International Journal of GEOMATE, Aug., 2022, Vol.23, Issue 96, pp.145-152
ISSN: 2186-2982 (P), 2186-2990 (O), Japan, DOI: https://doi.org/10.21660/2022.96.3433
Geotechnique, Construction Materials and Environment

VERIFICATION AND VALIDATION OF THE PILE DESIGN METHOD WITH CONSIDERATION OF DOWN DRAG

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ABSTRACT: This article evaluates the performance of the pile design method that considers Down Drag (PDwDD method). The design approach has the main advantage of determining whether or not the proposed pile can meet the design requirements for both bearing capacity and long-term displacement. The approach, however, does not determine specific values of bearing capacity or settlement. This paper uses the PDwDD method to analyze the behaviors of the designed piles and the actual pressed-in piles at a specific project in Binh Duong, Vietnam. The traditional analytical methods used 340 PC piles of 500 mm in diameter and 29 m in length. The final project used 340 PC piles with a diameter of 500 mm and a length of 16.5 m. The article determined whether or not the piles, which were designed with a length of 29 m, and then were reduced to 22 m after the static load test and were finally reduced to 16.5 m after the PDA test, could withstand the building load and satisfy different requirements such as bearing capacity and settlement. The results from the PDwDD method show that the 16.5 m long piles satisfy both bearing capacity and long-term settlement.

Keywords: Negative friction, Down drag, Consolidation, Pile group, Settlement.

1. INTRODUCTION

For a long time, pile design has been mainly based on a combination of empiricism and experience. The first design was theorized by Terzaghi and Peck [18]. There is a view that it is not necessary to improve the pile design method. Many studies have indicated a strong relationship between the real behaviors of piles and the theoretical behaviors of piles despite that the theoretical method holds several limitations Poulos [14] classified three groups of pile desmethodsthod: (1) Experimental methods, which are not based on soil mechanics principles, that use simple or related laboratory and field test results; (2) Methods based on simple theories o diagrams that apply principles of soil mechanics – soil models can be linearly elastic, nonlinear elastic and elastoplastic; (3) Based on the theory of special analysis of the field and principles of soil mechanics - soil models can be linear elastic or rigid plastic; the non-linearity can be allowed in a relatively simple manner or non-linearity is allowed with proper constitutive models of soil behavior

According to Fellenius (2004) [5, 6, 7], practical pile design often does not include settlement calculations. The common understanding is that if the bearing capacity of the pile is satisfied, the settlement is also considered satisfactory. However, this approach is uneconomical and wasteful, and it is not always safe. Therefore, for the design of piles, many issues need attention, such as the long-term distribution of pile shaft resistance and pile toe resistance about the load at the pile head; the drag loads due to shaft negative friction, especially at the position of the neutral plane; the position of the neutral plane on which the shaft friction of the pile changes from negative to positive; the displacement–load relationship at the pile toe; the load distribution in the pile. Furthermore, it is necessary to distinguish settlement caused by external loads from settlement caused by forces other than external loads.

The behavior of subsoil around piles is often neglected in conventional pile design. The subsoil under the foundation raft is always facing the consolidation settlement over time, Settlement can be caused by the superstructure loads, loads of urbanization, or the lowering of underground water. Koerner & Mukhopadhyay stated that a settlement of only a few millimeters of the subsoil can develop a mimum negative friction on the pile [10]. Fellenius found that apart from load acting on a pile, in addition to the direct load from the superstructure distributed to that pile, there is a down drag load that is developed due to soil consolidation settlement. Kitiyodom & Matsumoto [9] used Mindlin's equations to determine the load developed along pile length due to pile-pile, raft-and pile interactions, which causes an increased pile load along pthe ile bodhe Fellenius and Kitiyodom's studies show that the down drag load due to negative friction (or interaction forces) above the neutral plane is always mobilized, while the pile bearing capacity includes the pile toe resistance and positive shaft friction may not be mobilizTwo cases may happen: (i)

Positive shaft friction is mobilized to the maximum when the pile load is equal to or greater ultimate pile bearing capacity, at that time a pile starts to move; (ii) Positive shaft friction is not fully mobilized when the pile load is smaller than ultimate pile bearing capacity, then the pile is considered as over-designed

Fellenius [5, 7] in his paper proposed a unified pile design method emphasizing bearing capacity and settlement analysis simultaneously. According to Fellenius, to design the pile it is necessary to establish (1) the resistance curve and the load curve distributed along the length of the pile and a plane passing through their intersection; and (2) the settlement curve of the soil and the settlement curve of the pile along the length of the pile and a plane passing through their intersection. If these two planes coincide, it is called the “neutral plane”. The calculation, which ensures that these four curves meet at the neutral plane, is very difficult.

The neutral plane is determined at the intersection of the four curves, i.e. the load curve, the load capacity curve, the pile displacement curve, and the settlement curve of the ground. These curves depend on the level of negative friction mobilization and/or the consolidation settlement of the soil.

There is research that suggests values and conditions under which the positive and negative friction can mobilize. The values can vary from 0 to the maximum possible value.

The bearing capacity and displacement of piles, whether determined from the analytical methods or calculated in the computer programs, are only estimated. The reason for this is that the physical-mechanical properties and strength of the soil are not stable like those of other elastic materials.

Besides, there is also very little research on the consolidation settlement of the subsoil under the raft of the pile group. Fellenius has restricted the use of the Unified method to single piles due to the aforementioned problem.

Therefore, the balance of the factors of load, the negative friction, the bearing capacity of the soil, the settlement of the soil, and displacement of the pile at the neutral plane [3] are unrealistic.

To simplify the unified method, Cao Van Hoa [3] proposed the PDwDD method in which the neutral plane where the four curves met at one plane, separated into two planes: the "Settlement Equality Plane (SEP)" where pile settlement curve met soil settlement curve and the "Force Equality Plane (FEP)" when the load curve met the capacity curve, and then compare them.

In practice, the settlement of the soil under the piled raft cannot be accurately estimated due to the settlement method and the determination of external loads acting on the subsoil. Thus, when calculating the friction between the soil and the pile, the settlement of the soil can only be estimated.

Therefore, the PDwDD method does not seek a balance of all factors, e.g. load, resistance, ground settlement, and pile displacement. The method does not seek a unique neutral plane, like the Fellenius' method. This method evaluates the bearing capacity and displacement of piles through the variation of negative frictional mobilization and the potential consolidation settlement.

If the range of value of mobilized friction between the pile and the soil is determined (e.g., from 0 to the value determined by the methods in [7] or the design codes), then the degree of consolidation settlement of the soil between the piles depends on the pressure acting on it. So far there is no theory to calculate this pressure.

If the foundation is shallow then there is an entire load of the superstructure acting on the contact surface between the raft and the bearing soil. The pressure due to the load from the superstructure distributed at any depth in the soil can be determined by the solutions of Boussinesq and Newmark [4, 13], from which the consolidation settlement of the soil can be calculated. If it is a pile group, almost all loads are transmitted through the pile to the bearing soil. Therefore, the pressure at any soil element under the raft can be determined based on pile-soil interaction, then Mindlin's first solution can be used [9]. If it is a pile raft foundation, then the pressure at any soil element is the sum of two components: 1) The pressure due to the portion of the load carried by the raft, calculated using the Boussinesq's solution, and 2) The pressure due to the portion of the load carried by the piles, calculated using Mindlin's solutions.

For the sake of simplicity, an equivalent pressure can be assumed to act on the soil surface and cause consolidation settlement The pressure value varies from 0 to a value given as follows:

i) The value of pressure at the equivalent raft, determined according to Tomlinson [19]. This pressure depends on the length of the piles and the size of the piled raft, but it can be assumed to be equal to about 50% of the pressure due to the entire superstructure load.

ii) It is assumed that at least 50% of the pile load is transmitted to the soil below the neutral plane. The pile moves so little that the effect of the pile-soil interaction becomes insignificant. Therefore, it is reasonable to use an equivalent pressure that is equal to about 50% of the pressure due to the entire superstructure load.

iii) According to the research from about 30 pile group foundations constructed around the world, the proportion of loads carried by raft accounts for about 2 - 50% of total superstructure loads, as mentioned in [2, 11, 15, 16]. Therefore, the equivalent pressure causing consolidation settlement in the pile foundation is at most half the pressure due to the entire load of the superstructure.

A more accurate method to determine the settlement of the ground around the piles involves the use of the Mindlin formula described by Kitiyodom & Matsumoto [9]. This method will be studied in subsequent papers.

2. RESEARCH SIGNIFICANCE

The present research considers pile designing techniques. More particularly, the present research relates to a system for pile designing with consideration of down drag.

With the PDWDD method, the engineer can evaluate the bearing capacity and displacement of piles through the variation of negative/positive frictional mobilization. While the equivalent pressure value is used to calculate consolidation settlement, the locations of SEP and FEP planes. The value of equivalent pressure considers other factors such as urbanization, groundwater extraction, and possible risks during project operation.

This study introduces the concept of a "safe zone" of the pile design because the safety factors in the calculation of pile bearing capacity of pile settlement are not real.

3. ANALYTICAL METHOD

In this article, the Pile Design method that takes into consideration the Down Drag (PDWDD) [3] is employed to verify the piles designed by a conventional method, and the piles used in actual construction. This method matches the force equality plane (FEP) with the settlement equality plane (SEP).

3.1. The Force Equality Plane

The force equality plane is defined as the plane that passes through the intersection of the pile load distribution curve and the pile resistance distribution curve. Figure 1 shows the location of FEP at depth of H_{FEP}.

The total resistance capacity of the pile includes the toe resistance and the shaft resistance. In this study, the resistances were calculated using the three analytical methods based on the subsoil's physical and mechanical properties, the strength properties of the subsoil, and the results from the dynamic penetration test. The results from the above methods were evaluated to determine the load–resistance distribution over the pile length. The pile bearing capacity R_z at depth of z, can be determined by:

$$R_z = (\sum R_{f,} + R_{toe}) - P_{f,z} \quad (1)$$

where: $\sum R_{f,}$ is total pile shaft friction resistance, R_{toe} is toe bearing capacity of the pile, - $P_{f,z}$ is negative friction accumulative to depth z.

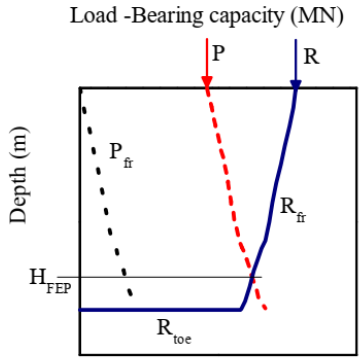

Fig. 1 The Force Equality Plane

The load applied on the top of each pile was determined using Safe software. The load distribution along the pile length can be determined from the load applied on the top of the pile plus the expected negative friction component with the assumption that the soil under the raft is being consolidated. Pile load P_z at depth z, can be determined by:

$$P_z = P + P_{f,z} \quad (2)$$

where: P is the pile load due to superstructure load and $P_{f,z}$ is positive friction.

According to Koerner & Mukhopadhyay [10] the frictional resistance is always mobilized maximum even though there is a slip of only a few millimeters between the pile and the soil. The friction resistance can be determined by various methods described in [7] or by the design codes from different countries.

For greater accuracy, it is advisable to perform strain tests to determine the down drag force, rather than using the less precise analytical methods which are mentioned above. These tests are relatively common nowadays, and they can provide the designer with a reliable negative friction value.

3.2. The Settlement Equality Plane

The settlement equality plane is at the depth where the subsoil settlement curve intersects with the pile displacement curve as displayed in Fig. 2. The figure shows the location of SEP at the depth of H_{SEP}.

The pile displacement curve is a straight line. At the top of the curve is the allowable value, and the tip value is the top value minus the elastic deformation of the pile. The pile displacement $s_{p,x}$ at depth x is determined by:

$$s_{p,x} = s_{p,allow} - s_e \quad (3)$$

where $s_{p,x}$ is the settlement of the pile elements along its length, $s_{p,allow}$ is the allowable settlement at the pile top, and s_e is the elastic deformation of the pile section from the top to the considered depth.

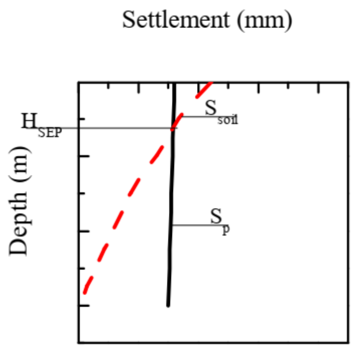

Fig. 2 The Settlement Equality Plane

The settlement of the subsoil below the raft causing negative friction on the piles is the consolidation settlement. In this study, the consolidation settlement occurs because of superstructure loading; this is not affected by other factors such as lowering of the groundwater level or a new embankment. The soil consolidation settlement is determined by:

$$s_{s,x}=\sum_{i=1}^{n}\frac{e_{1i}-e_{2i}}{1+e_{2i}}h_i \quad (4)$$

where n is the number of soil layers to the depth of x, h_i is the thickness of the ith soil layer, e_{1i} is the initial porosity coefficient and e_{2i} is the porosity coefficient after the consolidation of the ith soil layer.

The pressure that causes the consolidation settlement of soil can be calculated more precisely by the method described in [9]. In this study, the pressure is assumed to be equal to 50 - 75 - 100% of superstructure load acting on the interface between raft and subsoil. This assumption aims to find an equivalent pressure that causes soil consolidation settlement in the pile group.

The determination of the portion of the load that causes the pressure acting on the subsoil through the raft is very important to the estimation of the consolidation settlement of the soil. This portion of the load can vary from a few percent to about 50% [2, 11, 15, 16] of the superstructure load. The 50% figure may also be assumed because there are two load-bearing structures in the foundation system: the piles and the raft.

3.3 Pile Design

After constructing the curves and determining SEP and FEP (Fig. 3), the analysis will be based on three possible cases:

- Piles are considered to be capable to carry the external load if the FEP's location is below the location of the SEP, because it's bearing capacity is greater than the load, e.g., $\Delta H = H_{FEP} - H_{SEP} \geq 0$
- Piles are not capable to carrying the load if FEP is above SEP, meaning its bearing capacity is smaller than the external load, e.g., $\Delta H = H_{FEP} - H_{SEP} \leq 0$
- If these two planes coincide, the pile bearing capacity is equal to the external load, in this case these planes are called "Neutral Planes" as per many researchers. Negative and positive friction values
can be determined when the neutral plane is in this equilibrium, e.g., $\Delta H = H_{FEP} - H_{SEP} = 0$

Considering the real lengths of piles used and varying the load pressures acting on the soil surface, one can find the equivalent pressure that causes the subsoil consolidation in a case study

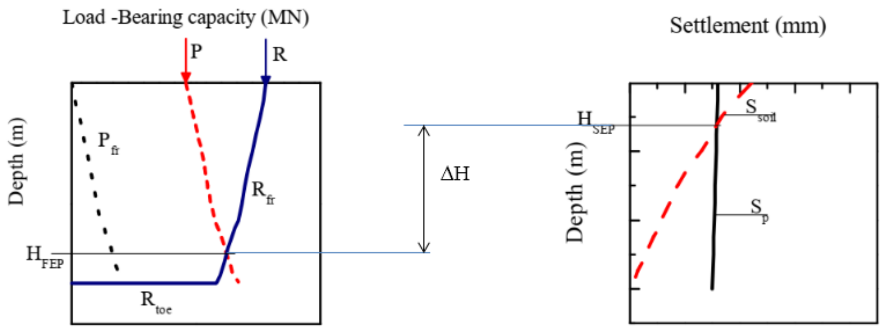

Fig. 3 Pile design and safety gap

4. CASE STUDY - AN APARTMENT BUILDING IN BINH DUONG PROVINCE

4.1. Design Parameters

The case study is an apartment building in Thu Dau Mot City, Binh Duong Province, Vietnam; it has an area of 63 m by 53.6 m and is 43.6 m tall, with 13 floors and 1 basement level. The foundation raft was placed at a depth of -4.5 m from the ground level. Construction started on June 29, 2018 and was completed on December 30, 2019. It was not reasonable to use the press-in method in the case of a thick fine sand layer. In this particular case, the 29 m long piles had to be pressed in through a 20 m thick sand layer. Therefore, when the test piles were pressed in, it was necessary to pre-drill holes; however, the piles could only be pressed into a depth of 21~22 m. Fig. 4 shows the apartment building.

Fig. 4. The studied apartment building in Binh Duong.

The soil investigation was carried out by the SSTCIC company, whose registered office is in Go Vap District, Ho Chi Minh City. The sampling method was rotary drilling combined with bentonite washing. The geological profile was synthesized from three boreholes. In general, there were two soil layers from the bottom of the raft to a depth of -50.5m: Soil Layer 1, from -4.5 to -8.5 m, was sand mixed with gravel, semi-stiff to stiff; Soil Layer 2, from -8.5 to -50.5 m, was fine to medium sand mixed with powder, of medium stiffness. The laboratory tests included direct shear tests, triaxial tests, and consolidation tests, which were carried out to determine nine physical and mechanical properties. SSTCIC engineers suggested the use of a pile foundation with a pile length of about 2130 m, and that the pile toe should be located in the soil layer with an N-value of >20. The groundwater level at the time of soil investigation was at about -8.5 m. Dynamic penetration tests were also performed at all three boreholes. The graph of the N-value with depth is shown in Fig. 5 [17].

Consolidation compression tests were carried out using test equipment from Nanjing Soil WG (China) [17]. The parameters obtained from the tests are shown in Table 1. The test results show that the pre-consolidation stress of the soil layer from 0.00 to -4.00 was greater than the over-burden stress (32 > 22, 9 kPa), meaning that this layer was slightly over-consolidated. From a depth of -4.5 m, the over-burden stress was larger than the pre-consolidation stress. The settlement of the subsoil under this building was calculated as normally consolidated.

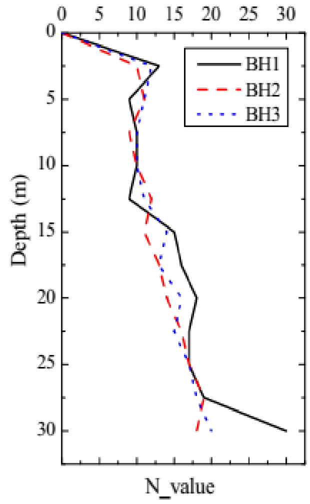

Fig. 5 SPT N-values

Table 1. Consolidation test results

Depth (m)	e_0	p_0 (kN/m^2)	C_c	C_s	γ (kN/m^3)
0	0	0	0	0	0
2.5	0.822	32	0.104	0.012	18.3
5	0.7	41	0.088	0.008	19.3
7.5	0.634	40	0.077	0.01	19.6
10	0.81	60	0.116	0.005	8.8
12.5	0.779	52	0.151	0.014	8.2
15	0.775	63	0.132	0.005	8.3
17.5	0.671	74	0.106	0.005	8.46

4.2. Piled Foundation Design

The foundation of the building was designed according to the traditional analytical method. The load acting on the pile heads was calculated using SAFE software, and the results showed that the maximum load acting on the pile heads was about 1800 kN. Then, the piles were designed to resist the above load according to Vietnamese standard TCVN 10304 - 2014, applying a reliability factor of 2. On that basis, the designer proposed the use of 340 PC piles with a diameter of 500 mm and a length of 29 m for the entire foundation of the building. The press-in method was proposed for the piling work [12].

Fig. 6 shows the load distribution curve (including the expected negative friction) and bearing capacity distribution curves corresponding to pile lengths of 29.0, 22.0, and 16.5 m. From this, it was possible to determine the force equality planes FEP 1, FEP 2, and FEP 3; the planes pass through the intersections of the curves corresponding to each pile length as mentioned above.

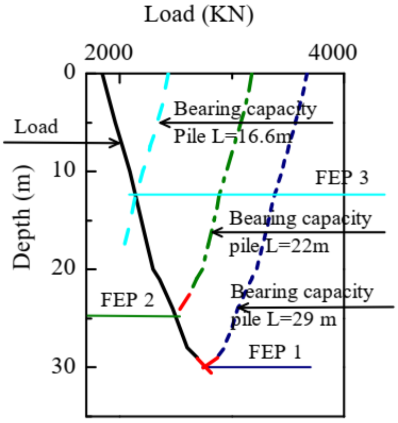

Fig. 6 Force equality planes

Figure 7 shows the expected SEP of the piles when the settlement of the subsoil is estimated with the assumption that the consolidation pressure corresponds to the total building load and the displacement of the pile is within the allowable limit. The figure also shows that the short-term displacement of the static loading test piles [20] and the PDA test piles [1] are insignificant.

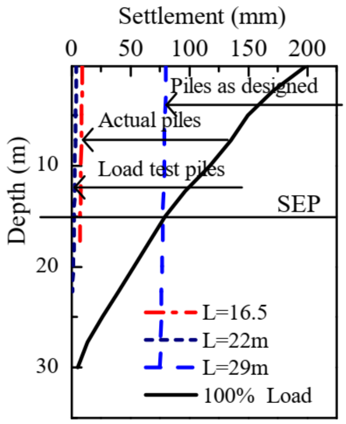

Fig. 7 Settlement equality planes

Three static load test piles were carried out before starting the project, but they could not be pressed into the design depth of -29 m. So, before pressing in, the holes of 400 mm in diameter and 15 m long had to be pre-drilled; however, the piles could only be pressed into the depth of -21, -24, and -21 m. The static load test was performed in two cycles: 1) the load is increased up to 100% of the design load and 2) the load is increased up to 200% of the design load. The displacement values at 100% of the design load were 7 mm, 4.5 mm, and 4 mm. The displacement values at 200% load were 21.72 mm, 14.06 mm, and 13.84 mm, respectively. After removing the test load, the rebound displacements of 5.7 mm, 3.52 mm, and 4.9 mm, were observed. It can be seen that the bearing capacity of the 21 m long pile was 20-25% larger than the capacity expected by the design.

During the mass pressing in, the piles can only be pressed to a depth of 16-18 m. Therefore, the contractor proposed to carry out additional PDA testing for two pile lengths, 16.3 and 16.5 m. The test equipment used was a Pile Analyzer (PDA), PAK brand, Institute of Kinetics, USA. The recorded total resistance values were 3650 kN (3103 kN as frictional resistance and 547 kN as toe resistance) for the 16.3 m pile and 3950 kN (3420 kN as frictional resistance and 530 kN as toe resistance) for the 16.5 m pile. This shows that the 16.5 m long pile still meets the design requirements in terms of bearing capacity.

After completing the concrete framework, the settlement monitoring was carried out [8]. The settlement monitoring results on May 7, 2020 recorded a very small settlement; the largest was 5.4 mm. This shows that reducing the pile length from 29 m according to the original design to 16–18 m in the construction phase still ensured the load capacity of the pile group. It has now been more than 3 years since the work was completed, and no excessive displacement has been recorded. This shows that the 16.5 m long piles also satisfied the design requirements in terms of displacement.

4.3. Discussion

Piles must have a length of 29-31 m to ensure that the load-bearing capacity can meet the requirement of the traditional analytical design methods. Fig. 3 shows that FEP 1, which passes through the intersection of the load curve (including building load and negative friction force) and the pile bearing capacity curve, estimated based on traditional analytical methods, is located at a depth of -30 m. Meanwhile, the location of the SEP, as shown in Fig. 7, where the consolidation settlement was estimated at a pressure of 100% of the building load, is at a depth of about -15 m. It can be said that a pile length of 29 m exceeds the design requirements of the load-bearing capacity as well as the allowable displacement requirements.

The pile length was reduced to 22 m during the static load tests; the test results show that the pile had a bearing capacity of greater than 1800 kN, which completely meets the design requirements for bearing capacity. Figure 6 shows the plane FEP 2, located at a depth of -25 m, while the SEP plane, in Fig. 7, lies at a depth of -15 m. Thus, the 22 m long pile, like the 29 m long pile, has a bearing capacity exceeding the design requirements and satisfies the allowable displacement.

The bearing capacity of the 16.3 m long and 16.5 m long piles according to the PDA test was 1800 kN, satisfying the design requirements in terms of load-bearing. Fig. 6 shows the FEP 3 plane, determined by traditional analytical methods, located at a depth of -12 m. Fig. 7 shows the SEP plane, estimated with the assumption that the subsoil is consolidated by a pressure corresponding to 100% of the building load, located at a depth of -15 m. This means that even though a 16.5 m long pile may, in the short term, have sufficient load-carrying capacity and meet the required displacement, in the long term, the displacement of the pile may be greater.

On the other hand, the largest settlement of the building according to settlement monitoring in 2019 and 2020 was only 5.4 mm. That means that the 16.5 m long piles used for this building completely meet the design requirements.

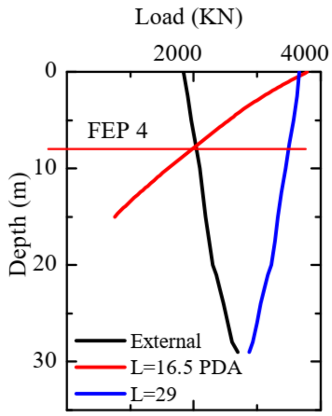

Fig. 8 Speculated force equality plane of the PDA test piles, L = 16.5 m.

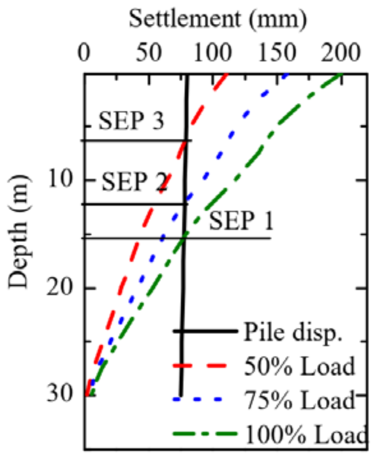

Fig. 9 Speculative SEPs for different loads acting on the soil

Fig. 8 shows the force equality plane (FEP 4) of a 16.5 m long pile located at a depth of about -7.5 m. Thus, to design a pile 16.5 m long that meets the requirements of bearing capacity and settlement, the settlement equality plane must be speculatively located at a depth of less than 7.5 m. Fig. 9 shows the positions of settlement planes at depths of 15, 12, and 7 m corresponding to 100%, 75%, and 50% of the building load used to estimate consolidation settlement. This shows that for the long-term displacement of the 16.5 m long pile to satisfy the permissible value, the load used in the calculation of consolidation settlement must be less than 50% of the building load. This shows that the calculation of the consolidation settlement of the subsoil using 100% of the building load, in this case, is too conservative.

5. CONCLUSIONS

From the above analysis, the following conclusions can be drawn:

The results of the PDWDD method and the monitoring data show that after reducing the pile length from 29.5 m, the 16.5 m long piles are capable of carrying the required design load and meeting the allowable settlement in the long term. However, the traditional analytical design method, static load test, and PDA test did not determine the long-term settlement of the pile, therefore it is not clear whether the pile after reducing its length to 16.5 m will settle in the future or not.

It is recommended that: (1) pile strain tests should be carried out to determine more precise frictional forces, and (2) a reasonable equivalent load should be determined to limit the uncertainty in the calculation of the consolidation settlement of the soil when applying the PDWDD method.

The concept of the equivalent distributed load is proposed to be used in the consolidation settlement estimate. This distributed load acting on the surface of subsoil may be proportional to the load from the superstructure. In this study, it appears to be less than 50% of the superstructure load. This load must also be greater than the portion of the superstructure load transmitted through the raft to the soil since the portion of the superstructure load transmitted through the piles also causes consolidation settlement due to interaction effects. Therefore, the load used to estimate consolidation settlement may be greater than the load assigned to the raft. The load used in consolidation settlement estimation may be as large as 50% of the total superstructure load. However, more in-depth studies are needed to account for the uncertainty when calculating the consolidation settlement of the subsoil under buildings.

In this study, the consolidation settlement is assumed to be only affected by superstructure load. Other factors that may influence consolidation settlement such as lowering of the groundwater level or a new embankment are not considered.

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