Settlement prediction using CPT data in the Central Okanagan Valley, Kelowna, BC

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ABSTRACT
This paper investigates two CPT-based settlement prediction approaches and the calibration of these approaches for the City of Kelowna, BC, which is underlain by glacio-lacustrine deposits of loose sand, soft silt, and clay. Constrained modulus and Janbu approach were analyzed using a database of 9 sites. For the constrained modulus approach, the average α coefficient for the 9 sites was 3.13 and the predicted settlement, based on this average, was within ±25% of the estimated ultimate settlement. The Janbu approach was also calibrated using this database and the results were within ±15% of the estimated ultimate settlement. The accuracy of settlement prediction was considerably improved for both methods, when the calibration results of this study were applied. As opposed to previously published values, these calibrated methods can be used as a valuable tool for geotechnical engineers to predict settlement in Kelowna, BC.

RÉSUMÉ
Ce document porte sur deux CPT règlement de prédiction fondées sur des approches et l'étalonnage de ces approches de la ville de Kelowna, BC, qui est sous-tend par glacio-lacustres, les dépôts de sable, de vase molle, et de l'argile. Constrained module et Janbu approche, ont été analysés en utilisant une base de données de 9 sites. Pour le module de contrainte approche, le coefficient moyen pour les 9 sites 3.13 et les prévisions de règlement, sur la base de cette moyenne, est de ± 25% de l'estimation finale de règlement. Le Janbu approche a également été calibré à l'aide de cette base de données et les résultats étaient de ± 15% de l'estimation finale de règlement. La précision de la prédiction de règlement a été considérablement amélioré pour les deux méthodes, lors de l'étalonnage des résultats de cette étude ont été appliquées. Par opposition aux valeurs publiées précédemment, ces méthodes d'étalonnage peuvent être utilisées comme un outil précieux pour les ingénieurs géotechniques de prévoir le règlement dans Kelowna, BC.

1 INTRODUCTION

Over the past few decades urban development has shifted dramatically from sprawling to towering. This can be attributed to several reasons such as increased land prices and the move towards sustainable land development practices. This transition has brought complex geotechnical design challenges, typically encountered in larger cities, to smaller communities.

Kelowna, BC is a good example of this trend. The first high rise structure in Kelowna was constructed in the early 1990's. However in recent years, most development proposals for central Kelowna call for buildings that are over 20 stories, some up to 30 stories. The soil profile in central Kelowna generally consists of normally consolidated glacio-lacustrine silts with interbedded silty sands and clay sediments over 30 m thick. (Nasmith 1962). Therefore, the majority of high rise structures being built or proposed to be built, in central Kelowna typically require soil improvement. Given extensive thickness of these deposits, soil improvement methods such as piles and vibro-replacement have proven to be costly. Consequently, preloading is being used extensively as a method of soil improvement. In general practice, settlement estimations and preload design are based on geotechnical investigation results, which typically include boreholes and laboratory tests.

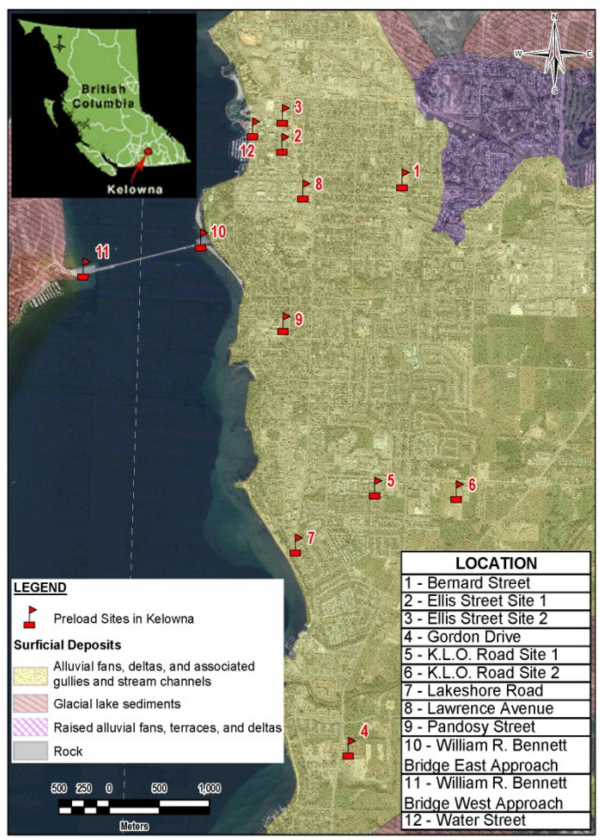

Figure 1. The sites included in the database.

In layered deposits, predicting settlement using borehole logs and discrete sampling may not provide the required accuracy. In these types of deposits, in addition to boreholes, cone penetration testing (CPT) can provide a continuous sounding of the soil profile and help better predict settlement.

Nevertheless, settlement of glacio-lacustrine silt deposits is highly influenced by several factors including soil structure, stress history, plasticity, and the applied load. As a result, there have been considerable variations between measured settlement and predicted settlement using standard correlation methods.

In this research, CPT data, preload geometry, and measured settlement information from 12 sites were compiled into a database (Table 1). This database was then used to calibrate two CPT based methods used for predicting setlement.

Table 1. The preload data for the sites in the database

Site	Maximum Pre-Load Height (m)	Maximum Surface Settlement (mm)
Bernard Street	11.0	840
Ellis Street Site 1	9.5	1192
Ellis Street Site 2	3.6	174
Gordon Drive	4.5	450
K.L.O. Road Site 1	3.3	77
K.L.O. Road Site 2	3.6	75
Lakeshore Road	4.5	507
Lawrence Ave	4.0	312
Pandosy Street	5.8	500
William R. Bennett Bridge East Approach	2.2	204
William R. Bennett Bridge West Approach	3.9	1110
Water Street	10.0	790

2 SETTLEMENT ESTIMATION

Due to its simplicity and reliability, the CPT is frequently used to determine the cone tip and shaft resistance from which soil stiffness and strength can be estimated.

Over the last few decades, researchers have presented various empirical relationships to correlate cone tip and shaft resistance to measured foundation performance, which has shown its value and reliability in settlement predictions (Schmertmann 1986, Mayne and Frost 1988; Marchetti et al. 2001). These computations include converting the cone tip and shaft resistance to constrained modulus; and calculating the uniaxial strain, ε, and the resulting settlement in each sublayer, ρ. For each sublayer, the change in stress is calculated from classical elastic theory solutions (Poulos and Davis 1970; Mayne and Poulos 1999). The total settlement can be obtained summing up the settlement from all sublayers:

$$\rho_{total} = \sum \frac{\Delta\sigma_v}{M} \Delta z \qquad [1]$$

where:

$\Delta\sigma_v$ = stress increase

M = the constrained modulus

Δz = sublayer height

Several researchers published empirical or semi-empirical relationships to estimate the constrained modulus (Schmertmann 1970; Massarch 1994). In this study, two different approaches namely; constrained modulus approach, and Janbu approach were used to test the database, which consists of CPTs and measured settlement data.

2.1 Constrained Modulus Approach

Schmertmann (1970) was the first researcher who related the settlement modulus directly to cone tip resistance particularly for fine sandy soils (i.e., $E = 2q_c$).

Typically, for a cohesionless soil, an average applied stress limited to a value of 25% of the estimated ultimate bearing resistance is used for settlement calculations (Fellenius, 2009):

$$E_{25} = \alpha q_t \qquad [2]$$

E_{25} = secant modulus for a stress equal to about 25% of the ultimate stress

α = an empirical coefficent

q_t = the cone penetration resistance corrected for pore water pressure

The Canadian Foundation Engineering Manual (2006) suggests that when correlated to plate load tests on sand, α , in Eq.2 varies between 1.5 to 4 (Table 2).

Based on a review of CPT results for normally consolidated, uncemented sand, Robertson and Campanella (1986) suggested a range for α between 1.3 and 3.0 which agrees with the findings of Schmertmann (1970).

Table 2. Typical α values from static CPTs (CFEM 2006)

Soil Type	α
Silt and sand	1.5
Compact sand	2.0
Dense sand	3.0
Sand and gravel	4.0

For cohesive soils, Senneset et al. (1989) proposed the following relationship:

$$M = \alpha\left(q_t - \sigma_{v0}\right) \qquad [3]$$

where:

σ_{v0} = the total vertical stress corresponding to q_t

The empirical coefficient, α, depends on several factors such as overconsolidation, plasticity, stress history and soil consistency as well as the applied load level. In general practice α in Eq. 2 varies from 5 to 15 for over-consolidated soils and 4 to 8 for normally consolidated soils (Kulhawy and Mayne 1990).

The constrained modulus *M*, is the ratio of applied stress to the measured strain in an oedometer (consolidation) test in which lateral expansion is constrained. However, three dimensional problems associated with footings and mats require elastic modulus *E*. The elastic modulus, *E*, can be obtained from the constrained modulus as follows:

$$E = \frac{(1+\nu)(1-2\nu)}{(1-\nu)} M \qquad [4]$$

where:

ν = Poisson's ratio

For ν = 0, *E* = *M* and for the normal case where ν=0.2, *E* is 90% of *M*. Therefore in genereal practice, they are used somewhat interchangeably (Mayne 1999).

Table 2 shows the preload height and measured settlement for each site in the database. The ultimate settlement was estimated using the hyperbolic method proposed by Tan et al. (1991). The details of this method can also be found in Laws and Catana (2009).

Table 3. Back calculated α values for cohesive soils for the sites in the database

Site	Estimated Ultimate Settlement (mm)	Back calculated α values for cohesive soils(Eq. 3)	α values used for cohesionless soils (Eq. 2)
Bernard Street	966	3.40	3.0
Ellis Street Site 1	1293	3.15	2.0
Ellis Street Site 2	182	n/a*	n/a*
Gordon Drive	539	2.30	2.0
K.L.O. Road Site 1	81	3.45	3.0
K.L.O. Road Site 2	98	3.15**	3.0**
Lake Shore Road	566	2.60	2.5
Lawrence Ave	342	3.80	3.0
Pandosy Street	540	n/a*	n/a*
William R. Bennett Bridge East Approach	236	2.80	2.0
Water Street 1	746	3.50	2.5

* Available data is impacted or not adequate.
** Based on the borehole logs, CPT data from a closeby site was used.

Following the soil type determination for each sublayer, the α values for cohesionless soils were estimated using Robertson and Campanella method (1986) as shown in Table 3. Subsequently, the α values for the cohesive soils were back calculated using Eq.1 and the estimated ultimate settlement (See Table 3).

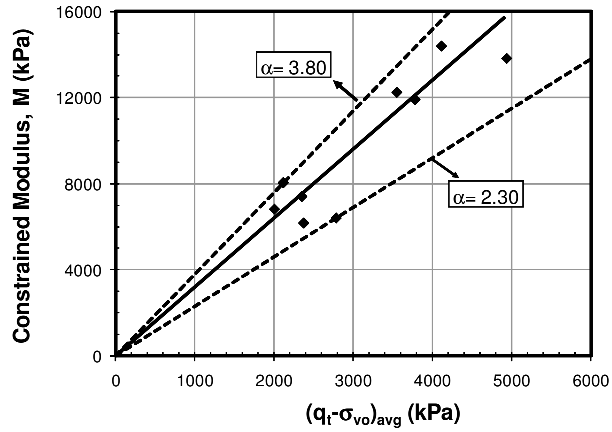

Figure 2. The correlation of Constrained Modulus, M and $(q_t - \sigma_{v0})_{avg}$

For every site in the database, the constrained modulus for each sublayer (i.e.,h=0.05m) was calculated using the site specific CPT data and Eq 2. The critical point in back calculating the α values for cohesive soils was to determine the type of soil for each sublayer. This was accomplished by using borehole logs which were available for most of the sites. In the absence of borehole logs, the soil type estimation was based on CPT data (Robertson and Campanella, 1986).

The back calculated α values for cohesive soils were plotted in Figure 2. Based on the studied sites, the α value ranges from 2.3 to 3.8. Using the average α value of 3.13, the total settlement for each site was calculated to determine the variation of settlement with respect to an average α value.

Figure 3 shows the predicted settlement using the average α value and the estimated ultimate settlement. The results show that, based on the studied sites, settlements in The City of Kelowna area can be predicted with a reasonable accuracy of ±25% using the constrained modulus approach.

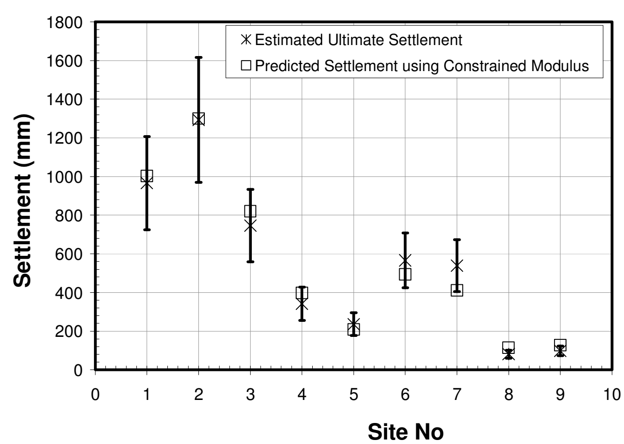

Figure 3. Settlement comparison within ±25% error bars

2.2 Janbu Approach

Janbu (1967) proposed another approach to estimate settlement in early 1960s. His approach combines the basic principle of linear and non-linear stress-strain behaviour and is applicable to both clays and sands.

In his method, Janbu uses two non-dimensional parameters: a stress exponent, *j*, and a modulus number, *m,* to define the stress-strain relation. These non-dimensional parameters *j* and *m* are unique for every soil.

Following the definition of tangent modulus, Janbu proposed the following generic equation:

$$M_t = \frac{\partial \sigma}{\partial \varepsilon} = m\sigma_r \left(\frac{\sigma'}{\sigma_r}\right)^{1-j} \qquad [5]$$

where:

ε = strain induced by increase of effective stress
σ' = effective vertical stress
j = stress exponent
m = a modulus number
σ_r = reference stress equal to 100 kPa

Assuming elastic stress strain behavior for dense, coarse-grained soils, such as glacial till, the stress exponent, *j*, is equal to unity:

$$\varepsilon = \frac{1}{100m}\left(\sigma_1' - \sigma_0'\right) \qquad [6]$$

where:
σ'_0 = original effective stress
σ'_1 = final effective stress

Based on Janbu's approach, the stress exponent, *j*, gradually moves from zero to unity, as the soil gradation changes from clay to gravel. Consequently, values of *j* other than 0 or 1, are used for sandy or silty soils:

$$\varepsilon = \frac{1}{5m}\left(\sqrt{\sigma'_1} - \sqrt{\sigma'_0}\right) \qquad [7]$$

The stress exponent, *j*, for cohesive soils is zero. Therefore, for normally consolidated soils:

$$\varepsilon = \frac{1}{m}\ln\frac{\sigma_1'}{\sigma_0'} \qquad [8]$$

For over-consolidated soils, the equation is as follows:

$$\varepsilon = \frac{1}{m_r}\ln\frac{\sigma_p'}{\sigma_0'} + \frac{1}{m}\ln\frac{\sigma_1'}{\sigma_p'} \qquad [9]$$

where:
σ_p' = pre-consolidation pressure

In addition to the parameters required for a standard settlement calculation; such as effective stress and stress increase, Janbu's method requires one additional parameter, i.e., the modulus number, *m*. Table 4 provides typical range of modulus numbers provided for various different soil types. (Canadian Foundation Engineering Manual, 2006).

Table 4. Typical modulus numbers

Soil Type	Modulus Number	Stress Exponent *j*
Till, very dense to dense	1,000 - 300	1
Gravel	400 - 40	1
Sand		
Dense	400 - 250	0.5
Compact	250 - 150	
Loose	150 -100	
Silt		
Dense	200 - 80	0.5
Compact	80 - 60	
Loose	60 -40	
Silty Clay and Clayey Silts		
Hard, stiff	60 - 20	0
Stiff, firm	20 - 10	
Soft	10 - 5	
Soft Marine Clays and Organic Clays	20 - 5	0
Peat	5 - 1	0

Massarsch (1994) proposed a semi-empirical relationship between modulus number and the cone tip resistance adjusted for depth:

$$m = a\left(\frac{q_{tM}}{\sigma_r}\right)^{0.5} \qquad [10]$$

where:

m = modulus number
a = empirical modulus modifier, which depends on soil type
q_{tM} = stress-adjusted cone tip resistance
σ_r = reference stress, 100 kPa

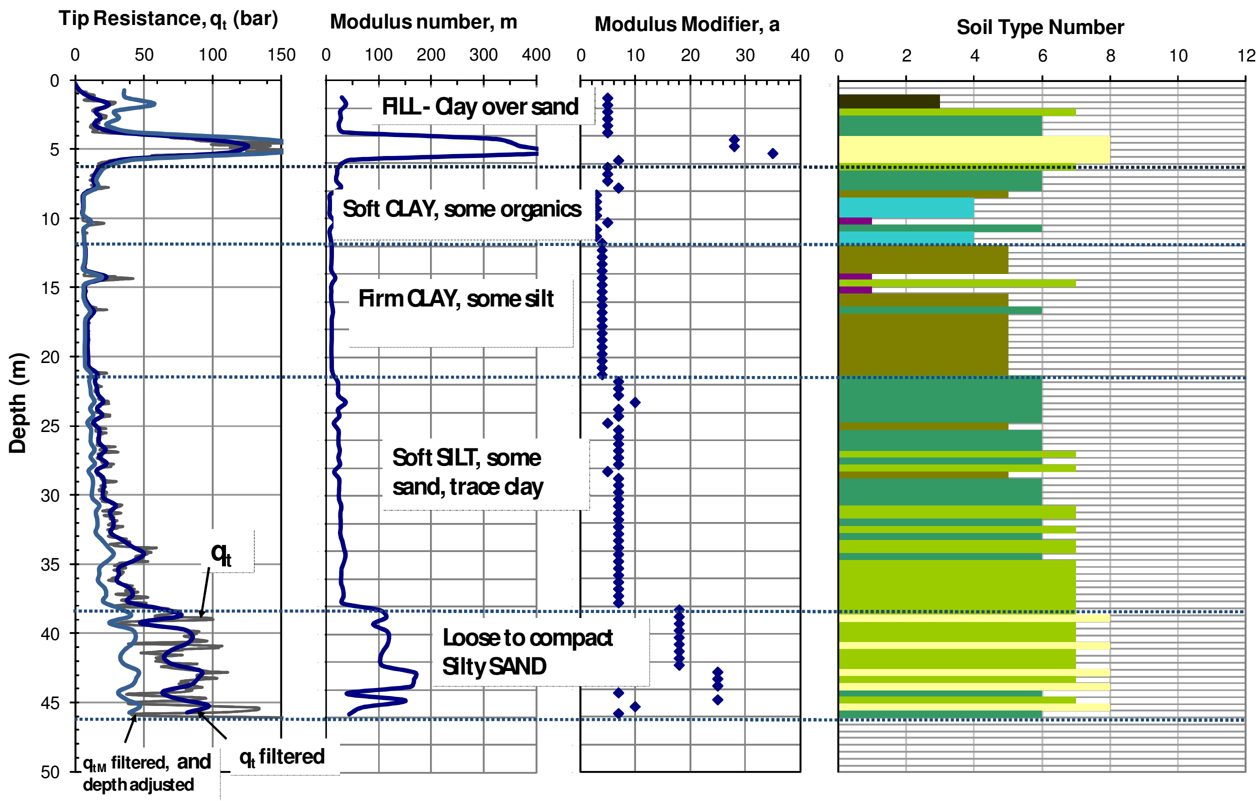

Figure 4. Modulus number and modifier profile for Ellis Street Site

The stress-adjusted cone tip resistance in Eq. 4 is calculated as follows (Massarsch, 1994):

$$q_{tM} = q_t \left(\frac{\sigma_r}{\sigma'_m} \right)^{0.5} \qquad [11]$$

where:

q_t = unadjusted cone tip resistance corrected for pore pressure

σ'_m =mean effective stress

Using Eq. 11 the cone tip resistance is adjusted for depth, such that it can be used for compressibility calculations as the affect of effective overburden stress is accounted for (Jamiolkowski et al. 1988).

Based on his evaluation of field and laboratory data, Massarsch (1994) proposed modulus modifier, *a*, values for different soil types as shown in Table 5. The modulus modifier values provided by Massarsch (1994) have been verified in compacted hydraulic fills but not naturallly deposited soils.

In this study, the modulus modifier values were estimated using the CPT data and the method was calibrated against the studied sites. Figure 4 shows the CPT results for Ellis Street site. The q_t values were filtered using geometric average over 0.5m length and were adjusted for depth using Eq. 11. The modulus modifier values were then calculated using q_{tM} and soil type based on borehole logs and CPT interpretation. Table 5 shows the calibrated modulus modifier values used in this study in comparison with Massarsch (1994) findings.

Table 5. Modulus Modifier, *a*

Soil Type	Massarsch (1994)	Used in this study
Soft Clay	3*	3
Firm Clay	5*	4
Silt, organic soft	7	5
Silt, loose	12	7
Silt, compact	15	10
Silt, dense	20	15
Sand, silty loose	20	18
Sand, loose	22	24
Sand, compact	28	30
Sand, dense	35	35
Gravel, loose	35	35
Gravel, dense	45	45

*Based on limited data on lacustrine and marine clay

The results show that the lacustrine silt deposits behave more like a firm clay, thus, lower modulus modifier values were used for predicting settlement in this study.

Table 6. Predicted settlement using Janbu approach for the sites in the database

Site	Estimated Ultimate Settlement (mm)	Predicted Settlement using Janbu Approach
Bernard Street	966	855
Ellis Street Site 1	1293	1228
Ellis Street Site 2	182	n/a
Gordon Drive	539	564
K.L.O. Road Site 1	81	119
K.L.O. Road Site 2	98	131
Lakeshore Road	566	538
Lawrence Ave	342	428
Pandosy Street	540	n/a*
William R. Bennett Bridge East Approach	236	284
Water Street 1	746	640

* Available data is impacted or not adequate.

Table 6 shows the predicted settlement using the calibrated modulus modifier, *a*, numbers shown in Table 5. The results are also plotted in Figure 5, which shows the settlement of each site was successfully predicted within ±15% of the estimated ultimate settlement.

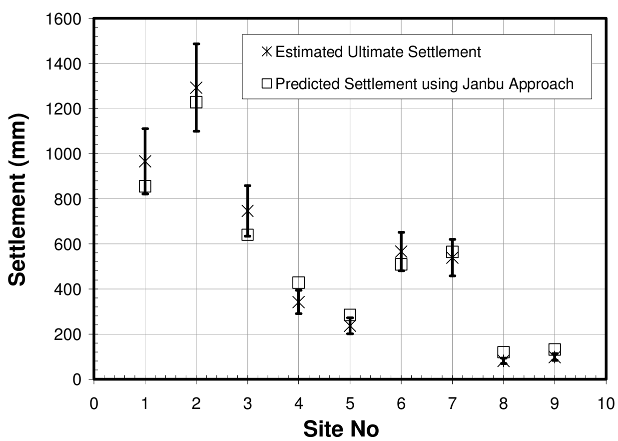

Figure 5. Settlement comparison using Janbu approach with 15% error bars

3 CONCLUSIONS

Both the constrained modulus and Janbu approach provided reasonable settlement predictions when the calibration results of this study were applied. The authors believe that these calibrated methods will be a valuable tool in predicting settlement for the geotechnical engineers practising in Central Okanagan Valley, particularly in Kelowna, BC.

One of the key points that influence the accuracy of settlement prediction using both methods was found to be the determination of the soil type. Therefore, along with CPT data, borehole logs provided valuable information from which the soil type can be properly assessed.

The authors are still in the process of expanding their database and analyzing the available data to further refine these methods of predicting settlement for Central Okanagan Valley, BC.

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