Predicted foundation settlements using cptu versus measured settlements in glacial till deposits in Oshawa

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ABSTRACT
This paper presents the results of settlement prediction based on piezocone penetration testing (CPTu) for a site in the City of Oshawa, Ontario, which is underlain by extensive lightly overconsolidated glacial deposits. The settlement prediction is compared to the data taken from settlement monitoring of a two-storey building and calibration of the settlement prediction approach is discussed. Determination of the in-situ engineering properties of foundation materials has always been a geotechnical engineering challenge, particularly for the normally consolidated or lightly over-consolidated glacial deposits with mean grain size close to 0.075 mm criterion. Enhanced in-situ testing such as CPTu when combined with selected traditional soil sampling and testing provides a continuous profile of subsurface conditions which is a strong theoretical basis for interpretation and provides confidence in the geotechnical site characterization. This presents an advantage for assessment of bearing capacity and settlement of the foundations in difficult soil conditions where routine soil borings would have resulted in recommending pile foundations for the planned two-story building. Conversely, after using enhanced in-situ testing (CPTu), the building was designed with conventional footings. Based on the results of the settlement monitoring of the subject building during and post construction, the effectiveness of settlement prediction by CPTu is evaluated. The constrained modulus of the subsurface soil is back calculated based on the actual settlement and suitable calibration of the CPTu data for the specific soil type within the subject site is discussed. These calibration methods can be used as a valuable tool for geotechnical engineers to predict settlement in similar subsurface conditions.

RESUMEN
Este artículo presenta los resultados de la predicción de asentamientos con base en el ensayo de penetración con piezocono (CPT$_U$) llevado a cabo en un terreno en la ciudad de Oshawa, Ontario, el cual está compuesto por extensos depósitos glaciales moderadamente sobreconsolidados. La predicción de asentamientos es comparada con los datos obtenidos del monitoreo de los asentamiento de un edificio de dos pisos y se discute la calibración del método de predicción de asentamientos. La determinación in-situ de las propiedades ingenieriles de los materiales de fundación ha sido siempre un reto para la Ingeniería Geotécnica, en particular cuando se trata de depósitos glaciales normalmente consolidados o moderadamente sobre consolidados con tamaños de partículas cercanas a 0.075 mm. Ensayos mejorados in-situ, como es el caso del CPT$_U$ cuando es combinado con ensayos y muestreos tradicionales de suelos, suministran un perfil continuo de las condiciones del subsuelo y proporcionan una buena base teórica para su interpretación, al igual que mayor seguridad en la caracterización geotécnica del suelo. Esto representa una ventaja en la evaluación de la capacidad portante y asentamientos de fundaciones en condiciones difíciles de suelos en las que normalmente, con base en perforaciones de suelos de rutina, se habría recomendado un sistema de fundación con pilotes para la edificación de dos pisos. Sin embargo, después de usar el ensayo mejorado in-situ (CPT$_U$), el edificio fue diseñado con zapatas convencionales. Con base en los resultados del monitoreo de asentamientos antes y después de la construcción del edificio previamente mencionado, la efectividad de la predicción de asentamientos con el ensayo CPT$_U$ es evaluada. El Modulo Confinado del subsuelo es calculado con base en el asentamiento real y se discute la calibración adecuada de los datos del CPT$_U$ para el tipo especifico de suelo encontrado en este terreno. Estos métodos de calibración pueden ser usados como una herramienta valiosa para los ingenieros de Geotecnia en la predicción de asentamientos de suelos en condiciones similares.

1 INTRODUCTION

Settlement calculation is usually the key factor in designing foundations and can considerably affect a project cost. Therefore, it is essential to assess the subsurface profile and related soil mechanical properties in order to conduct adequate settlement prediction. Standard penetration test (SPT) is one of the most common methods in Ontario for a site investigation which is simple and inexpensive. However, the soil samples are disturbed and available correlations generally provide a wide range for soil geotechnical properties. The cone penetration test with pore pressure measurement (CPTu) or piezocone has several advantages relative to SPT. CPTu is faster, repeatable, can provide continuous data, and is highly applicable in indentifying soil geotechnical properties and subsurface strata type. Piezocone has a strong theoretical background and there are several semi-empirical correlations to estimate soil geotechnical parameters. In early 2010, Coffey Geotechnics Ltd. undertook a geotechnical investigation for proposed commercial buildings located in the west portion of Oshawa. Both SPT and CPTu were conducted at the site location. In addition to conventional split-spoon samples, several undisturbed samples were obtained for laboratory testing. Due to the past prediction of large settlement for the proposed development before our involvement, it was essential to properly predict the footing settlement. In this paper, the results of the CPTu tests, settlement back calculations, and finally a comparison between various settlement prediction approaches are presented. In order to estimate the constrained modulus (M) from the CPTu data, three methods were applied including, Sanglerat (1972), Kulhawy and Mayne (1990) and Robertson (2009).

2 PROJECT BACKGROUND

Figure-1 illustrates the approximate footprint area of two buildings named E1 and E2. A previous geotechnical investigation was completed at the site by others in 2006. Three boreholes were advanced in 2006 within the area of the proposed buildings (not shown), using a track-mounted drill-rig equipped with hollow-stem and solid-stem augers. Soil samples were obtained while conducting SPT. Based on the borehole logs prepared by others, the subsurface conditions under the proposed buildings in 2006 included a thin layer of topsoil over 1.4 m of fill soils, underlain by native soils to the termination depth of the boreholes (up to 19 m below ground level). The predominant native soil reported was a glacial till deposit of sandy silt with some clay. Furthermore, the main native soil was reported to be loose to very loose, based on the low SPT values. The existing topsoil and uncompacted fill was removed within the proposed building footprint in 2007 and replaced with compacted engineered fill material to develop the required building pads. During this earthwork operation, additional engineered fill was also placed and the original site grades were raised by 1 m to 2.5 m to the design final site grade. Prior to placement of the engineered fill, three settlement plates (SP401, SP402, and SP403) were installed over native soil at the locations shown on Figure-1 and settlement monitoring was conducted for the prepared building pads between March to July, 2007.

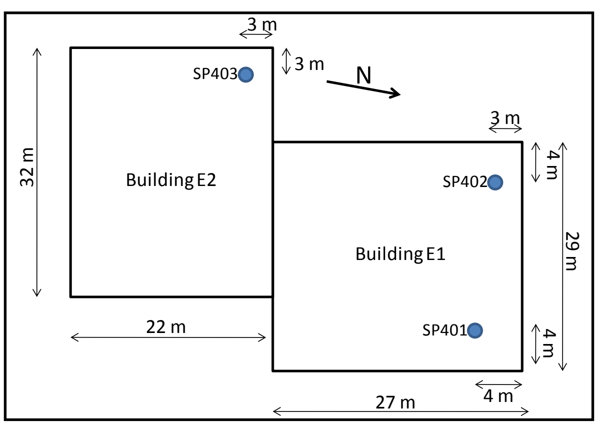

Figure-1: The plan view of the constructed building pads in 2007 and the location of the settlement plates

The designed location and footprint of the proposed buildings changed after placement of the engineered fill in 2007. The former and new building areas are shown in Figure-2. Furthermore, based on the findings of the previous geotechnical investigation, the project designer was considering the use of pile and structural slab for one of the proposed buildings (two-storey). Subsequently, Coffey was retained by the owner to carry out a geotechnical review in 2010.

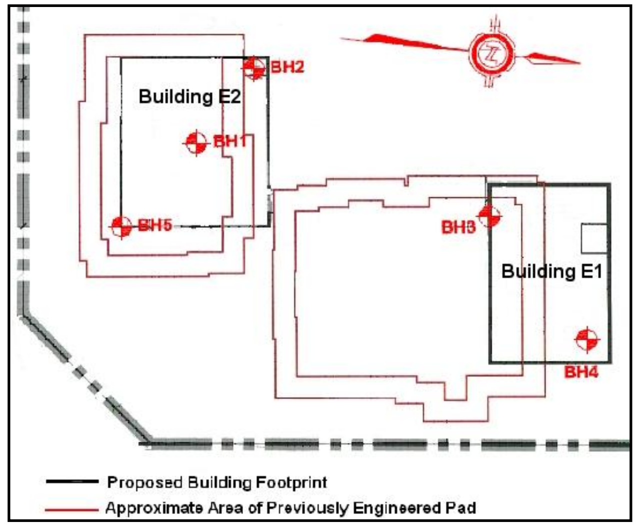

Figure-2: Previous and new building location plan

3 SITE GEOTECHNICAL INVESTIGATION

Following the geotechnical review, it was decided to conduct a supplementary geotechnical investigation. Five boreholes were advanced within the new building footprints (BH1 to BH5) as shown on Figure-2. In addition, enhanced in-situ geotechnical testing was conducted by advancing CPTu at the location of each borehole.

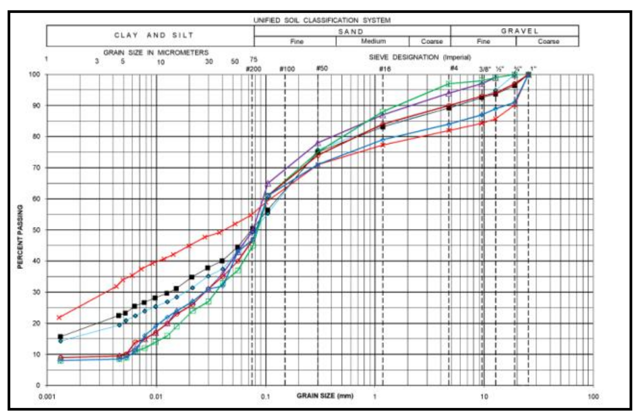

Figure-3: Grain size analyses of the main glacial till deposit

Based on the conditions encountered in the advanced boreholes and results of the laboratory tests on collected samples, the soil profile consisted of 2.1 m to 3.3 m thick fill soils underlain by native deposits. The predominant unit of the native soils was a stiff to soft glacial till extending to depth of 16.5 m. The main native soil unit was underlain by a 1.5 m thick layer of silty clay over compact to very dense silty sand. The results of the grain size analyses conducted on the samples of the main native soil unit (glacial till) collected during the current geotechnical investigation and in 2006 by others are shown in Figure-3. The Atterberg Limits tests determined liquid limit ranging from 11 to 17% and plasticity index ranging from 2 to 7%. Based on the laboratory test results and in accordance with the Unified Soil Classification System (USCS), the main native soil unit is classified as Silt (some clay, some sand to sandy) to Silty Clay (some sand to sandy). The groundwater level was recorded in the piezometer installed in BH1 at a depth of 1.6 m.Prediction of soil type based on CPT is referred to as Soil Behaviour Type (SBT). Robertson (1990) proposed a CPT soil behaviour chart based on normalized CPT data. Figure-4 illustrates the variation of SBT Index below ground level and associated soil layers type at the location of CPTu 1 (adjacent to BH1, Figure-2).

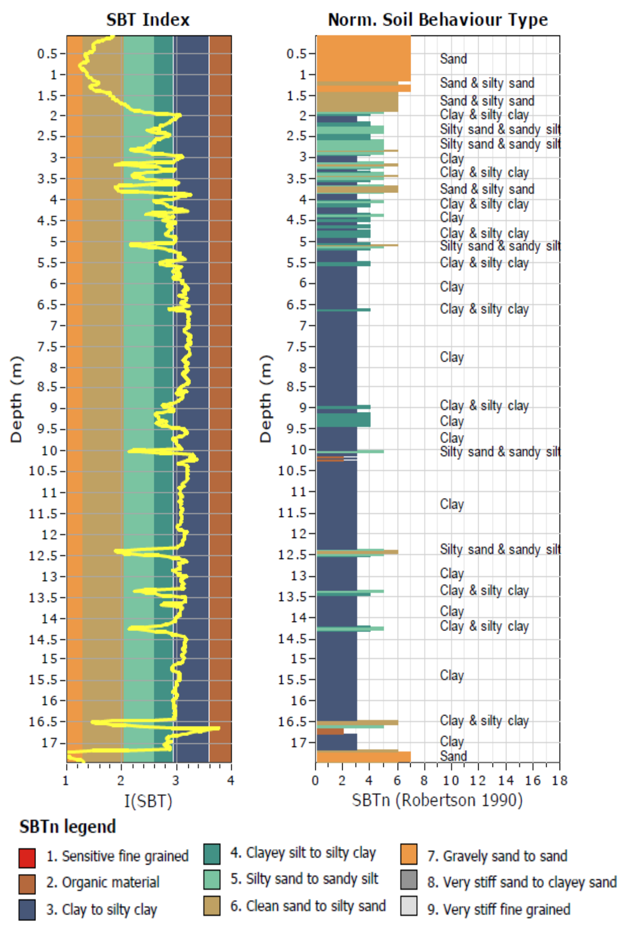

Figure-4: Variation of SBT Index below ground level at the location of CPTu 1

It should be noted that the results of the soil classification for main native soil unit based on the USCS (Silt to Silty Clay) is somehow different than the predicted soil type based on CPT (Clay and Silty Clay). According to Robertson (1990), "*the CPT cannot be expected to provide accurate predictions of soil type based on physical characteristics, such as, grain size distribution but provide a guide to the mechanical characteristics of the soil, or the soil behavior type (SBT)*". In other words, soil classification based on CPT (SBT) is more indicative of the soil behaviour during penetration process and is not necessarily identical to the soil classification based on the index testing (e.g., grain size, Atterberg). This difference is particularly more significant for soils with mean grain size close to 0.075 mm criterion. In summary, the soil classification based on CPT interpretation is a Soil Behavioural Type and does not necessarily replace the index testing description.

Subsequent to completion of the supplementary geotechnical investigation, it was ultimately recommended to support the proposed buildings over conventional footings. Building E2 is planned to be a two story slab-on-grade structure with no basement.

4 SETTLEMENT MONITORING PROGRAM

Prior to construction of the building in 2010, the previously constructed buildings pads (in 2007) were amended based on the new footprint and location of the building. The settlement of the Building E2, during and after completion of the construction was monitored. The column loads for this building is reported to be as high as 650 kN (at SLS) with maximum dead load of 360 kN. The location of nine settlement points (SSP) marked on foundation of Building E2 is shown on Figure-5.

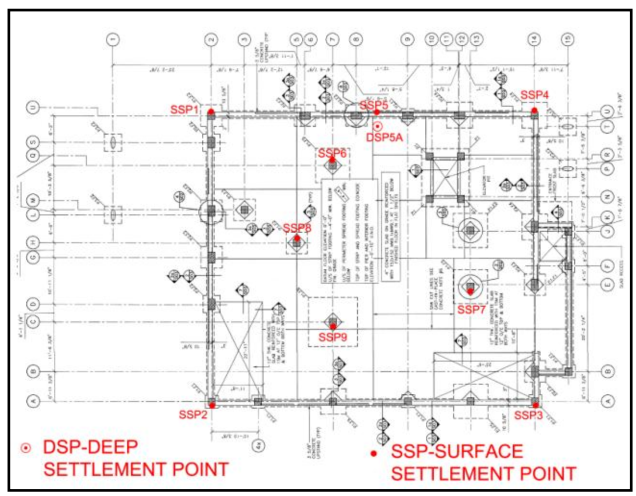

Figure-5: The plan view of the foundations for Building E2 and the location of the settlement markers

These locations included four footings at each corner of the building (SSP1 to SSP4) and four interior footings (SSP6 to SSP9). The accuracy of the settlement reading is about +/-2 mm. Construction of the building was completed around late February 2011 and the settlement monitoring program is still ongoing at the time of preparation of this text. Estimating the constrained modulus (*M*) of the subsurface soils is vital in assessment of settlement. Three methods, Sanglerat (1972), Kulhawy and Mayne (1990), and Robertson (2009) were used to estimate the constrained modulus. Table 1 presents each of these equations and their conditions.

Table-1 Constrained modulus correlation from CPTu test

Sanglerat (1972)	$M = \alpha_m q_c$ q_c =measured cone resistance α_m =constant and estimated as below: Silt(ML): 3 to 6, where $q_c < 2$ MPa 1 to 3, where $q_c > 2$ MPa Clays (CL): 3 to 8, where $q_c < 0.7$ MPa 2 to 5, where $0.7 < q_c < 2$ MPa 1 to 2.5, where $q_c > 2$ MPa
Kulhawy and Mayne (1990)	$M = a(q_t - \sigma_{v0})$ q_t =total cone resistance σ_{v0} =total vertical stress, *a*=constant, typically equal 8

Robertson (2009)	$M = \alpha_m (q_t - \sigma_{v0})$
	If $I_c > 2.2$ then
	$\alpha_M = Q_t$ when $Q_t \le 14$,
	$\alpha_M = 14$ when $Q_t > 14$
	If $I_c < 2.2$ then
	$\alpha_M = 0.0188[10^{(0.55 I_c + 1.68)}]$
	I_c =soil behaviour type index
	$Q_t = \frac{(q_t - \sigma_{v0})}{\sigma'_{v0}}$
	σ'_{v0} =effective vertical stress

The above mentioned methods are evaluated through comparison between calculated settlements and recorded settlements for the foundation of Building E2 (SSP1 to SSP9) in 2010/2011 and the settlements recorded during the original grade raise (SP401 to SP403) in 2007. The results of the closest piezocone tests to the location of each settlement marker are used in calculation of settlements. Figure-6 compares the estimated constrained modulus of the subsurface soil based on the three different methods for CPTu1. The upper end of α_m constants (e.g., 6, 8) were used in estimating the constrained modulus for Sanglerat method (Table-1). The difference between the three methods are larger in the upper portion of the strata (corresponding to sandy/silty soils) and as the soil becomes more clayey, the three methods provide closer results (Figure-4 and Figure 6). Similar trend was noted for the other piezocone tests.

From elasticity theory the relationship between constrained modulus and elastic Young's modulus is:

$$E' = \frac{(1+\upsilon')(1-2\upsilon')}{(1-\upsilon')} M' \qquad [1]$$

in which E' is the soil Young's modulus, M' is the soil's constrained modulus (M) and υ' is the soil's Poisson's ratio. Therefore, it is possible to calculate the elastic Young's modulus of the subsurface below the plate level. An effective Poisson's ratio of 0.2 to 0.25 is assumed.

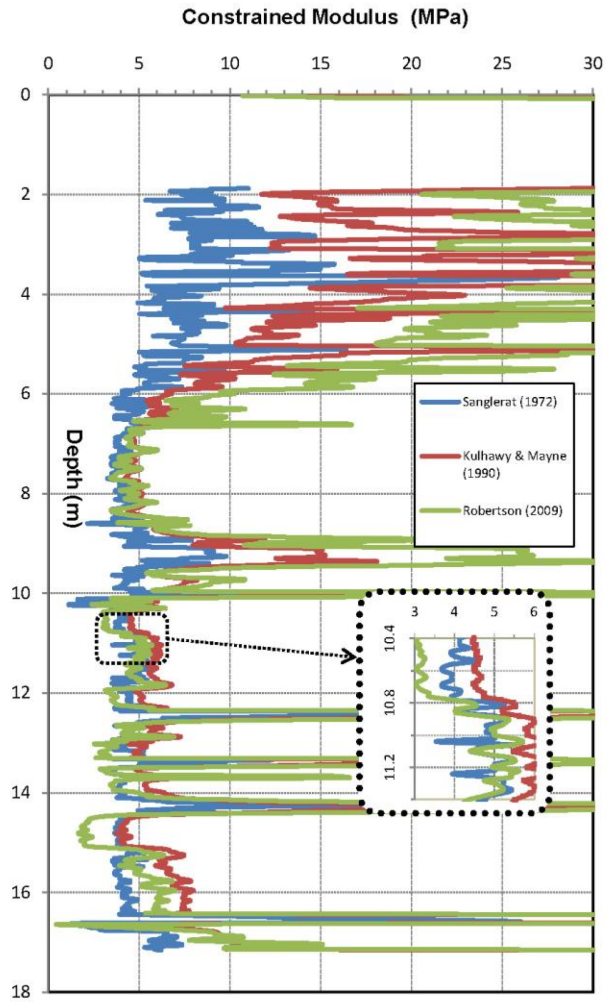

Figure-6. Comparison between the Constrained Modulus calculated based on three different methods at the location of CPTu 1

The ground settlement under the settlement plates (SP401 to SP403) was then calculated due to the weight of engineered fill material used to develop the required grade raise within the constructed building pads E1 and E2. Figure-1 illustrates the plan view of the constructed building pads and the location of the settlement plates SP401 to SP403. Similarly, the ground settlement under the footings (SSP1 to SSP9) was calculated based on the dimension of the footings and dead load of the associated column. The change in stress is calculated at the same intervals as the CPTu test readings to develop continuous stress distribution below the settlement plate/footing to the bottom of the borehole.

The settlement of each layer is calculated from simple elasticity theory as follows:

$$S_d = \frac{H\left(\Delta\sigma_z - \upsilon'\left(\Delta\sigma_x + \Delta\sigma_y\right)\right)}{E'} \quad [2]$$

in which H is the thickness of soil layer for which the settlement is calculated. Also $\Delta\sigma_z$, $\Delta\sigma_x$, and $\Delta\sigma_y$ are the change in stress in vertical (z) and horizontal (x and y) directions. The ultimate settlement (S_d) is calculated for each of the three different methods of constrained modulus (Table-1). The calculated and measured settlement under SP401 to SP403 (original grade raise) and SSP1 to SSP9 (footings of Building E2) are compared for each method of estimating the constrained modulus. The maximum measured settlements until May 2011 are used for the foundations. The settlement monitoring of plates SP401 to SP403 during original grade raise (March to July, 2007) showed that the time dependent settlements were completed within 70 to 90 days of the applied surcharge. Therefore, it is anticipated the consolidation settlement of the foundations is predominantly completed by May 2011. For the purpose of comparison, the normalized settlement ratio (predicted settlement over the maximum recorded settlement, S_d / S) are illustrated in Figure-7. In general, the settlements associated with the constrained modulus calculated based on the third method (Robertson 2009) is the best fit with the measured results and return the closest results, particularly in comparison with Sanglerat (1972) method.

5 CONCLUSION

Geotechnical investigation was conducted for a proposed commercial buildings located in Oshawa, Ontario. Both SPT and CPTu at the site location were carried out. The results of the CPTu tests, settlement back calculations, and finally a comparison between various settlement prediction approaches are presented and discussed in this paper. Three methods, Sanglerat (1972), Kulhawy & Mayne (1990) and Robertson (2009) were used to estimate the constrained modulus from the CPTu test results. These methods were evaluated through comparison between calculated settlements and measured settlements. In general, the Robertson method returns the closest results to measured settlement for the subject site, in comparison with the two other methods. However, Kulhawy & Mayne (1990) method is practically very close to Robertson.

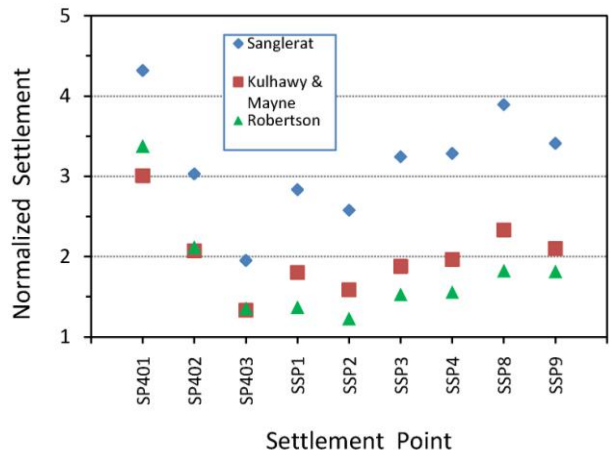

Figure-7 Normalized settlement ratio, predicted settlement over the maximum recorded settlement, S_d / S

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