AXIAL PILE CAPACITY: PREDICTED VERSUS MEASURED RESPONSE IN SOUTHERN ALBERTA CLAY TILL

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ABSTRACT
A full scale pile load test on a 1.4m diameter, 14.1m long cast-in-place concrete bored pile was carried out as part of the foundation design for the Calgary Foothills Medical Centre (FMC). The load test provided an opportunity to compare pile design techniques to measured pile performance. The test pile was instrumented with strain gauges and subsequently loaded to failure with an Osterberg Cell (O-Cell). The test pile was founded in clay till. Using the load test data, the axial pile response was back analyzed. This paper compares the predictions from six axial pile capacity methods with the results obtained from the full-scale pile load tests. The prediction methods used data obtained from a series of piezocone penetration (CPTu) and flat plate dilatometer tests (DMT). Each predictive method is summarized and the most applicable method of analyses is identified.

RÉSUMÉ
Dans le cadre du dessein de fondement pour le Centre Médical Foothills de Calgary, un essai de chargement en vraie grandeur de pieu a été effectué sur un pieu moulé en béton avec un diamètre de 1.4 m et une longueur de 14.1 m. L'essai de chargement a fourni l'opportunité de comparer les techniques du dessein de pieu à des performances de pieux calculées. L'essai de pieu a été équipé avec des jauges de contrainte et subéquemment chargé à panne avec une Cellule Osterberg. L'essai de pieu était fondé en argile rigide. Les données de l'essai de chargement ont été utilisées dans l’analyse retour de la réponse du pieu axial. Cet exposé compare les prédictions de six méthodes de capacité de pieu axial aux résultats obtenus durant l'essai de chargement en vraie grandeur de pieu. Les méthodes de prédiction ont utilisé les données obtenues d'une série de tests in situ au piézocône (CPTu) et dilatomètre (DMT). Chaque méthode pronostique est résumée et la méthode d’analyse la plus applicable est identifiée.

1 INTRODUCTION
The prediction of pile capacity is a complicated task due to variations in soil types and behaviour, pile types and shapes, construction methods and project limitations. Many design methods and approaches have been proposed in the literature in order to accommodate these variable conditions. These design methods are well established in the geotechnical community. They incorporate large empirical data sets, site investigation data, laboratory tests, and full scale load tests. However, the applicability of each method varies with site conditions. It is recommended that the geotechnical engineer should consider a variety of solutions along with local knowledge before identifying the best predictive method for a given set of conditions (Lunne et. al 1997). This paper compares the results of six principal pile design methods against an instrumented pile load test in Alberta clay tills. The details of the site investigation and subsurface conditions are included. Pile installation and instrumentation details and procedures are discussed. Axial pile capacity prediction methods considered include four based on piezocone penetration test data (CPTu), one on flat plate dilatometer (DMT) data, and one using the conventional effective stress approach (Beta). Discussions and recommendations on the applicability and shortcomings of each method are included.

2 SITE INVESTIGATION PROGRAM
The site investigation program at the FMC site included a large number of in-situ tests, including CPTu, Seismic CPTu (SCPTu), DMT, and the Standard Penetration Test (SPT).

CPTu and SCPTu tests were completed using a high capacity 15cm² compression type seismic piezocene (Figure 1). CPTu and SCPTu tests provide near continuous measurements of tip stress, sleeve friction, and dynamic pore pressure response versus depth. Shear wave velocities were also measured for the soil strata at this site using the SCPTu method. However, the shear wave data is not considered for this paper. In addition, the CPTu measures axial inclination and probe temperature as it is advanced. CPTu data is recorded in
real time from an on board computer controlled data acquisition system. CPTu results are used directly in pile design, or used to reliably determine common geotechnical parameters for input into pile design methods. A 30 ton CPT Truck Rig operated by ConeTec Investigations Ltd. of Edmonton, Alberta was used for SCPTu and CPTu testing on this project. A more detailed description of cone penetration testing is presented by Robertson et al, 1986.

Flat plate dilatometer testing or DMT, is an in-situ test commonly used in Europe and the east coast of the United States. Recently, DMT is being incorporated into geotechnical practice in western Canada. The test is performed by hydraulically pushing an instrumented blade into the ground and incrementally recording data every 0.2m (ASTM D 6635-01). CPT equipment is often used to deploy the DMT. At each test depth, the stainless steel membrane is pressurized using an inert gas (Figure 2). The pressure required to counter the in-situ soil stresses and begin to move the membrane (lift-off pressure, \( P_0 \)) is recorded using a pressure gauge. The operator then records the pressure, \( P_1 \), required to inflate the membrane (i.e. displace the membrane into the soil) 1.1mm. By using \( P_0 \) and \( P_1 \) along with groundwater information many soil parameters can be calculated including undrained shear strength \( (s_u) \), over consolidation ratio (OCR) and material index, Id. DMT testing was performed by ConeTec Investigations Ltd of Edmonton, Alberta for this project. A more detailed description of the DMT is presented by Howie et al. (2007).

Auger Drilling with SPT Testing were completed on this project. SPT Energy calibrations were not performed to measure the actual hammer energy. Due to the poor repeatability of the SPT and the discontinuous nature of the results, the SPT data was not used in this paper. The auger drill hole logs and visual soil classifications were useful to verify the soil behaviour types indicated by the CPTu and DMT.

In addition, a test hole was drilled approximately 5m north of the test pile location using a 0.4m diameter pile auger to assess the need for casing during construction of the test pile. This hole was visually logged during drilling.

For simplicity, only results from one CPTu test and one DMT test are presented in this paper (Figure 3). Analyses using data from the other tests produced similar results.

3 SUBSURFACE CONDITIONS

The subsurface conditions at the test pile site are comprised of interlayered sand, silt and silty clay layers that are from lacustrine origin, overlying very stiff to hard silty clay till (Figure 3). The top of the very stiff to hard silty clay till is located approximately 7.5m below the existing ground surface in the general area of the test pile location. In this investigation, the top of the pile is at 8m below the ground surface as shown in Figure 4. Therefore, the entire test pile was located in the very stiff to hard silty clay till. Soil samples indicate the till to be low plastic silty clay containing trace amounts of sand, gravel and coal fragments as well as occasional cobbles and boulders. Layers of dense, saturated, silty sand and sandy silt were occasionally encountered within the silty clay till layer.

Figure 1. Schematic of Seismic Piezocone

Figure 2. Schematic of Flat Plate Dilatometer Blade (After Marchetti and Paloa Monaco 2001)
The test pile was a 1.4m diameter cast-in-place concrete pile. The pile was reinforced longitudinally with 12 No. 30M reinforcing bars, and transversely with No. 10M reinforcing bars at 0.3m vertical spacing. Longitudinal reinforcing bars were extended to the base of the pile.

4.1 Test Pile Installation

The base of the test pile was located 22.11m below the existing ground surface (Figure 4). It is anticipated that the new construction will incorporate a basement and crawl space that will extend about 8m below the existing ground surface. In order to limit the amount of concrete and steel required to construct the test pile, as well as ensure the pile would fail under the applied load, the total length of the test pile was limited to 14.11m with the upper 8m of the test hole remaining empty. The 14.11m long test pile therefore extended from a depth of 8m to 22.11m. A steel casing, extending from the ground surface to a depth of 8m, was left in the ground until completion of the test to ensure that the upper portion of the hole remained open. Loose soil was removed from the base of the pile excavation using the piling auger. The shaft wall and base condition were confirmed by downhole video prior to placing the steel and concrete. No seepage, sloughing or collapse was noted during the installation of the test pile.

Concrete slump values ranged from 0.20m to 0.23m, with an average of 0.22m. Four cylinders were tested in unconfined compression on the day of the pile load test (12 days after placement). Compressive strengths at the time of the pile load test ranged from 31.9MPa to 34.1MPa, with an average of 33.1MPa.

4.2 Test Pile Equipment and Instrumentation

The test pile was instrumented with an O-cell (Osterberg, 1998) installed in a pre-assembled rebar cage. The O-cell is a high capacity hydraulically activated jacking device that was installed in the rebar cage and concreted into the test shaft. The cell used for the FMC test was capable of applying a maximum load of 10MN in both directions. During the test, the O-Cell is pressurized to apply pressure upwards against the upper portion of the shaft, and downwards against the lower portion of the test shaft. Because the cell is installed within the test shaft, the reaction force required to carry out the test is derived from the pile/soil system. The upper portion of the pile provides the reaction force for the lower portion and vice-versa. This eliminates the need for reaction beams and tension piles typical of conventional pile load tests. An O-cell load test typically continues with increasing loads until one of the following occurs:

a) The ultimate shaft friction capacity of the upper portion of the test pile is reached;

b) The combined ultimate end bearing and shaft friction capacity of the lower portion of the test pile is reached, or;

c) The maximum O-cell capacity is reached.

The O-cell used for the FMC pile load test was instrumented with a pressure gauge and a vibrating wire pressure transducer to record both applied pressure and expansion. Three telltales consisting of 6.4mm diameter rods installed within 12.5mm diameter steel pipes were used to independently monitor the movement of the top and bottom of the O-cell. Two Linear Variable Displacement Transducers (LVDT) were used to monitor the movement of the top of the pile with respect to a fixed reference beam. Ten vibrating wire strain gauges were installed in pairs at five levels within the test pile to
monitor axial pile loads. The locations of the O-cell and the various instruments installed within the test pile are indicated on the schematic test pile section shown in Figure 4. A digital survey level was used to monitor movement of the reference beam during the test.

4.3 Test Pile Procedures

The pile load test was carried out in accordance with the Quick Load Test for Individual Piles (ASTM D1143 Standard Test Method for Piles under Static Axial Load). A total of 10 load increments were applied up to the maximum bi-directional load of 5.4MN. Loading was halted after the tenth increment as the shaft friction capacity of the upper portion of the pile was reached, and additional loading could not be maintained. Each load increment was maintained for a total of eight minutes. The test pile was unloaded in five decrements of load. A data logger was used to record instrument readings at 30 second intervals throughout the load test. The maximum movements of the upper and lower sections of the pile measured during the load test were 67.4mm and 14.8mm respectively.

4.4 Estimating the Ultimate End Bearing Capacity

The upper portion of the pile failed in shaft friction (i.e. no additional reaction capacity available to continue testing the lower portion of the pile) before the lower portion failed in combined shaft friction and end bearing. The ultimate end bearing capacity was therefore not reached during the test. However, enough data was collected to allow extrapolation of the test data to estimate the ultimate failure load of the section of the pile below the O-cell using both the Brinch-Hansen and Chin Failure Criteria, as outlined in the Canadian Foundation Engineering Manual (CFEM). These two methods were used for the analyses, as independent estimation methods, to compare the ultimate end bearing capacity against values predicted using the six method discussed in this paper.

5 AXIAL PILE CAPACITY

The ultimate axial pile load capacity, \( Q_u \), is calculated as the summation of two components: end bearing resistance, \( Q_p \), and friction resistance, \( Q_b \). The end bearing resistance is calculated as the product of the unit end bearing stress, \( q_p \), and the pile end area, \( A_p \), in the case of closed-end piles. On the other hand, friction resistance is calculated as the summation of the unit skin friction, \( q_b \), multiplied by the outer area of the pile shaft, \( A_{si} \) at every layer, \( i \). Therefore, the ultimate pile capacity can be expressed as:

\[
Q_u = Q_p + Q_b = q_p A_p + \sum q_b i A_{si}
\]

5.1 Prediction of Axial Pile Capacity in Clay

There are many different established methods in the literature for predicting axial pile capacity. In this paper, six predictive methods using conventional approach (Beta method) and relatively modern approaches (CPTu and DMT methods) are evaluated. A brief description of each method used in the current investigation is described in the following section.

5.2 Conventional Methods for Pile Design

The total stress approach, commonly termed “Alpha method”, is used to evaluate undrained pile capacity in saturated clays. In this approach, the unit skin friction is calculated using factored undrained shear strength which can be expressed as:

\[
q_b = \alpha s_u F1
\]

Where \( \alpha \) is the adhesion factor, \( s_u \) is the undrained shear strength and \( F1 \) is a reduction factor for pile slenderness. The unit base resistance in this approach is obtained from undrained bearing theory as:

\[
q_p = N_c s_{ub} + \sigma_{ob}
\]

Where \( N_c \) is the bearing capacity factor, \( s_{ub} \) is the average shear strength over the depth of 2D below the pile base and \( \sigma_{ob} \) is the total overburden stress at the pile base level. The factor \( N_c \) is calculated as:
\[ N_c = 6 + L/D \leq 9 \]

Where \( L \) is the pile length and \( D \) is the pile diameter.

The effective stress approach “Beta method” is also used for the assessment of axial pile capacity. It is argued that the excess pore pressure developed in the vicinity of the pile shaft due to pile loading dissipates rapidly. Thus, it is more appropriate to consider drained conditions at soil-shaft interface. Hence, the unit skin friction is related to the effective vertical stress, \( \sigma_v' \), using the expression:

\[ q_b = \beta \sigma_v' \]

Where \( \beta \) is the shaft friction coefficient.

For normally consolidated clay, \( \beta_{nc} \) varies between 0.24 and 0.29 when the effective angle of friction of the clay falls within the range of 20 to 30 degrees. For over consolidated clay, \( \beta \) is greater than \( \beta_{nc} \). Meyerhof (1976) recommended that \( \beta \) can be related to the over consolidation ratio (OCR) as:

\[ \beta = \beta_{nc} \text{ (OCR)}^{0.5} \]

In this paper, \( \beta \) was assumed to be 0.5 (based on OCR = 4 from DMT and \( \beta_{nc} \) assumed to be 0.25). It should be noted that the effective stress approach is only used to evaluate the unit skin friction. The unit end bearing resistance is still obtained using the total stress approach.

### 5.3 CPT Methods for Pile Design

CPTu test results are often utilized to predict the axial pile capacity for both bored and driven piles using different methods (e.g. De Ruiter and Beringen, 1979; Bustamante and Gianeselli, 1982; Eslami and Fellenius, 1997). In these methods, both unit skin friction resistance and unit end bearing resistance are evaluated using a factored tip resistance.

\[ q_b = \sum c_{ai} q_i \]

\[ q_p = c_b q \]

where \( q_i \) is the representative cone tip resistance for layer \( i \), \( c_{ai} \) is the empirical factor to the convert \( q_i \) to unit skin friction of the pile, \( q \) is the representative cone tip resistance at the pile base and \( c_b \) is an empirical factor to convert \( q \) to the unit end bearing of the pile. The representative cone tip resistance is identified differently in each method. This is usually done by averaging the data over a depth interval that is related to the relative depth \( L/D \).

More recently, CPTu sleeve friction, \( f_s \), and excess pore pressures (\( \Delta u \)) during penetration are utilized to evaluate the side friction of bored and driven piles, as opposed to using a factored tip resistance (Takesue et al. 1998).

De Ruiter and Beringen (1979) proposed procedures to evaluate both unit skin friction and unit end bearing capacities in clay using factored undrained shear strength, \( s_u \), where \( s_u \) is computed from the uncorrected cone tip resistance, \( q_c \).

\[ s_u = q_c / N_k \]

\[ N_k = 15 \] to 20

De Ruiter and Beringen then evaluated the pile capacity in clay from the expressions:

\[ q_p = \alpha s_u \]

Where \( \alpha \) is 1 for normally consolidated and 0.5 for over consolidated clay.

\[ q_p = N_c s_u \]

\[ N_c = 9 \]

Bustamante and Gianeselli (1982) method, commonly called the LCPC method, is based on data from 197 pile load tests carried out on a wide range of soil types with different pile types and construction procedures. In this method the unit end bearing capacity is calculated as the product of equivalent average CPT tip resistance, \( q_{ca} \), and the end bearing coefficient, \( k_p \). The unit skin friction is calculated by dividing the uncorrected cone tip resistance, \( q_c \), by a friction coefficient, \( \alpha_{LCPC} \).

\[ q_b = q_c / \alpha_{LCPC} \]

\[ q_p = k_p q_{ca} \]

More details about the calculation of \( q_{ca} \) and the selection of \( \alpha_{LCPC} \) can be found in the original paper by Bustamante and Gianeselli (1982). It should be noted that Bustamante and Gianeselli recommended maximum values for \( q_p \) to avoid overestimating the pile capacity due to the use of high \( q_{ca} \). This could be a result of, for example, thin hard-soil layers or construction procedures.

Eslami and Fellenius (1997) described a method to evaluate pile capacity using corrected cone tip resistance, \( q_c \), as well as the dynamic pore pressure (\( u \)) measured at the pile shoulder. Eslami and Fellenius then evaluated the pile unit shaft friction using the expression:

\[ q_b = C_a q_E \]

where \( C_a \) is the shaft friction coefficient and \( q_E \) is the effective cone resistance. \( q_E \) is obtained by subtracting pore pressure, \( u \), from \( q_p \). In this method, end bearing resistance of the pile can be evaluated as:

\[ q_p = C_t q_E \]

where \( C_t \) is the end bearing coefficient and \( q_E \) is the is the geometric average of \( q_c \) over the depth of influence above and below the pile base. The reader is advised to refer to the original paper (Eslami and Fellenius 1997) for more details about obtaining \( C_a \), \( C_t \), and \( q_E \).

Takesue, et al (1998) proposed a method to evaluate the unit side friction of the pile directly from the CPTu sleeve friction measurement (\( f_s \)) and the excess porewater pressure, \( \Delta u \):

For \( \Delta u < 300\) kPa:

\[ q_b = f_s \left( \Delta u / 1250 \right) + 0.76 \]
For $\Delta u > 300$ kPa:  
$q_b = f_s ([\Delta u / 200] - 0.50)$

In clays, the end bearing resistance is the effective cone resistance, $q_{ce}$ (Eslami and Fellenius 1997).

5.4 DMT Method for Pile Design

For compression piles in clays Powell et al. (2001) proposed the evaluation of unit side friction using the DMT material index:

For $Id < 0.6$:  
$q_b / (P_1 - P_0) = -1.1111 Id + 0.775$

For $Id > 0.6$:  
$q_b / (P_1 - P_0) = 1.11$

Powell et al. evaluated the end bearing as the product of the DMT bearing capacity factor, $K_{di}$, and $P_1$ at the pile base.

$q_p = P_1 K_{di}$

6 RESULTS AND DATA PRESENTATION

Each axial pile capacity predictive method presented above is compared to the measured pile response during the full scale load test. Results from each of the methods are presented in Figures 5 and 6. The predicted and measured ultimate side friction along the pile shaft, $q_{fp}$ and $q_{fm}$ respectively, are presented in Figure 5. These values are calculated as the product of the shaft friction stress, $q_b$, and the outside area of the pile shaft, $A_s$.

Figure 5b shows the ratio of predicted versus measured side friction resistance along the pile shaft. It should be noted that the LCPC method is used with and without limiting values for discussion purposes. Figure 6 evaluates the end bearing resistance predicted using the predictive methods, $q_p$, described in this paper against the maximum end bearing measured at the end of the test, $q_{p_m}$. Figure 7 presents the ratio of the ultimate end bearing resistance, $q_{pe}$, that were estimated using extrapolation approaches (Section 4) and $q_{p_m}$.

7 DISCUSSION

The effective stress approach (Beta method) is shown to give a reasonable prediction of side friction values (Figure 5). However, this method incorporates a wide range of Beta factors. For the data presented in this paper, a back calculated Beta factor of 0.5 is shown to give the best prediction of overall side friction capacity. Therefore, the Beta method should not be used without local knowledge and/or pile load tests for confirmation.
Similarly, De Ruiter and Beringen method uses a wide range of $N_k$ (15-20). Furthermore, De Ruiter and Beringen method incorporates the shear strength, $s_u$, in the shaft friction prediction which is calculated as $(q_c / N_k)$. A more recent approach calculates $s_u$ as $[(q_t - \sigma_v)/N_k]$ (Lunne et al 1997). Use of the more recent approach with De Ruiter and Beringen method would result in a reduced shaft friction predicted value in this particular case studied in this paper (i.e. closer to the measured value).

For the clay till soil found at this site the LCPC method without limits produced improved predictive values for shaft friction capacity (Figure 5). A similar observation was reported by Almeida et al. (1996).

![Figure 6. Predicted/Recorded End Bearing Ratio (q$_{pm}$ recorded at the end of the pile test)](image)

Figures 5 and 6 show that Takesue method which incorporates all measured CPTu parameters (i.e $q_t$, $f_s$ and $u$) excessively overestimated both side friction and end bearing capacities. Similar overestimation is encountered using Eslami and Fellenius Method for end bearing, compared to the general trend of the other predictive methods (Figure 6) as well as the methods used to estimate the ultimate end bearing capacity using extrapolation approaches described in Section 4.4 and presented in Figure 7. Both methods incorporate the CPTu dynamic pore pressure measurement ($u$). Under scrutiny, it appears that the pore pressure measurements below the stiff fine grained layer at 17m is subdued (Figure 3). This could be due to the dilative nature of the stiff fine grained layer desaturating the CPTu pore pressure element. The pore pressure measurement does not appear to fully regain saturation until a depth below the pile base. Therefore, portion the overpredictions of the pile end bearing resistance might be due to the limitations of dynamic pore pressure measurements in stiff fine grained soils. The predicted end bearing capacity using both methods would be reduced by about 10% if the dynamic pore pressure was maintained at about 100m of water pressure. The Eslami and Fellenius method has also shown to provide slightly an overestimated value for shaft friction capacity for the bottom portion of the pile shaft (Figure 5). Again, the dilative behaviour and the desaturation of the pore pressure element observation contribute to this overestimation. The Same reduction factor of 10% in the predicted value is achieved if the dynamic pore pressure is maintained at 100m of water pressure. On the other hand, the Takesue method would further overestimate the shaft friction capacity if the pore pressure was higher.

Despite limited experience using DMT data in overconsolidated clay till, the Powell et al. (2001) method, for compression piles, provided reasonable predicted values for the pile shaft friction (Figure 5).

![Figure 7. Estimated/Recorded End Bearing Ratio (q$_{pm}$ recorded at the end of the pile test)](image)

Unfortunately, the DMT test was not conducted to the depth of the pile base and so the DMT method could not be used to make a prediction of the end bearing resistance.

By excluding the Takesue method and averaging the predicted side friction values from each of the other methods, the averaged prediction arrives at a very close prediction of the overall frictional pile response (Figure 5c).

8 CONCLUSIONS

In situ test data, and specifically piezocone penetration testing and flat plate dilatometer testing adequately estimated pile capacity for the clay till site investigated in this paper with the exception of Takesue method. However, in the absence of local experience with the applicability of the predictive methods, these methods must be evaluated prior to applying them for design.

For the case of fine grained till, care must be taken when using predictive methods that incorporate piezocone dynamic pore pressure. The dynamic pore pressure measurements are sensitive to highly dilative layers that can subsequently subdue the pore pressure response.
This can result in misleading prediction for the pile capacity. Based on this case study alone it is recommended that predictive methods incorporating piezocone dynamic pore pressure, strictly, Eslami and Fellenius 1997, can be used in clay till with caution when dilative behaviour is encountered.

Finally, a single predictive method should not be used alone but rather several methods should be considered. In this case study, a simple average of the methods (after excluding one apparent outlier) appears to work well (Figure 5c).

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