\[
z'_{W}(x) = \alpha_{W} \left( A_{1W} e^{\alpha_{1W} x} - A_{2W} e^{\alpha_{2W} x} \right).
\]

The tensile force per meter geosynthetic:
\[
S(x) = \epsilon(x) / J = H \cdot \sqrt{1 + z'^2(x)}.
\]

3.2. Result of model

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 \( s = 70 \text{cm} \), 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.

| Case 1 | \( p \) (kPa) | \( \sigma \) (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 \( s = 70 \text{cm} \), 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

|   |   |   |
|---|---|---|
| $p$ (kPa) | $\sigma$ (kN/m²) | $h$ (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 geotechnik walz (igw) used for Hung Loi metro in Can Tho City, Vietnam

The model uses cylindrical piles with diameter $D = 300$mm, spacing between piles $S=4000$mm, reinforced concrete dimension of $1500 \times 1500 \times 300$mm, above is geotextile with the height of 500mm for big sand particles. Concrete layer $10 \times 20$ B.15 thickness of $250$mm, rock layer $0 \times 40$mm thickness of $350$mm.

The model is illustrated in Fig.13.

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

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

![Fig. 14. Differential settlement of Hung Loi Metro project](image)
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

| Properties                          | Index | Layer 1       | Layer 2       | Layer 3       | Sand layer | Sand embankment |
|-------------------------------------|-------|---------------|---------------|---------------|------------|----------------|
| Soft soil                           |       | 0.214*10^-6  | 1.2*10^-4     | 2*10^-2       | 3*10^-2    | cm/s           |
| stiff clay                          |       |               |               |               |            |                |
| sand                                |       |               |               |               |            |                |
| Modulus of elasticity of the unstabilized soil surrounding | $E_{sed}$ | 1252          | 14900         | 28860         | 30000      | kN/m²          |
| Poisson ratio                       | $\nu$ | 0.35          | 0.33          | 0.3           | 0.3        | -              |
| Cohesion                            | $C'$  | 8             | 71            | 1             | 1          | kN/m²          |
| Angle of friction of soil           | $\varphi'$ | 18°16'       | 26°58'        | 30°           | 30°        | degree         |

Table 4

Properties of reinforced concrete pile

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

Table 5
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 piles, the limit subsidence is $S \leq 10\text{mm}$

5.1.2. The effect of geotextile
Geotextile with high expansion, there should not be any damage on geotextile under load.

5.1.3. Sand layer
Sand particles are big with $c' = 1\text{kN/m}^2$, $\varphi' = 30\text{ degrees}$, 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](image1)

![Fig. 17. The vertical stress on top of the top pile](image2)

![Fig. 18. The tensile force per meter geosynthetic](image3)

5.2.1. Stress Distribution
Below are the graphs of the relationship between stress distribution and the pile spacing.
With the pile spacing $S = 1$ m and $S = 1.5$ m, 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.

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

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 $S=1$ m, $H = 1$ m

When $S = 1$ m, $H = 1$ m. 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 $S=1.5$ m, $H=1$ m.

When $S = 1.5$ m, $H = 1$ m. The stress concentration factor at the top of the pile is decreased $n = 4.93$, 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 \( S = 2 \text{m}, H = 1 \text{m} \).

When \( S = 2 \text{m}, H = 1 \text{m} \). 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 \( h_{\text{dap}} \geq S \) will make the spherical dome become clearer. When \( h_{\text{dap}} \geq S/2 \) then \( h_g = S/2 \). When \( h_{\text{dap}} < S/2 \) then arching height \( h_g = h_{\text{dap}} \).

- When the height \( h_{\text{dap}} < S/2 \) the deformation is not uniform, and the other way around. When choosing the sand layer, we should choose \( h_{\text{dap}} > S/2 \) 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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With the rapid growth of the economy and the trend of industrialization and modernization of the country, the demand for 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 are 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 evaluation 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.

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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 инфраструктуры 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. Библіогр. 14 назв.

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Fig. 23. Ref. 14.

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