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# A Method for Estimating Pile Group Settlement Considering Distribution of Pile Shaft Friction (SDF) – Application for Pile Groups in Vietnam

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**ABSTRACT:** Pile bearing capacity and settlement of pile groups are two most important considerations when design a pile foundation. The prediction of pile group settlement is always a difficult task. A model for pile group settlement considering distribution of friction along the pile, named SDF, is presented in this paper. Applications of the model for a full scaled experimental model by Koizumi et al (1967), Phung (1993) and two major projects in Vietnam (Camau Fertilizer Plant and – Ecopark Tower 2) are presented to illustrate the proposed model. Comparison of the calculated settlements with the results using other methods and the measured data shows that the SDF method provided the best prediction for all these cases.

**KEYWORDS:** Pile group, Settlement, Friction along pile, Vietnam

## 1. INTRODUCTION

It is known that the settlement of a pile group differs significantly from that of a single pile at the same average load level. There are a number of approaches commonly adopted for the estimation of the settlement of pile groups: 1) The empirical and semi-empirical methods (Skempton, 1953; Meyerhof, 1959; Vesic, 1969, etc); 2) The equivalent raft method (Terzaghi, 1943, SniP 2.02.03-85, Pile foundation - Russian Design Standard); 3) Interaction factor method (Poulos, 1968; Hain & Lee, 1978); and 4) Numerical methods (finite element method or finite difference method). A detailed review of methods of calculating pile group settlement was made by Phung (1993). In Vietnam, various methods of settlement calculation for pile group have been used, including the equivalent raft method, the empirical methods, and numerical methods.

A study conducted for the evaluation of methods for predicting settlement of pile groups in Viet Nam showed that the empirical method (Vesic, 1969) and the equivalent raft method provided reasonable results for small pile groups. For large pile group, however, the calculated results using these methods were different from the monitoring data, Duong et al (2014b). Numerical methods have many advantages in modeling complicated pile-soil interaction problems in heterogenous soils using advanced soil models; however, these methods are complicated, time-consuming and require specialized knowledge and experience. Besides, with the current geotechnical investigation in Vietnam, it is sometimes difficult to provide reliable input parameters for numerical models.

A method for estimating the pile group settlement considering the distribution of pile shaft friction and pile tip resistance, so-called SDF, is proposed by author, Duong et al (2014a). Based on *τ-z* curves, the relationships between the unit shaft friction *τ* versus the pile movement *z*, and *q-z* curves, this model presents the pile end unit bearing *q* versus the pile toe movement *z*, and the distribution of load in the pile. The pile depth and the relative distances *x* and *y* between the piles are both taken into account to establish a 3-dimensional model and to estimate the interaction between the piles in a pile group.

In this paper, using results from the full scale test, conducted by Koizumi (1967), Phung (1993) and settlement monitoring data of two projects in Vietnam, the authors aim to: 1) compare and evaluate the results from SDF method to the actual measurement from Koizumi (1967) and Phung (1993); and 2) compare the results from SDF method with settlement monitoring data from two major projects in Vietnam.

## 2. SDF METHOD

Pile group settlement estimated using the SDF method, proposed by Duong et al (2014a) is presented in this section. In this method the stress at any point in the subsoil is calculated, considering the distribution of friction forces along the pile and the tip resistance.

### 2.1 Assumptions and calculation steps in SDF simple method

In the SDF method, the following assumtions are used:

- Cap is not in contact with the soil surface and the applied load is therefore transferred entirely to piles;
- All piles have the same length;
- Soil condition is the same within pile group.

The calculation steps are as follows:

Step 1: Determine the force on the pile head.
If force on piles head are the same than $P_0 = P/m$, in which P is the working load of the pile group, $P_0$ is load on pile head and *m* is number of piles in the pile group.

Step 2: Divide a pile equally into n segments. Length of each segment is $d_h = L/n$, in which L is the length of the pile. Each segment must be located within one soil layer.

Step 3: If friction force at segment i[th] is $F_i$, and the tip resistance is $P_t$ (Figure 1b), force equilibrium for each pile can be written as :

$$P_0 = \sum F_i + P_t \tag{1}$$

More details are presented in section 2.2.

Step 4: Apply the $F_i$ and $P_t$ forces calculated for single pile to all piles in group. Based on distance of piles along x and y directions, model three-dimensionally with the loads applied at depths within the soil (Figure 1c).

Step 5: Calculate stress (section 2.3).

Step 6: Calculate settlement of soil layers below the pile tip plane.
With different loads at pile heads, after determine the force on the pile head for all pile we calculated $F_i$ and $P_t$ forces from step 2 and 3.

### 2.2 Distribute force in each pile

The τ-z and q-z curves are used to distribute force $P_0$ in each pile to friction forces along pile $F_i$ and tip resistance $P_t$ at pile tip (figure 1b). The τ-z curve represents the relationship between the unit shaft friction versus the relative movement between the pile shaft and the soil. the q-z curve represents the pile end unit bearing capacity versus the pile toe movement. The steps to distribute $F_i$ and $P_t$ forces are as follows:

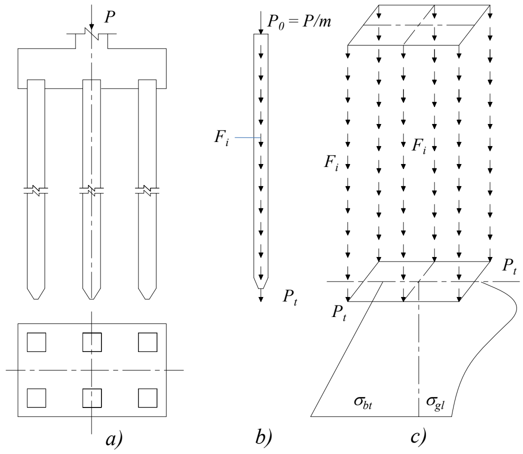



Figure 1 SDF Model with force on piles head are the same

Step 1: Choose the model τ-z and q-z curves applied to each soil layer. Any theoretical models, empirical curves, or actual measurement data for soil layers can be used (Figure 2a). Duong et al (2015a and b) studied the τ-z curve based on pile load test results which used strain gauges along pile in numerous major projects in Vietnam, and recommended that the τ-z curve be applied for some soil types in Vietnam.

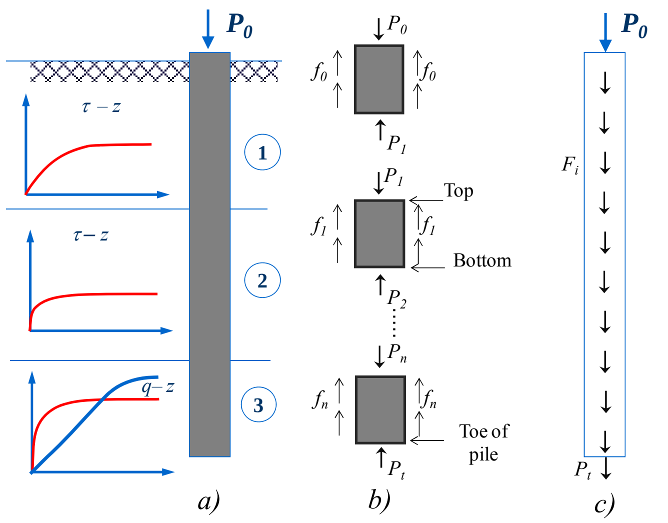



Figure 2 Distribute force in the pile

Step 2: Assuming small displacement of pile tip, $z_1$ from q-z curve; the pile base load, $P_t$, corresponding to $z_1$ is given in Eq.2.

$$P_{j,t} = p_{j,t} \cdot A_t \tag{2}$$

Where: j = name of the pile in the group;
$p_t$ = pile end unit bearing mobilized (from q-z curve);
$A_t$ = area of pile tip.

Note: A small settlement $z_1$ depends on the dimension of the pile and force on the pile head $P_0$. If $P_0$ value is small enough then $z_1$ will became zero, indicating no movement at the pile tip or no pile tip resistance mobilization.

Step 3: Assume that deformation in one pile segment is constant. Determine friction force $\tau_n$ from settlement $z_1$, based on τ-z curve for pile tip segment to. Friction force is determined by the following formula:

$$F_{j,n} = f_{j,n} \cdot U \cdot d_h \tag{3}$$

In which:
U = perimeter of pile;
$d_h$ = length of pile segment; and
$f_n$ = unit shaft friction between soil and pile.
$P_{j,n}$ at bottom of pile tip segment is determined by the following formula:

$$P_{j,n} = P_{j,t} + F_{j,n} \tag{4}$$

Step 4: Displacement of pile segment (n-1) equals to $z_1$ plus elastic deformation. Determine friction force $f_{j,n-1}$, $F_{j,n-1}$ and $P_{j,n-1}$ from new displacement, based on τ-z curve . The same calculation can be completed for the remaining pile segments. Finally, we have force at pile head $P_{1,0}$ and displacement $f_1$.

Step 5: Compare $P_{1,0}$ and $P_0$

+ If $P_{1,0} = P_0$, we will have friction forces and pile tip resistant.

+ If $P_{1,0} < P_0$ then increase $z_1$ up to $z_2 = 2.z_1$. Repeat step 3 to step 5 until $P_{j,0} \geq P_0$. At each iteration, value of $z_1$ is added once. At the end of each iteration, we have friction and tip resistance at step i and i-1. By interpolating from two steps, we can determine friction forces and pile tip resistance corresponding to $P_0$.

+ If $P_{1,0} > P_0$, settlement $z_1$ can be assumed again. If $z_1$ is insignificant (less than $10^{-9}$ mm for example) but $P_{1,0}$ still larger than $P_0$ then we can ignore the tip resistance and proceed the calculation with only friction at pile segments. Pile segments near pile tip can be omitted until the $P_{1,0} \leq P_0$ condition at the first step is found). This condition occurs when the force $P_0$ at the pile head is insignificant. Friction components of upper soil layers have mobilized enough strength and friction and/or tip resistance of the soil layer below are not mobilized.

The results of this step are $F_i$ force located at the center of pile segments and pile tip resistance $P_t$ (Figure 2c).

### 2.3 Calculate stresses

We have friction point load $F_i$ located at the center of pile segments and $P_t$ at pile tip (Figure 1b). Apply the $F_i$ and $P_t$ forces calculated for one pile to all piles of the group. Based on distance of piles along x and y directions, model three-dimensionally with the loads are applied at various depths within the soil. Apply Mindlin equation (Mindlin 1936) to calculate stress from pile tip plane to the end of compressible depth of soil. Mindlin (1936) proposed an equation to calculate the stresses in an elastic half space induced by a point load applied at some finite depth as below:

$$\sigma_z = \sum_{i=1}^{m}\sum_{j=1}^{n} \frac{P}{8\pi(1-\upsilon)}(F_{i,j,1} - F_{i,j,2} + F_{i,j,3} + F_{i,j,4} + F_{i,j,5}) \tag{5}$$

Where:
Forces from $F_{i,j,1}$ to $F_{i,j,5}$ are determined according to Mindlin (1936), corresponding to pile i and pile segment j.

### 2.4 SDF advanced method

The SDF advanced method can be used for the following conditions:
+ Different loads at pile heads;
+ Soil condition changes within pile group;
+ Pile group with different pile lengths.

## 3. COMPARE SDF METHOD WITH RESULTS OF FULL SCALE TEST

The full scale experiment was conducted by Koizumi et al (1967) and Phung (1993) with the following conditions:
- Pile cap was not in contact with the ground surface (Figure 3);
- Pile group had a rigid cap;
- All piles were of the same length;

- The soil condition was the same within pile group.

Experimental conditions satisfied the assumptions of the SDF method.

### 3.1 Full scale experiment Koizumi et al (1967)

Koizumi et al (1967) analyzed the effect of pile driving on the surrounding soil and compared the settlement of single pile to that of pile group. Distance from the tested single pile to pile group was 4.2 m. All piles were made of steel with the elastic modulus of 200,000 MPa, 300 mm in diameter, 1.6 mm in thickness, and 5.5 m in length. For the pile group foundation, the piles were spaced at 900 mm (three times diameter of pile) center to center in a square array. Distance from bottom of pile cap to ground surface was 1.3m. The single pile was numbered as 1; the piles of pile group were numbered from 2 to 10 as shown in Figure 3.

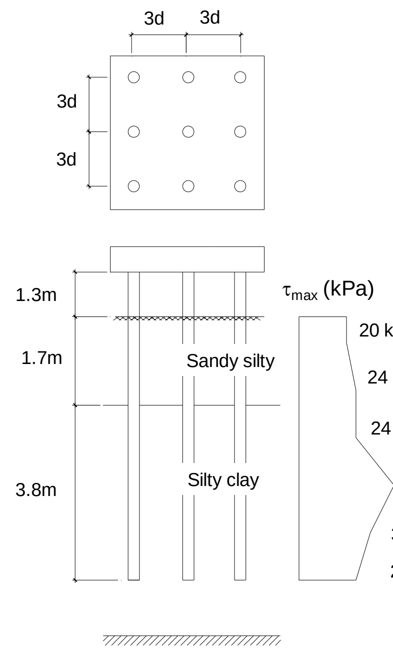



Figure 3 Full scale test presented by Koizumi et al (1967): layout of the pile group and subsoil model

Three borings and six Dutch cone penetration tests were conducted within the test area. The soil profile at this site includes 1.7 m of sandy silt and 13.5 m of silty clay. Below this depth, gravel and dense sand layers extended to bedrock. The values of shear strength measured from laboratory and in situ tests are shown in Figure 3. The shear strength of the clay was approximately 25 kPa at the foundation level and about 40 kPa at the proposed pile tip.

#### 3.1.1 τ-z and q-z curves

We use the τ-z models that were proposed by Vijayvergiya (1977) and Heydinger & O'Neill (1986), and q-z model that was proposed by Vijayvergiya (1977) for the analysis. Because shear strength varies with depth, the τ-z curves can be determined based on the average of shear strength within 1 m, maximum of displacement $z_c$ is 5 mm. Pile tip resistance is 40 kPa with the maximum of displacement of three percents of pile diameter d. From the back-analysis of the single pile determined from the loading tests, the value of elastic modulus $E_s$ can be estimated using method proposed by Roberto & Enrico (2006). The estimated values of the elastic modulus of the upper and lower soils are 12.8 MPa and 15.6 MPa, respectively.

#### 3.1.2 Calculation results

Small displacement of pile tip, $z_1$ was 0.01 mm. Number of displacement step was 8000. Length of segment was 0.1 m. A calculation program was written using Visual Basic Application in Excel (VBA).

The results of calculation using SDF approach is shown in Figure 4 for the Vijayvergiya (1977) τ-z model, and the Vijayvergiya (1977) model for sand and Heydinger & O'Neill (1986) model for clay.

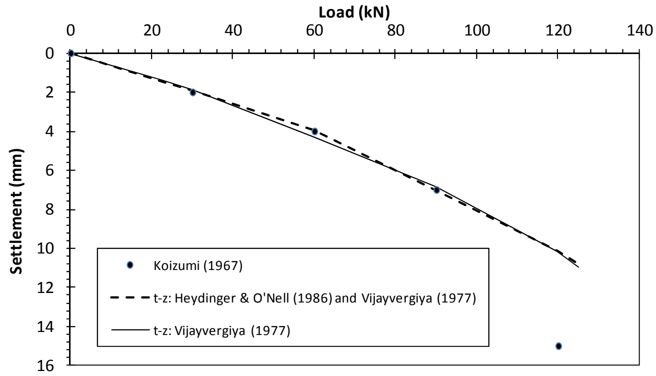



Figure 4 Comparison between the predicted and measured load – settlements for pile group

Comparisons between the measured load - settlement curve for the pile group and that calculated using the SDF method in the previous sections are shown in Figure 4. The results show that:

- SDF method predicts that the greatest external load on pile reaches approximately 140 kN (corresponding to 1260 kN on pile group) when the entire friction and tip resistance are mobilized.
- At elastic stage, the results of SDF method (using both τ-z models) are consistent with the full scale test results of pile group settlement. However, the calculated results using the τ-z models of Vijayvergiya (1977) for sand and Heydinger & O'Neill (1986) for clay agree better with the measured values.
- At high load levels, however, the calculated results may differ significantly from those measured. Calculated settlements are larger than the measured results from Koizumi (1967). This difference is probably due to the assumption of plastic deformation stage in the τ-z and the q-z models.
- SDF method does not describe the plastic deformation stage when the load on pile continues to increase or unchange. This is because the stress of τ-z and q-z models at plastic deformation stage is constant when the displacement increases.

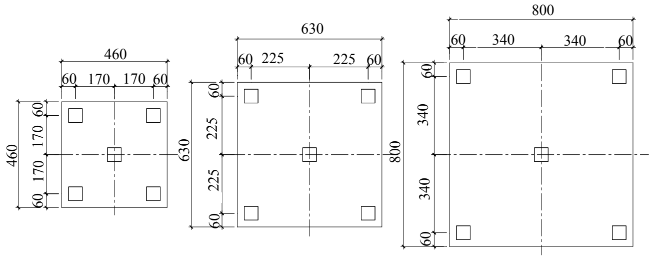
### 3.2 Large- scale field model test by Phung Duc Long (1993)

In order to clarify the overall cap-soil-pile interaction and the load-settlement behaviour of piled footings in non-cohesive soil, systematic large-scale field model tests were performed (Phung, 1993). Through the study, the Author has tried to create a better understanding of the load-transfer mechanism and of the load-settlement behaviour of a piled footing in non-cohesive soil, as well as the overall pile-cap-soil interaction. Three different series of large-scale model tests (denoted as T1 T2 and T3) were performed. Each test series consisted of four separate tests on a shallow footing, a single pile, a free-standing pile group, and a piled footing under equal soil conditions and with equal geometry.

In this paper we compare the experimental result of free-standing pile groups with the calculation using the SDF method.

The model piles in the field tests were hollow pile with a square cross-section, 60x60 mm, and a wall thickness of 5 mm. The pile length was about 2.1m in the tests on free-standing pile group. The test pile groups were square and consisted of five piles: one central pile, pile No.1 and four corner piles, piles Numbers 2 to 5, according to the driving order.

The footings were made of pre-fabricated reinforced concrete. The first test (T1G), the footing was 460x460mm and 300mm in thickness. In the T2G and T3G tests, the footing size was 630x630x350mm, and 800x800x400mm, respectively (Figure 5).




a) T1G test b) T2G test c) T3G test

Figure 5 Model footings in plan

The soil is sand with a relative density of 38%, 67% and 62% respectively in the tests T1G, T2G and T3G. The secant modulus $E_{25}$, $E_{50}$ had determined by back-calculated from tests on shallow footings at 25% and 50% of failure load. For T1G, T2G and T3G tests, $E_{25}$ was 18 MPa; 30.4 MPa and 23.5 MPa, $E_{50}$ was 6.5 MPa; 18.7 MPa and 14.2 MPa, respectively. The τ-z, q-z models according to Vijayvergiya (1977) or API (1993) are used.

### 3.2.1 Free-standing pile group T1G

The test for the smallest pile group, Test T1G, was performed by one loading sequence to failure, with a maximum load of about 26 kN and a load step of 2 kN. From the test result, it can be seen that the load distribution among the piles seems to depend on the driving order. The central pile took the largest load.

Two cases with diffirent loads on pile heads are analysed using the SDF method: one with the same load per pile, another with loads taken from the the test done by Phung (1993). The results from the two analyses are however quite similar. The SDF calculated result is compared with Test T1G in Figure 6A.

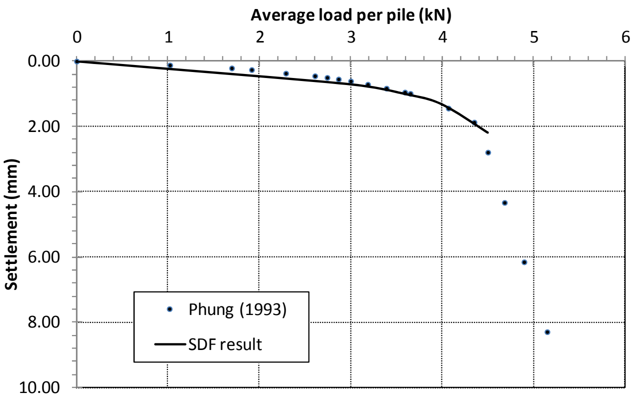



Figure 6A Comparison of the predicted and measured load – settlement behaviour for the pile group T1G

### 3.2.2 Free-standing pile group T2G

Test T2G was performed by one loading sequence to failure, with a maximum load of about 85 kN and a load step of 5 kN. The SDF calculated result is compared with Test T2G in Figure 6.

### 3.2.3 Free-standing pile group T3G

Test T3G was performed by one loading sequence to failure, with a maximum load of about 85 kN and a load step of 5 kN. The SDF calculated result is compared with Test T3G in Figure 7.

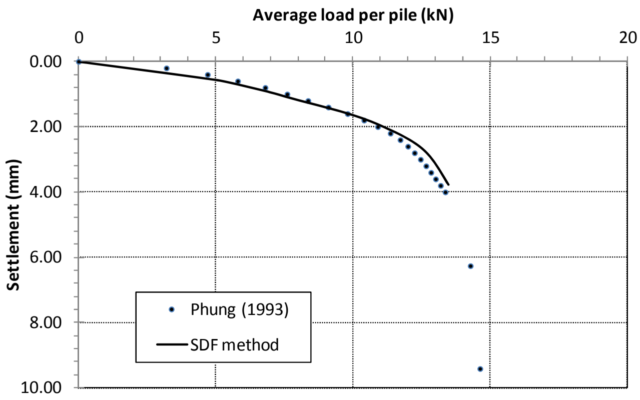



Figure 6 Comparison of the predicted and measured load – settlement behaviour for the pile group T2G

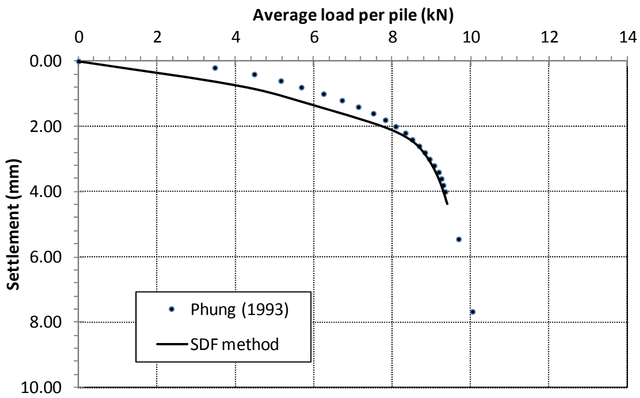



Figure 7 Comparison of the predicted and measured load – settlement behaviour for the pile group T3G

The comparisons show a quite good agreement between the SDF calculated and the measured results. For simplicity, the loads on piles can be taken uniformly distributed.

## 4. CALCULATION FOR SOME PROJECTS IN VIETNAM

### 4.1 Ca Mau Fertilizer Plant Project

Ca Mau Fertilizer Plant (Figure 8) is the largest project in Gas – Power – Fertilizer combination project at Khanh An Commune, U Minh District, Ca Mau Province, Vietnam. The project was started on 07/26/2008, the construction and installation, commissioning was completed, and the first commercial product was provided on 01/29/1012. The plant commenced its commercial operation on 04/20/2012. Soft soil improvement for this project involved the use ofprefabricated vertical drain (PVD) and vacuum pump.

Local soil stratigraphy consists of four layers. There are fill sand, fat soft clay, stiff to very stiff clay and firm to stiff clay with sand, with corresponding thicknesses of 3.8 m, 15.5 m, 7.0 m and undefined depth. The groundwater level is at 10 m depth. The geology of the project area was relatively stable between bore holes. The characteristics of layer 4 are shown in Table 1 and undrained shear strength ($S_u$) profile is shown in Figure 9. Average of undrained shear strength is 110 kPa for layer 3 and 85 kPa for layer 4 (according to BS 8004:1986).

The results of two-year settlement monitoring are shown in Figure 13. The monitoring results were averaged from 8 settlement points.

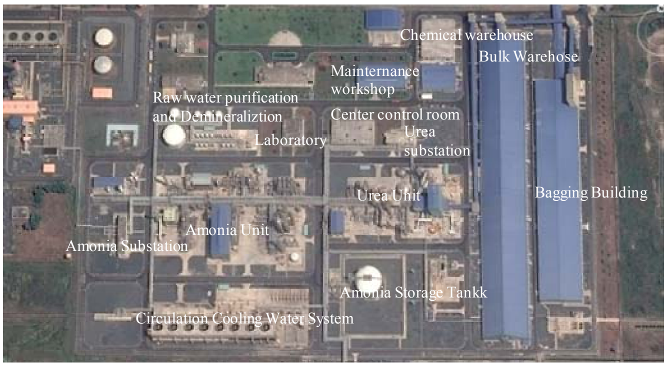



Figure 8 Location Plan of Ca Mau Fertilizer Plant (https://www.google.com)

Table 1 Soil parameters for layer 4

| Depth (m) | $\gamma$(g/cm$^3$) | $e_0$ | $C_c$ | $C_r$ | Pc (kG/cm$^2$) |
|---|---|---|---|---|---|
| 33 – 33.5 | 1.93 | 0.771 | 0.179 | 0.032 | 3.22 |
| 42 – 42.5 | 1.82 | 0.944 | 0.422 | 0.083 | 7.14 |

Duong et al (2015a and b) recommended that some $\tau$-z models be used and adjusted for soil conditions in Vietnam. We use $\tau$-z and q-z models of Vijayvergiya (1977). The $\tau$-z curves can be determined based on the average of undrained shear strength within layers, maximum displacement $z_c$ is 5mm. The q-z curve can be determined with a maximum displacement of 3%d (d=pile diameter) and $N_c$ = 9 (where Nc is the coefficient required to determine maximum pile tip resistance; recommended by Aschenbrener & Olson, 1984).

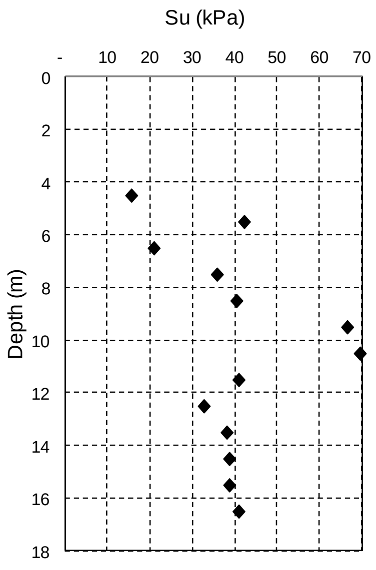



Figure 9 Undrained shear strength

The monitoring systems are located primarily in the important structures such as tanks and compressors. The number of Fertilizer Plant units is calculated as follows:

### 4.1.1 Amonia Storage Tank

Pile plan and monitoring systems for the Amonia Storage Tank foundation is shown in Figure 10. Thickness of pile cap is 0.7 m and the tank is 28 m high. In the analysis, a tank load of 11 kN/m$^2$, a dead load of 27.15 kN/m$^2$ and an equipment operation load of 117 kN/m$^2$ are used. A total of 334 piles with 0.4 m diameter, 33 m long piles were constructed. Load on pile head is 412 kN.

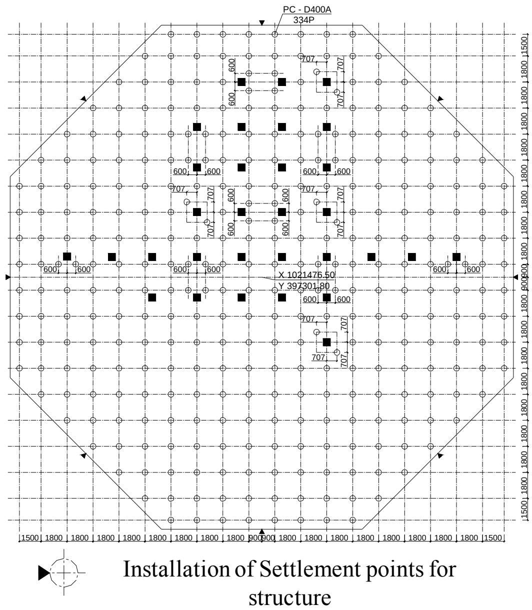



Figure 10 Pile plan and monitoring systems of Amonia Storage Tank foundation

The calculation result of the SDF method converged with $z_1 = 10^{-8}$ mm. In this case, it is almost impossible to mobilize pile tip resistance component. Skin friction, axial force in pile and maximum stress of soil under pile tip plan are shown in Figures 11 and 12.

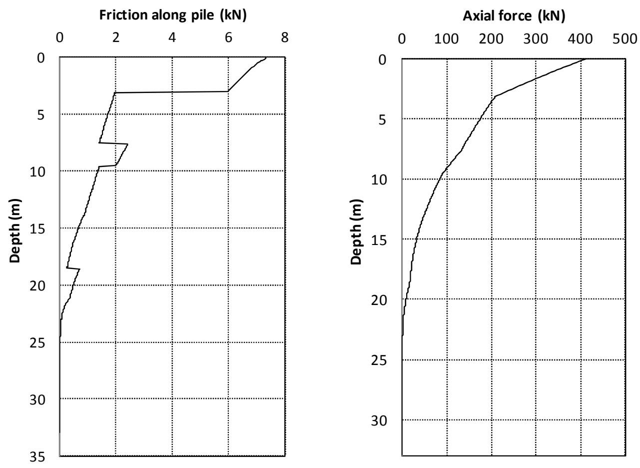



Figure 11 Pile skin friction and axial force versus depth

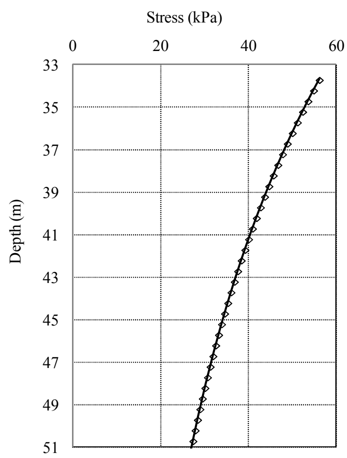



Figure 12 Maximum stress of soil under pile tip plan

The results of two-year settlement monitoring are shown in Figure 13. The monitoring results were averaged from 8 settlement points.

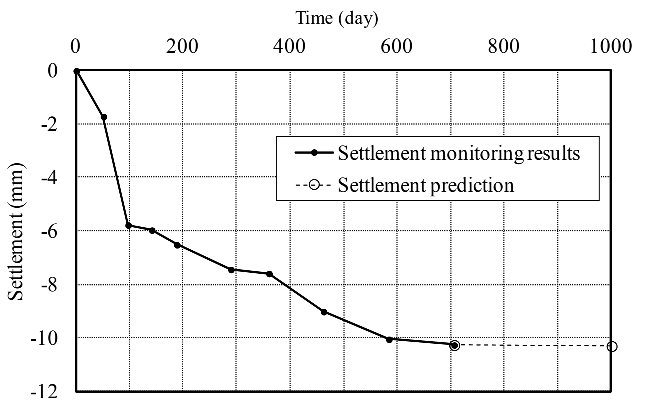



Figure 13 Settlement monitoring results of Amonia Storage Tank foundation

The results of settlement calculation using the SDF approach were compared with settlement monitoring results and those using other methods (eg SniP 2.02.03-85; Vesic, 1969) as shown in Table 2.

Table 2 Calculated Results

| Foundation | SniP (cm) | Vesic (1969) (cm) | SDF Method (cm) | Measured (cm) | Prediction (cm) |
|---|---|---|---|---|---|
| Distillation Tower | 0.36 | 3.59 | 0.52 | 0.48 | 0.50 |
| Amonia Compressor | 0.58 | 2.79 | 0.76 | 0.52 | 0.55 |
| Amonia Storage Tank | 14.19 | 5.56 | 1.19 | 1.026 | 1.03 |

### 4.1.2 Distillation Tower

Pile plan and monitoring systems of Distillation Tower foundation is shown in Figure 14. Thickness of pile cap is 1.2 m and tank height is 32.52 m. An equipment operation load of 265 kN, a dead load of 5,810 kN and a live load of 325 kN were used. A total of 6 piles with 0.5x0.5 m square cross-section of and 38 m long of were constructed. Load on pile head is 412 kN.

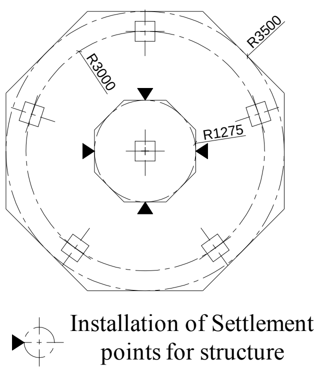



Figure 14 Pile plan and monitoring systems of Distillation Tower foundation

The calculation results using SDF method converged with $z_1$ = 0.001 mm. In this case, the pile group is small but load on pile head is large. The tip resistance is mobilized. Skin frictions, axial force in pile and maximum stress of soil under pile tip plan are shown in Figures 15 and 16.

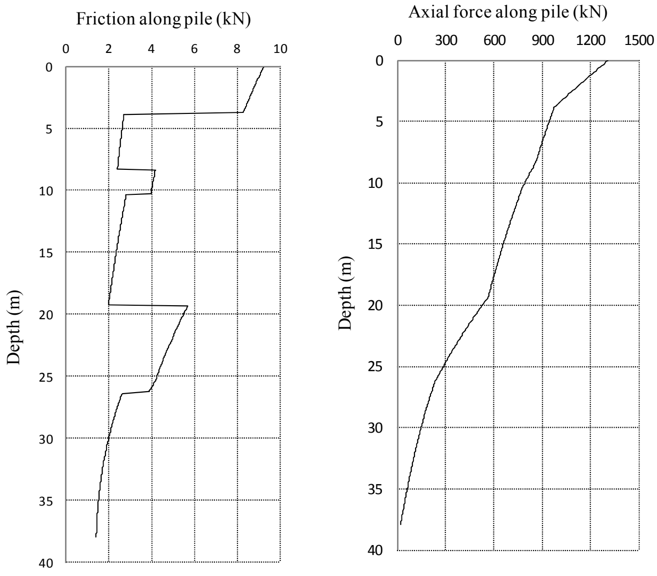



Figure 15 Pile skin friction and axial force

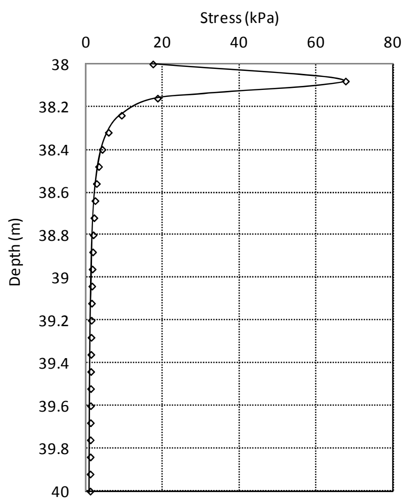



Figure 16 Maximum stress of soil under pile tip plan

The results of two year settlement monitoring are shown in Figure 17. The monitoring results were averaged from 4 settlement points.

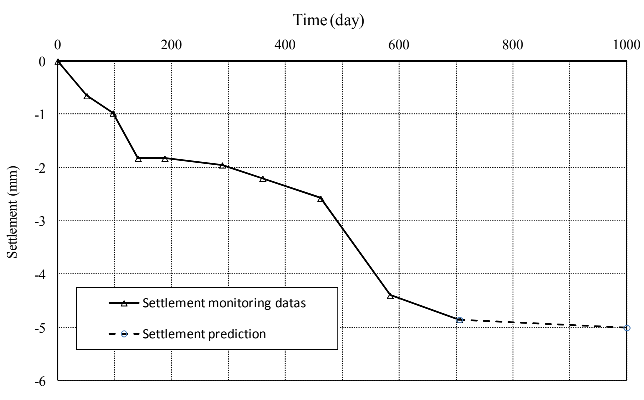



Figure 17 Settlement monitoring results of Distillation Tower foundation

Calculated results using SDF approach were compared in Table 2 with settlement monitoring results and those using other methods (eg SniP 2.02.03-85; Vesic, 1969).

### 4.1.3 Amonia Unit – Amonia Compressor

Pile plan and monitoring systems of Amonia Unit – Amonia Compressor foundation is shown in Figure 18. In the analysis, a dead load of 203kN/m$^2$ and a live load of 10kN/m$^2$ are used. A total of 20 piles with a 0.5 m diameter and 30 m long were constructed.

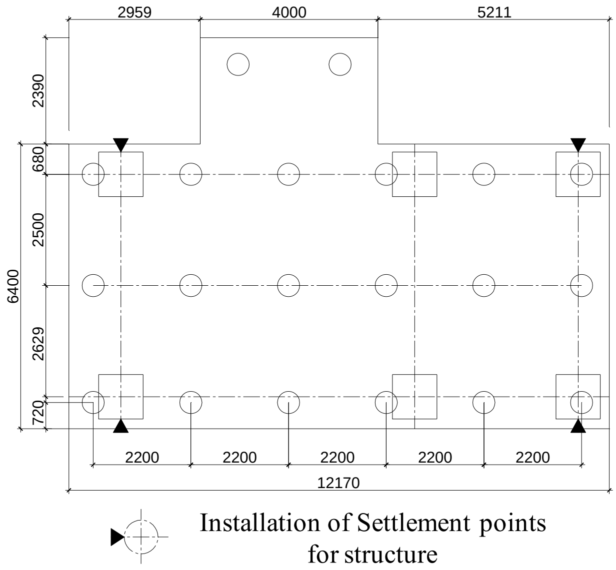



Figure18 Pile plan and monitoring systems of Amonia Unit – Amonia Compressor

The results of calculation using SDF method converged with $z_1$ = 0.001 mm. The same as above, skin frictions, axial force in pile and maximum stress of soil under pile tip plan are shown in Figures 19 and 20.

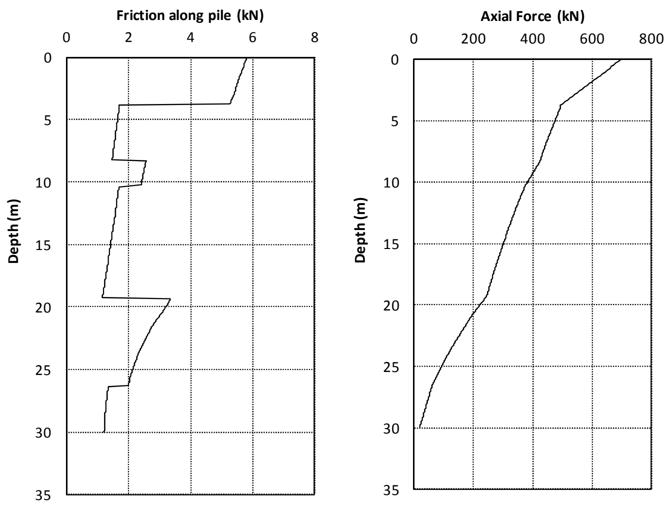



Figure 19 Pile skin friction and axial force versus depth

Settlement monitoring results during two years is shown in Figure 21. The monitoring results were averaged from 4 bench makers.

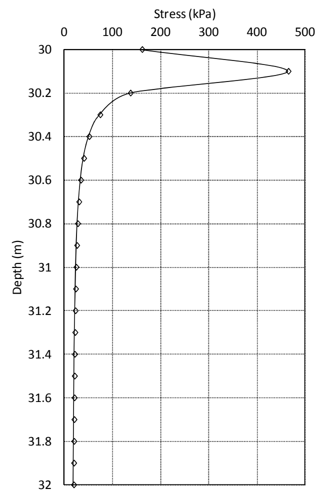



Figure 20 Maximum stress of soil under pile tip plan

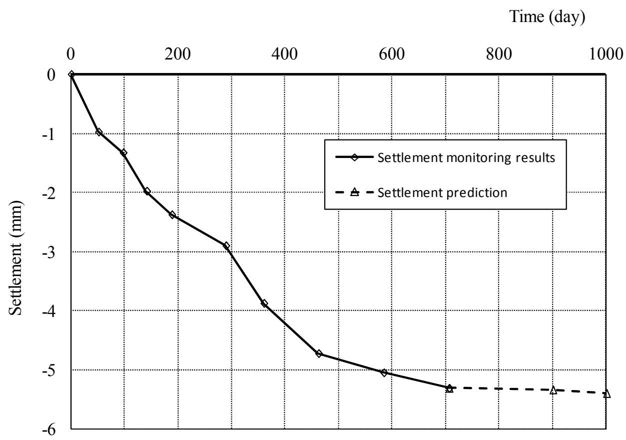



Figure 21 Settlement monitoring results of of Amonia Unit – Amonia Compressor

Calculated results derived from SDF approach were compared with settlement monitoring and other methods (eg SniP 2.02.03-85; Vesic, 1969) as Table 2.

### 4.1.4 General comments on the results

For 3 units in Ca Mau Fertilizer Plant, we have calculated some scenarios, in which pile group with various sizes and loads were considered. Soil investigation data is adequate and subsoil consdition is relatively typical. The results suggest that Snip 2.02.03-85 method is suitable with small pile groups. Empirical method proposed by Vesic (1969) is not suitable because the settlement only depends on the foundation width without considering other factors. The SDF method is more suitable. This method has distributed friction along pile and tip resistance. Stress of subsoil under pile tip plan is decreased.

### 4.2 Tower 2 - Ecopark urban in Hung Yen Provice

Condominium Project 1A-01 was built in the north of the Ecopark urban at Van Giang district, Hung Yen Province. The condominium functions primarily as apartments and shopping areas in the first floor. The project consists of 5 towers, namely Tower 1 to Tower 5 (Figures 22 and 23).


Figure 22 Five Tower Location Plan (https://www.google.com)

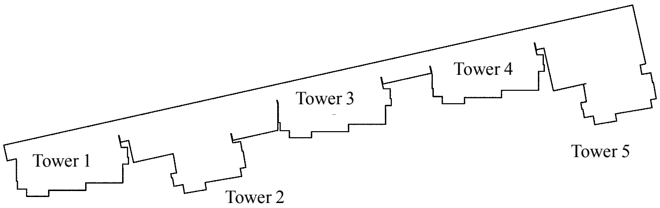



Figure 23 Five Tower Plan at Ecopark Urban

The project location is near the Red River with complicated geological and hydrological conditions. Bored pile trials were constructed; however, quality of construction was not ensured because of varying ground water level. Therefore, this project used reinforced concrete pile raft for all Tower.

In this paper, only Tower 2 is considered. This Tower consists of 2 blocks of 19 floors - 19 floors with a total size is 36.80 x 55.91 x 66.30 m. Thickness of pile raft is 2 m. Pile raft consists of 381 square piles with 450x450 mm cross-section and 32 m long. Total load on pile raft is about 440.710 kN. Pile plan of Tower 2 foundation is shown in Figure 24.

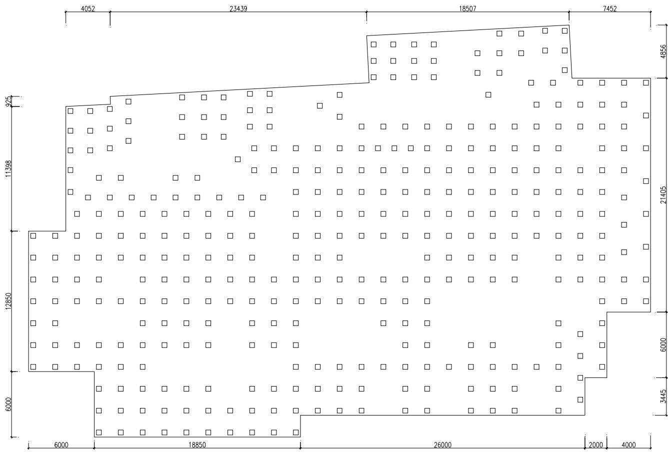


Figure 24 Pile Plan of Tower 2

Soil investigation for Tower 2 includes three boreholes (B4, B5 and B6). We calculate settlement of pile raft for the worst geological case B4 and the best geological case B6 (Table 3). The groundwater level is from 2.82 m to 3.35 m depth.

Duong et al (2015a and b) proposed that τ-z models be used and adjusted to suit the soil conditions in Vietnam. We used the Vijayvergiya (1977) τ-z and q-z models. The model parameters can be selected as follows:

+ Layers 1, 2a and 2b would be excavated during the construction.

+ Layer 3, maximum unit skin friction $\tau_u$ is 15 kPa (BS 8004:1986).

+ Base on the correlation of IL: Layer 4a, 7 $\tau_u$ = 45 kPa. Layer 5 $\tau_u$ = 50 kPa and layer 4b $\tau_u$ = 100 kPa.

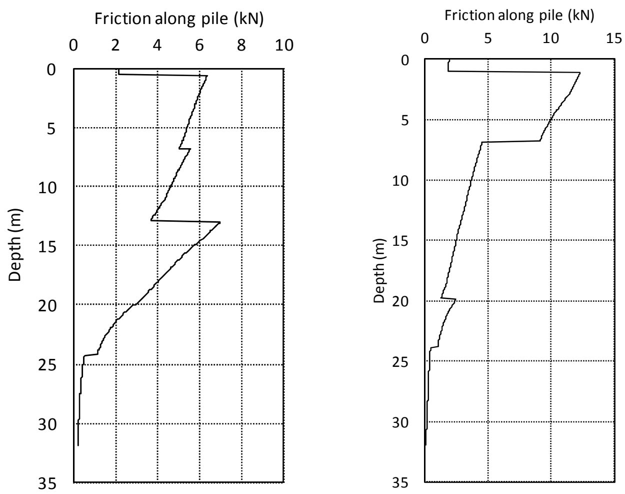
+ Layers 6a and 6b, $\tau_u$ = 95.76 kPa according to Vijayvergiya (1977).

The τ-z curves can be determined based on the average of undrained shear strength within layers, maximum displacement $z_c$ of 5 mm. The q-z curve can be determined with maximum displacement of 3%d (d=pile diameter) and $N_c$ = 9.

Table 3 The characteristics of soil at boreholes B4 and B6

| Layer | Soil name | Depth (m) | | γ g/cm$^3$ | IL |
|---|---|---|---|---|---|
| | | B4 | B6 | | |
| 1 | Fill sand | 1.1 | 1.2 | - | - |
| 2a | Clay, stiff | 0.9 | 0.8 | 1.81 | 0.31 |
| 2b | Clay, firm | - | | 1.75 | 0.58 |
| 3 | Very soft clay | 5.2 | 5.8 | 1.51 | 1 |
| 4a | Clay, sandy clay, soft to firm | 6.2 | - | 1.96 | 0.65 |
| 4b | Clay, sandy clay, firm to stiff | - | 5.7 | 1.97 | 0.25 |
| 5 | Sandy clay, firm | 6.1 | 13 | 2.03 | 0.56 |
| 6a | Medium dense sandy | 9.9 | 3.9 | - | - |
| 6b | Silt dense sandy | 1.4 | - | - | - |
| 7 | Sandy clay, soft | 16 | 10.1 | 1.78 | 0.69 |
| 8 | Sandy clay, soft to firm | - | - | 1.92 | 0.74 |
| 9a | Silt sandy to gravel, dense | 6.9 | 10.7 | - | - |
| 10 | Very dense gravel | - | - | - | - |

The results of SDF method converges with $z_1$ = 0.01 mm for both borehole 4 and 6. Number of displacement step is 80.000. Length of segment is 0.1 m. Side frictions, axial force in pile and maximum stress of soil under pile tip plan are shown in Figures from 25 to 27.




a) Borehole 4 b) Borehole 6

Figure 25 Friction along pile

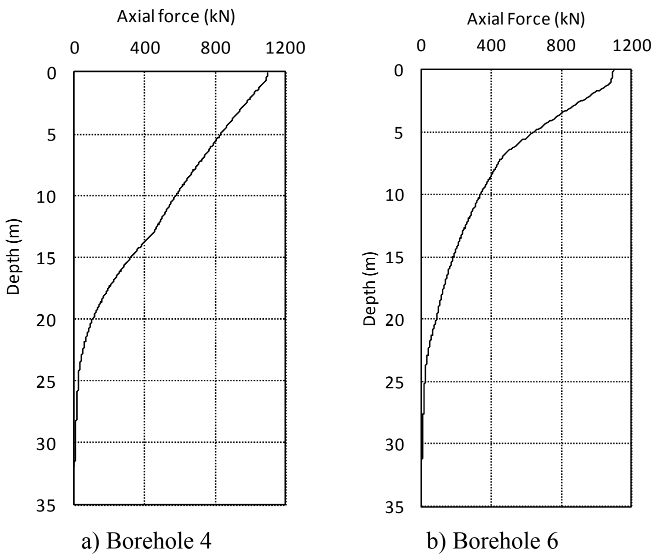



Figure 26 Axial force in pile

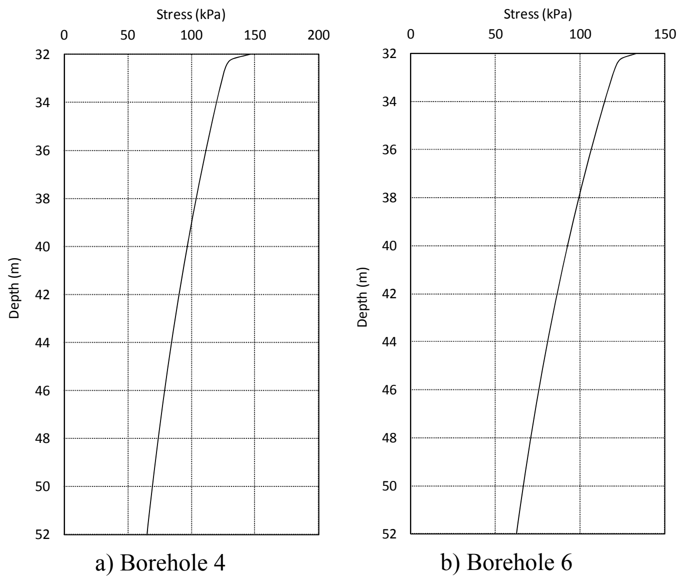



Figure 27 Soil Stress under pile tip plan

The settlement monitoring was made after the construction work was completed. Settlement monitoring was conducted from 05/9/2011 to 05/08/2013 for 10 monitoring trips. Locations ofsettlement monitoring points is shown in Figure 28.

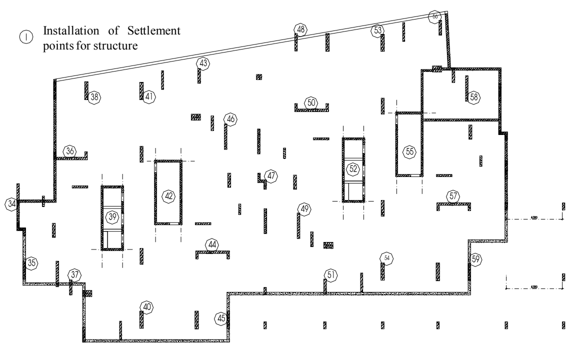



Figure 28 Monitoring point location plan for Tower 2

The result of two-year settlement monitoring is shown in Figure 29. A total of 26 settlement points, numbered from M34 to M59 was monitored. The maximum pile raft settlement is 16.82 mm at settlement point M52 location.

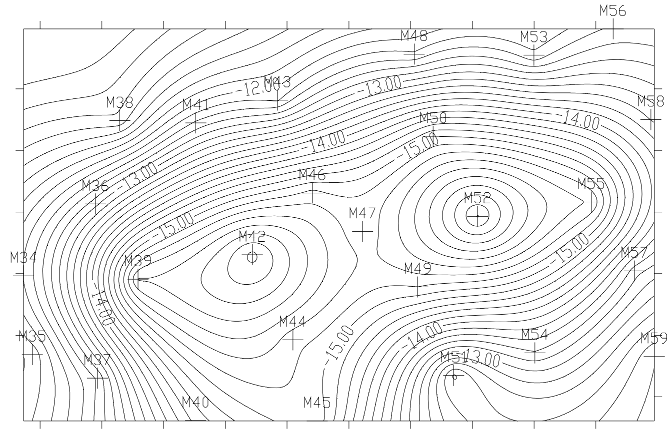



Figure 29 Settlement Photomap of Tower 2 (mm)

The calculated results using SDF approach were compared with those using other methods as well as settlement monitoring results in Table 4.

Table 4 Comparison between the calculation results and settlement monitoring data

| Foudation/Borehole | Equivalent raft* (cm) | SDF method (cm) | Monitoring (cm) |
|---|---|---|---|
| Tower 2 /B4 | 15.38 | 7.62 | 1.68 |
| Tower 2/ B6 | 12.86 | 4.49 | 1.68 |

* Refer to the predicted settlement of Geotechnical and Construction Engineering Institute.

### 4.3 General comments on the results

According to Vietnamese Building Code, the allowable pile settlement is 8 cm. The design consultants for the project conducted the pile group settlement calculation; however, theresults were much larger than the allowable settlement. This issue has caused some controversy and difficulties in obtaining project approval from the authorities.

The results calculated by using equivalent raft method provide too large deviations. The geological and hydrological conditions at this project site are very complicated. We calculated settlement for both the worst geological (B4) and the best geological (B6). Results of two cases are very different.

The calculated results using SDF method differ slightly from the settlement monitoring data. This is likely due to the delay in settlement monitoring, which was conducted after the completion of construction work. In reality, the ground experienced settlement during the construction. Some parameters of soil were not tested. So, the parameters in the τ-z and q-z model must be selected based on the correlations. In SDF method, interactive was considered primarily as piles - piles and piles – soil. Although there is a slight different, the calculation results using SDF method result is acceptable.

## 5. CONCLUSION

Based on the calculation and analysis presented above, the authors have the following comments:

- At present, no work has been done to address the suitability of settlement calculation methods, with possible adjustments for the local geotechnical conditions in Vietnam.
- With the limited geotechnical investigation in Vietnam, it is difficult to provide the full range of input parameters for numerical modeling.
- The results of SDF settlement calculations are quite close to the measured ones from full scale experimental results conducted by Koizumi (1967). For two major projects in Vietnam, results of SDF method are slightly different from settlement monitoring data. However, this difference appears to be acceptable.
- SDF method considers two interactions: piles - piles and piles - soil. Although SDF method may be difficult for calculating by hand, it is not as complicated as FEM method when model 3-dimensional problems.
- SDF method can be used to calculate the settlement of pile groups in Vietnam soil conditions.

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