Research Article

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Bearing capacity of floating geosynthetic encased columns (GEC) determined on the basis of CPTU penetration tests

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CPTU

cone penetration test

GEC

geosynthetic encased column

Abstract: Floating geosynthetic encased columns (GEC) are an increasingly popular method of strengthening weak subsoil. Design of floating columns is a difficult and not fully recognized issue. This paper treats the floating GEC column as a special kind of “pile” and its bearing capacity is calculated using five selected methods for calculating the bearing capacity of piles based on CPTU penetration tests. The calculations were done on the basis of in-situ tests carried out on one of the sections of the Bargłów Kościelny bypass. The paper contains a comparison of the bearing capacities of floating GEC columns calculated with different methods based on CPTU penetration tests.

Keywords: GEC columns, bearing capacity of piles, CPTU penetration tests, floating columns

1 Introduction

A floating column is a column with a base in weak subsoil. Floating columns are an increasingly popular method of strengthening weak subsoil, mainly for economic and technological reasons [1]. Guidelines for the design of reinforcement for geosynthetic encased columns (GEC) [2] apply only to end-bearing columns, and there are no guidelines for designing reinforcement for floating columns.

The floating GEC column can be treated as a special kind of “pile” and its bearing capacity can be calculated with methods based on CPTU penetration tests [3].

Main Nomenclature

A_k	column base area
A_s	column shaft area
c_u	undrained shear strength
D_k	column diameter
f_p	shaft resistance
f_s	sleeve friction
I_L	liquidity index
Q_{bk}	column base capacity
q_b	unit end-bearing pressure
q_c	cone resistance
Q_k	column bearing capacity
Q_{sk}	column shaft capacity
q_t	corrected cone resistance
z_k	depth of the column base

2 Methods of calculating the bearing capacity of piles based on CPTU tests

In this paper, the bearing capacity of the column was determined by five selected methods: LCPC [4], Beringen and De Ruiter [5], DIN 4014 [6], Schmertmann [7] and Philipponnat [8].

Pile bearing capacity is the sum of the base capacity Q_{bk} and shaft capacity Q_{sk} [9]:

$$Q_k = Q_{bk} + Q_{sk} \quad (1)$$

$$Q_{bk} = q_b A_k \quad (2)$$

$$Q_{sk} = f_p A_s \quad (3)$$

where A_k and A_s are the area of the pile base and pile shaft respectively, q_b is the unit end-bearing pressure and f_p is the average shaft resistance.

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2.1 LCPC method

In the Laboratoire Central des Ponts et Chausees (LCPC) method, the unit end-bearing pressure and average shaft resistance can be calculated directly from the tip cone resistance q_c [4].

$$q_b = k_c q_{c,avg} \quad (4)$$

$$f_p = \frac{q_{c,z}}{\alpha'} \quad (5)$$

where k_c , α' are coefficients that depend on the pile and soil type, $q_{c,avg}$ is the equivalent average cone resistance between $1.5D_k$ below and $1.5D_k$ above the pile tip, D_k is the diameter of the pile and $q_{c,z}$ is the cone resistance at depth z_k .

The value $q_{c,avg}$ should be calculated in three steps:

- Calculate q_{ac} as a mean value of q_c at a depth between $1.5D_k$ below and $1.5D_k$ above the pile tip.
- Eliminate q_c values higher than $1.3q_{ac}$ and lower than $0.7q_{ac}$.
- Calculate $q_{c,avg}$ within the range defined in the previous step.

2.2 Beringen and De Ruiter method

In the Beringen and De Ruiter method, also known as the European method, the values of q_b and f_p in Equations 2 and 3 should be determined from the following equations [5]:

$$q_b = 9c_u \quad (6)$$

$$f_p = \beta c_u \quad (7)$$

where $\beta = 1$ for normally consolidated soils, $\beta = 0.5$ for over consolidated soils and c_u is the undrained shear strength determined from Equation 8 [10].

$$c_u = \frac{q_t - \sigma_{v0}}{N_{kt}} \quad (8)$$

where q_t is the corrected cone resistance, σ_{v0} is the total initial stress at the considered depth, N_{kt} is a coefficient depending on the plasticity index I_p and the degree of soil consolidation. N_{kt} values are usually in the range of 10 to 20 [11].

2.3 DIN 4014 method

In the DIN 4014 method, the shaft resistance f_p and the unit end-bearing pressure q_b in cohesive soils are determined from the undrained shear strength c_u .

The value of c_u can be determined from Equation 8, using an average q_c value over a zone of three times the pile diameter under the tip.

2.4 Schmertmann method

In the Schmertmann method, the value of q_b is determined from Equation 9 [7].

$$q_b = \frac{q_{c1} + q_{c2}}{2} \quad (9)$$

where q_{c1} is the minimum average value of the cone resistance at a depth of $0.7D_k$ or $4D_k$ below the pile tip and q_{c2} is the minimum average value of the cone resistance in the zone equal to $8D_k$ above the pile tip.

For piles with a base in cohesive soils, the f_p value can be determined from Equation 10 [7].

$$f_p = \alpha_c f_s \quad (10)$$

where f_s is the sleeve friction and α_c is a coefficient selected in the range 0.2 to 1.25, depending on the sleeve friction and the type of pile. For values of f_s equal to 30 kPa and less, the value of the coefficient α_c is the same for all types of piles.

2.5 Philipponnat method

In the Philipponnat method, the values of q_b and f_p can be determined from Equations 11 and 12 [8].

$$q_b = k_b q_{ca} = k_b \frac{q_{ca(A)} + q_{cb(B)}}{2} \quad (11)$$

$$f_p = \frac{\alpha_s}{F_s} q_{cs} \quad (12)$$

where k_b is a coefficient that depends on the type of soil in which the pile tip is located (for clays, $k_b = 0.5$), $q_{ca(A)}$ is the average cone resistance in the zone equal to $3B$ above the pile tip, $q_{cb(B)}$ is the average cone resistance at a depth of $3B$ below the pile tip, F_s is a coefficient depending on the type of soil in which the pile tip is located (for clays, $F_s = 50$), α_s is a coefficient equal to 1.25 and q_{cs} is the average cone resistance along the pile shaft. For circular foundations $B = B'$ and a B' value can be calculated from Equation 13.

$$B' = \frac{1}{2} \sqrt{\pi} D_k \approx 0.886 D_k \quad (13)$$

3 Calculation arrangements

The GEC column is treated as a special kind of “pile” and its bearing capacity is calculated using five selected meth-

ods for determining the pile bearing capacity based on CPTU tests.

This paper considers the characteristic value of the bearing capacity Q_k (Figure 1) of a single column for reinforcing weak subsoil for plans with large dimensions (road embankments, spatial structures). Negative friction was omitted in the calculations because the reinforced soil mattress on the subsoil causes settlement of the column and the weak subsoil surrounding the column to be equal. The calculations were carried out for columns with diameter D_k equal to 0.8 m.

Figure 1: Geometry of the problem.

The bearing capacity of floating GEC columns was determined on the basis of CPTU tests carried out on one of the sections of the Bargłów Kościelny bypass, where a weak subsoil was reinforced with floating GEC columns.

On the analyzed section of the Bargłów Kościelny bypass, there are organic soils, mainly peat with a thickness of from about 3 m to about 9 m. Below the organic soils there are glacial deposits in the form of sandy clays. On top of the layer of glacial deposits, the clays are soft. With depth, the moisture of the cohesive soils decreases, and thus the liquidity index decreases, dropping to a value corresponding to stiff clay. The ground water level is located at a depth of 0.2–0.3 m below the ground surface.

The relationships between the liquidity index I_L of sandy clay and the cone resistance q_c , undrained shear strength c_u and sleeve friction f_s were determined on the basis of the CPTU penetration tests (Figure 2).

Bearing capacity of the columns was calculated for soil conditions determined in four selected locations located on the section of Bargłów Kościelny bypass un-

Figure 2: Relationships between: a) $q_c - I_L$, b) $c_u - I_L$, c) $f_s - I_L$.

der consideration. The analyzed locations were spaced about 30 m apart. The calculations were made for different depths of columns in the weak subsoil z_k (Figure 1) equal to 0, 0.75, 1.5 and 3.0 m for location 1 and 4; 0, 0.75, 1.5 and 2.5 for location 2; and 0, 0.75, 1.5 and 2.0 for location 3.

Different values of the maximum depth for locations 2 and 3 are due to the fact that a depth of 3 m would mean that the tip of the column would be located in stiff soil and the column would not be a floating column.

Figure 3 presents the results of the CPTU tests in the four selected locations located on the section of the Bargłów Kościelny bypass under consideration and the

Figure 3: Results of CPTU tests: a) location 1, b) location 2, c) location 3, d) location 4.

thickness of the layers of peat and sandy clay with a variable liquidity index I_L .

CPTU tests were carried out to a depth of about 12 m, which would be insufficient to make calculations based on the cone resistance graphs; therefore a forecast of the liquidity index was made to a depth of 20 m (Figure 3).

Table 1: Parameters of the sandy clay determined on the basis of the CPTU penetration tests.

Degree of plasticity	Cone resistance	Sleeve friction	Undrained shear strength
I_L	q_c	f_s	c_u
[-]	[kPa]	[kPa]	[kPa]
0.58	676.1	11.05	30.10
0.42	1193.4	23.67	56.98
0.31	1679.9	34.67	81.90

Table 1 shows the values of the cone resistance, sleeve friction and undrained shear strength from Figure 2 for different liquidity indexes of the sandy clay located at the tip of the floating GEC column.

4 Bearing capacity of the floating GEC columns

Figure 4 shows the bearing capacity of the floating columns at different depths of the column tip in the soft sandy clay for the five selected calculation methods.

Figure 4: Bearing capacity of the floating GEC columns: a) location 1, b) location 2, c) location 3, d) location 4.

5 Conclusions

From the calculations, comparable values of the column bearing capacity were obtained for the five calculation methods analyzed in this paper. Differences in the values for the different calculation methods are due to the different assumptions on which the methods were based.

In all the analyzed locations, the lowest bearing capacity of the column was determined using the LCPC method. The highest bearing capacity was calculated using the European and Schmertmann methods, and for location 3, using the DIN 4014 method.

The bearing capacity of the columns increased with increasing depth of the column tip in sandy clay. A more than three times higher bearing capacity that varied with the depth was observed in location 1.

The highest bearing capacity was determined for the columns in location 3, while the smallest was for the columns in location 1. This can be related to the soil conditions in each individual location.

The methods of determining the bearing capacity of floating GEC columns described in this paper do not allow the distribution of the load in the columns and weak subsoil to be included in the calculations; thus, it does not take into account cooperation between the column and the surrounding soil. However, the author believes that these methods can be successfully used to pre-estimate the bearing capacity of floating GEC columns based on the results of CPTU penetration tests.

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