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Estimation of static jacking load (P_{ep}) for the prestressed centrifugal concrete piles on some types of ground structure in Hai Duong city

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ABSTRACT

Nowadays, the prestressed centrifugal concrete piles (PCCP) are applied in most major construction projects in Hai Duong city. The pile bearing capacity and static jacking load (P_{ep}) are very important parameters for optimizing the pile foundation design, and they depend on the types of ground structure. The paper introduces the estimated method for the pile static jacking load which was applied for two types of ground structure in Hai Duong city (type I and type II). The results showed that the maximum static jacking load (P_{epmax}) was 66.6 tons and 76.6 tons at the depth of 32 m for types I and II, respectively. The results of evaluation and comparison between the calculated P_{ep} with the actual P_{ep} have provided the penetration coefficient (K) for each soil: K = 1.8 for fine sand; K = 1.5 for stiff sandy clay; K = 1.4 for firm sandy clay and K = 1.2 for soft soil.

1. Introduction

Hai Duong city is located at the Red River delta with a wide distribution of soft soil aged Holocene at the shallow depth (Le Hong Quan, 2009). Currently, the demand for urban construction in Hai Duong city is on the rise. Since the presence of soft soil is at shallow depth, the pile foundations are often used for most construction works. In particular, the prestressed centrifugal concrete piles (*PCCP*) have been widely used.

Foundation design needs to be optimized, technically met. With the pile foundation, these are two following technical requirements. Firstly, the pile is not broken during transportation or ground driving. Secondly, both the settlement and the stability of pile foundations are within allowed limits.

In which, the static jacking load of the pile (P_{ep}) is an important factor that helps to estimate the bearing capacity of the pile (P_{dn}) as well as determine the maximum jacking load (P_{epmax}). The P_{dn} value is used in pile foundation design, while the P_{epmax} value is used to execute the pile

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manufacturing and the pile driving at the site. Normally, the parameter P_{epmax} must be smaller than the bearing capacity of the pile material (P_{vl}). The estimation of P_{ep} is especially necessary in the case of stratigraphy with intermediate soil layers. In some practical cases, P_{ep} in the intermediate soil layers may be larger than the design value, causing the pile to break. Therefore, it is necessary to estimate P_{epmax} in the actual strata as the basis for P_{vl} design.

The *PCCP* is prefabricated with a fixed strength for each type of pile. However, in the actual construction in Hai Duong, there are many cases that the bearing capacity of the *PCCP* is much higher than P_{epmax} and $P_{đn}$. This problem causes significant waste.

In general, $P_{đn}$ and P_{ep} can be evaluated by the results of in-situ tests (Ardalan et al., 2009; Kempfert H., Becker P., 2010) or laboratory tests (Bogumił Wrana, 2015; TCVN 9394:2012, 2012). However, currently, there is no research on the estimation of pile static jacking load, especially for the soil ground structure in Hai Duong city. Thus, in this study, the estimation of pile static jacking load for *PCCP* on some typical types of ground structure in Hai Duong city will be conducted.

2. Method for estimating P_{ep}

Theoretically, the bearing capacity of the pile ($P_{đn}$) is the sum of tip resistance (Q_c) and total friction (Q_s) (Prakash Shamsher, D, Sharma Hari, 1999). The description of Q_c and Q_s is shown in Figure 1. The static jacking load P_{ep} must overcome $P_{đn}$ for the pile to be driven into the ground at a certain depth so that $P_{ep} = K.P_{đn}$. Then, the P_{ep} will be calculated by the following formula:

$$P_{ep} = K(Q_c + Q_s) \tag{1}$$

In which: K - actual coefficient; Q_c - tip resistance in total; Q_s - total sleeve friction of the soil around the pile.

The parameters Q_c and Q_s are calculated by using the expressions (2) and (3) (Ruwan Rajapakse, 2016).

$$Q_c = A.q_c \tag{2}$$

$$Qs = U \sum f_{si} l_i \tag{3}$$

In which: A - cross-sectional area of the tip pile; U - circumference of the cross-section of the pile; l_i - thickness of the soil layer; q_c - tip resistance; f_{si} - sleeve friction of the soil.

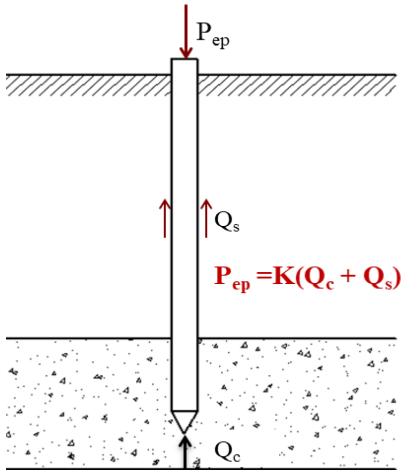

Figure 1. Diagram of estimated P_{ep}.

The values of q_c and f_s can be determined from the cone penetration testing (CPT) or estimated by the results of the laboratory tests, as follows (Terzaghi, 1943):

$$q_c = \gamma.d.N_\gamma + 0{,}5\sigma'_z N_q + C_u.N_c \tag{4}$$

$$q_c = \gamma.d.N_\gamma + 0{,}5\sigma'_z N_q + C_u.N_c \tag{5}$$

In which: γ - unit weight (kN/m^3) ; C_u - soil cohesion (kN/m^2); φ_u - internal friction angle of the soil (degree); N_γ, N_q, N_c are coefficients depending on φ_u; σ'_z - effective stress at a depth of pile toe (kN/m^2).

The shear strength parameters (C_u, φ_u) are determined in an undrained condition corresponding to the pile pressing process.

Using Terzaghi's bearing capacity equation, N_γ, N_q, N_c are determined as follows (Roy Whitlow, 2000):

$$N_q = e^{\pi tg\varphi_u} tg^2(45^o + \frac{\varphi_u}{2}) \tag{6}$$

$$N_c = Cotg\varphi_u(N_q - 1) \tag{7}$$

$$N_\gamma = 1{,}8(N_q - 1)tg\varphi_u \tag{8}$$

In equation (1), the coefficient K depends on the soil type and the penetration speed, usually equals 1.5÷2. It should be noted that when driving the pile through clayey layers, it often leads to the appearance of excess pore water pressure (ΔU), which decreases the soil shear strength. In this case, K should be chosen at a low threshold. By contrast, driving the pile in sandy soil will make soil denser and increases the soil shear strength. In this case, K should be chosen at a high threshold.

Thus, we can use the expression (1) to estimate P_{ep} at different depths of the pile. It is necessary to estimate P_{ep} in each meter of the pile according to Vietnamese standards (TCVN 9394: 2012).

3. Research results and discussions

3.1. The characteristics of ground structure in the study area

According to Le Hong Quan (2009) and Do Hong Thang (2011), there are two popular types of ground structure in Hai Duong city: type I is characterized by soft soil distributed from the ground surface to the depth of 25÷33 m, and covered on the sand layer (medium dense - fine sand). Type I is most widely distributed in Hai Duong city area, including Pham Ngu Lao, Tran Hung Dao, Binh Han, Ngoc Chau, Nhi Chau, Cam Thuong, and Quang Trung wards; type II is sandwiched between soft soil and hard soil, the arrangement order is soft soil - hard soil - soft soil - hard soil, narrowly distributed in the center of Hai Duong city with a radius of about 1.5 km^2.

On the basis of the ground structure characteristics described above, two research locations are selected to characterize the ground structure of two types: The first study site is in Truong An urban area (*VT1*), representing type I; The second study site is in Le Thanh Nghi ward (*VT2*), representing type II.

The stratigraphic characteristics and soil properties are listed in Tables 1 and 2. In which, the shear strength of the soil is determined in undrained conditions for clayey soils and in saturated conditions for sandy soils. The groundwater levels in both study sites have an average depth of 2 meters.

3.2. Results of estimating P_{ep}

Piles used in the two study sites are *PCCP* with Φ300 produced by Kien Hoa Dat Viet Joint Stock Company (Figure 2) with the capacity of 120 tons. P_{ep} at these two sites is calculated by using equation (1).

Table 1. The characteristics of soil layers in VT1.

Layers	Depth of distribution (m)	Soil type	Soil properties		
			Unit weight, γ (kN/m^3)	Cohesion force, C_u (kN/m^2)	Internal friction angle φ_u (degree)
1	0.0÷0.4	Fill soil	-	-	-
2	0.4÷4.2	Firm, yellowish grey sandy clay	18.6	16.1	7.0
3	4.2÷22.6	Mud of darkish grey sandy clay	16.4	6.2	2.0
4	22.6÷28.0	Soft, darkish grey sandy clay	18.8	10.2	4.0
5	28.0÷32.0	Stift, yellow grey sandy clay	19.2	20.8	10.0
6	32.0÷38	Medium dense, fine sand	19.0	0	30.0

Table 2. The characteristics of soil layers in VT2.

Layers	Depth of distribution (m)	Soil type	Soil properties		
			Unit weight, γ (kN/m^3)	Cohesion, C_u (kN/m^2)	Internal friction angle, φ_u (degree)
1	0.0÷3.5	Fill sand	18.0	0	25.0
2	3.5÷4.8	Firm, yellowish - brownish grey sandy clay	18.1	15.4	8.0
3	4.8÷6.0	Soft, brownish grey sandy clay	18.4	11.1	3.0
4	6.0÷11.6	Loose fine sand	17.8	0	25.0
5	11.6÷27	Mud of grey sandy clay	16.3	06.3	1.0
6	27.0÷34.0	Medium dense, fine sand	19.0	0	27.0

Figure 2. The PCCP piles with Φ300.

Accordingly, the calculation of P_{ep} is performed for each meter of the depth. The total sleeve friction Q_s is calculated by the formulas (5) and (3). The total of tip resistance Q_c is calculated by the formula (4) with the coefficients N_γ, N_q, N_c determined by the formulas (6), (7), (8) for the soil layer at the considered depth. The results of P_{ep} estimation in soil layers are summarized in Table 3.

Table 3. Results of P_{ep} estimation.

Layers	Penetration Coefficient, K	The average of P_{ep}/ The maximum of P_{ep}, $(P_{ep})_{av}/(P_{ep})_{max}$ (Tons)
VT1 Site		
2	1.4	8.1/12.6
3	1.2	19.7/27.0
4	1.2	32.2/35.4
5	1.5	58.3/66.6
6	1.8	91.7/97.6
VT2 Site		
1	1.8	4.8/7.5
2	1.4	8.3/8.3
3	1.4	11.8/8.3
4	1.8	33.1/44.4
5	1.2	29.6/35.6
6	1.8	73.8/82.3

The estimated results of P_{ep} depend on not only the pile size and soil properties but also the coefficient *K*. In general, *K* is selected according to the soil type as mentioned in Section 2. However, it is necessary to compare the calculation results with the actual data of pile jacking load at the site. Since then, *K* is selected suitably for each soil type.

The hydraulic static pile driver ZYC180BS - B type (T - Work, China) was used in both study sites. The driving works are done according to TCVN 9394 - 2012 standard. During the pile driving process, the actual jacking loads and speed parameters of the pile are recorded for each meter of depth.

Figure 3. The pile driver ZYC180BS - B.

The estimated results of P_{ep} and the actual data of pile jacking load are shown in Figure 4. From these charts, it can be seen that the coefficient *K* for each soil type, as shown in Table 3, is quite suitable. In which, *K = 1.8* for fine sand; *K = 1.5* for stift sandy clay; *K = 1.4* for firm sandy clay and *K = 1.2* for soft soil.

When designing the friction pile foundation, the designer usually drives the pile into the bearing soil layer of 5÷10 m, depending on the load. The estimated value of P_{epmax} at a depth of 37 m (in bearing soil layer of 10 m) is 97.6 tons in VT1 and 90.7 tons in VT2. However, in fact, it is not recommended to design the Φ300 pile beyond the depth of 32 m to ensure the requirement of pile limited slenderness. The value of P_{epmax} at a depth of 32 m is 66.6 tons at VT1 and 76.6 tons at VT2.

Figure 3b shows that the actual jacking load at the depth of 5÷10 m is 30% higher than the calculated values. The reason is that the density of loose sand increases significantly while driving the pile in sand.

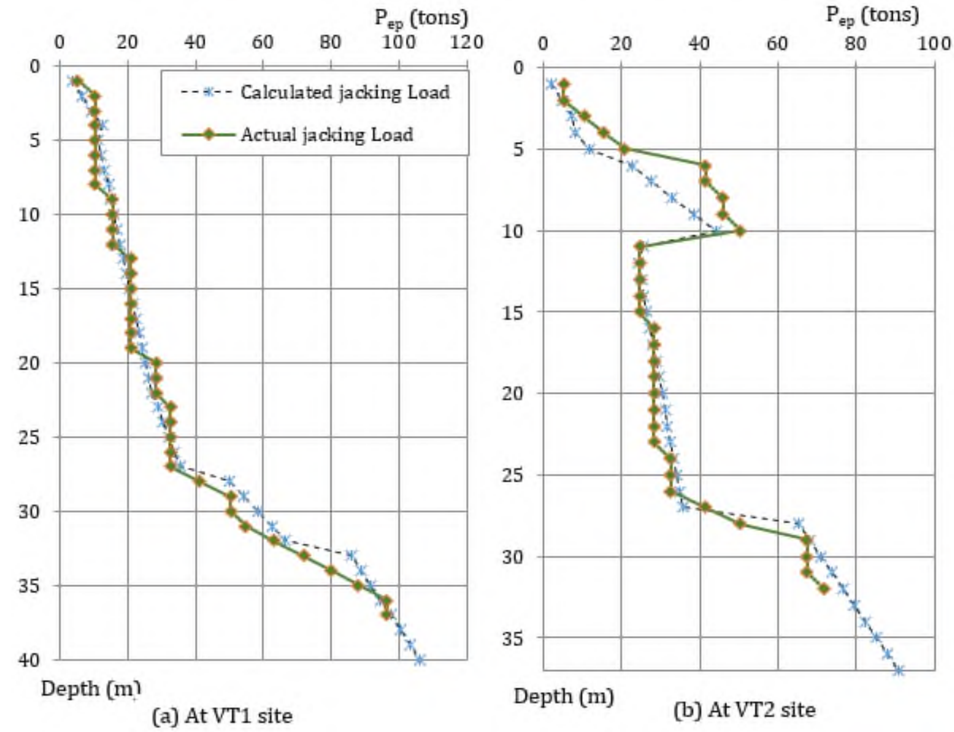

Figure 4. Calculated and actual jacking loads versus the depth at two study sites.

4. Conclusions and recommendations

The results received above lead to the following conclusions and recommendations:

When performing static driving of PCCP pile (Φ300) to the depth of 32 m in Hai Duong city, P_{epmax} is 66.6 tons in type I (VT1) and 76.6 tons in type II (VT2). In-ground structure type I, P_{ep} increases gradually with the depth of the pile. While in type II, P_{ep} reaches 50 tons when driving the pile through the middle fine sand layer (layer 4) at a depth of 6÷10 m.

The coefficient K is selected suitably for each soil type in the study area as follows: K = 1.8 for fine sand; K = 1.5 for stift sandy clay; K = 1.4 for firm sandy clay and K = 1.2 for soft soil.

Depending on the design depth of the pile, the estimated P_{ep} value can be used to prepare the jacking machines and counterweight. The P_{epmax} values should be used as a basis for pile manufacturing to ensure both technical and economical requirements.

Author contributions

The author Do Hong Thang proposes ideas and contributes to the manuscript. The author Nguyen Van Phong constructs the manuscript and contributes to the material analyses. The authors both declare no conflict of interest.

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