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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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

Main Nomenclature

| Symbol | Description |
|--------|-------------|
| $A_k$  | 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 |

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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].

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 Philippounnat [8].

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

\[ Q_k = Q_{bk} + Q_{sk} \]  

\[ Q_{bk} = q_b A_k \]  

\[ Q_{sk} = f_p A_s \]

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.
2.1 LCPC method

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

\[
q_b = k_c q_{c,\text{avg}}
\]  

\[
f_p = \frac{q_{c,z}}{\alpha'}
\]  

where \( k_c, \alpha' \) are coefficients that depend on the pile and soil type, \( q_{c,\text{avg}} \) 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 \( q_{c,\text{avg}} \) should be calculated in three steps:

• Calculate \( q_{ca} \) 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 \( 1.3q_{ca} \) and lower than \( 0.7q_{ca} \).
• Calculate \( q_{c,\text{avg}} \) 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]:

\[
q_b = 9c_u
\]  

\[
f_p = \beta c_u
\]  

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].

\[
c_u = \frac{q_t - \sigma_{vo}}{N_{kt}}
\]  

where \( q_t \) is the corrected cone resistance, \( \sigma_{vo} \) 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].

\[
q_b = \frac{q_{c1} + q_{c2}}{2}
\]  

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].

\[
f_p = \alpha_s f_s
\]  

where \( f_s \) is the sleeve friction and \( \alpha_s \) 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 \( \alpha_s \) 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].

\[
q_b = k_b q_{ca} = k_b \frac{q_{ca(A)} + q_{cb(B)}}{2}
\]  

\[
f_p = \frac{\alpha_s}{F_s} q_{cs}
\]  

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, \( F_s = 50 \), \( \alpha_s \) 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.

\[
B' = \frac{1}{2} \sqrt{\pi D_k} = 0.886D_k
\]  

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.](image)

The bearing capacity of floating GEC columns was determined on the basis of CPTU tests carried out on the section 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 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 under 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
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 $I_L$ | Cone resistance $q_c$ [kPa] | Sleeve friction $f_s$ [kPa] | Undrained shear strength $c_u$ [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.
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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