Settlement Prediction Using the Hyperbolic Method and Finite Element Analysis in the Central Okanagan Valley Kelowna, BC

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ABSTRACT
This paper examines three sites situated in The City of Kelowna underlain by extensive normally consolidated soils that have been subjected to preloading during site development and explores the relationships between predicted and actual settlements, using both the hyperbolic method and finite element analysis. Two constitutive soil models, namely the Mohr-Coulomb Model and the Hardening-Soil Model were considered when undertaking settlement prediction by finite element analysis.

RÉSUMÉ
Cet article examine trois emplacements situé dans la ville de Kelowna étée à la base par les sols normalement consolidés étendus qui ont été soumis à preloading pendant le développement d'emplacement et explore les rapports entre les règlements prévus et réels, en utilisant la méthode hyperbolique et l'analyse finie d'élément. Deux modèles constitutifs de sol, à savoir le modèle de Mohr-Coulomb et le modèle de Durcir-Sol ont été considérés en entreprenant la prévision de règlement par analyse finie d'élément.

1 INTRODUCTION

Nasmith (1962) describes in detail how the majority of the City of Kelowna is situated on normally consolidated Holocene deltaic deposits derived from the erosion of the Rutland fan and glacial lake silts by the Kelowna and Mission creeks as the level of Lake Penticton fell towards the end of the last glaciation. In turn, these deposits are underlain by extensive normally consolidated glacial lake silts. In the early thirties an unsuccessful oil and gas well was drilled near where Mission Creek crosses Lakeshore Road to a depth of nearly 1000 m before encountering bedrock as described in Roed et al (1995). These deposits typically comprise interbedded loose to compact silty sands and soft to firm clayey silts. The surficial geology of The City of Kelowna as defined by Nasmith (1962) is presented as Figure 1.

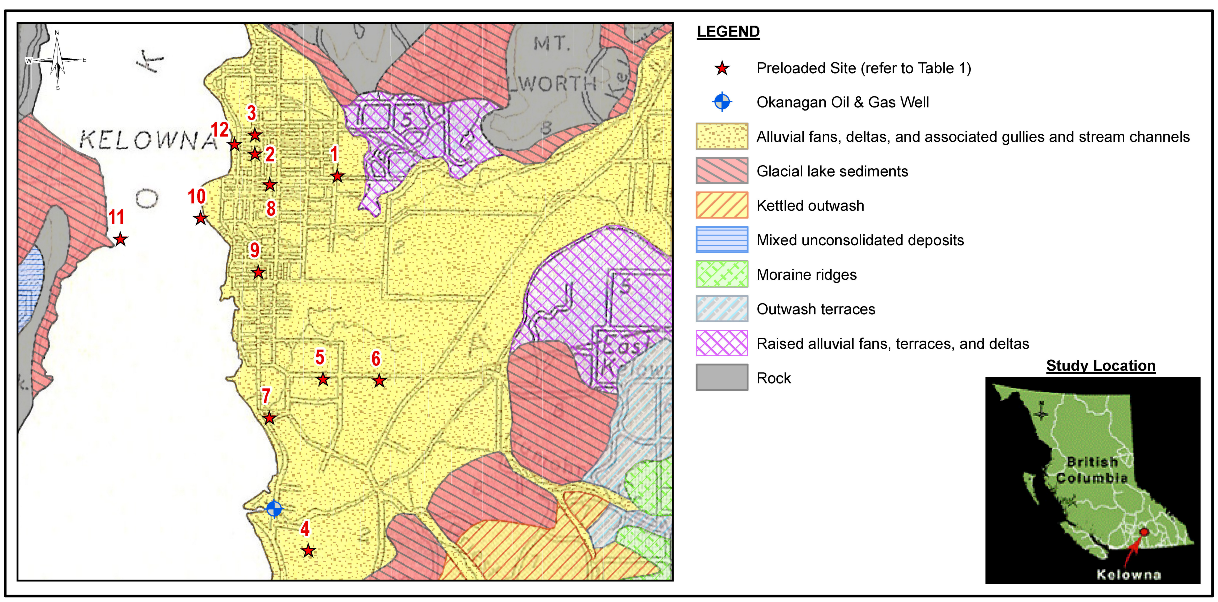

Figure 1. The Surficial Geology of Kelowna from Nasmith (1962)

The extensive thickness of these soil deposits normally excludes piling as a foundation solution and therefore ground improvement options are generally considered in site development. Ground improvement options considered typically would include preloading of the site to induce settlement in softer soils, vibrofloation or vibroreplacment to densify loose soils susceptible to liquefaction, in conjunction with a raft foundation to minimize the stresses applied from structural loading.

Due to the highly variable nature of soils in the Central Okanagan, the accurate prediction of settlements based on data obtained during a typical site investigation can be difficult. This can be due to many factors such as, the subsurface investigational technique utilized, the correlations applied in the estimation of engineering parameters from subsurface data and the multitude of analytical approaches available to practising Geotechnical Engineers. This paper presents several case histories of sites that have been preloaded in Kelowna, and explores the relationships between predicted and actual settlements using both the hyperbolic method and finite element analysis with the aim of assisting Geotechnical Engineers practising in the Central Okanagan to more accurately predict anticipated settlements.

2 PRELOADED STIES

A database of some sites that have been preloaded in Kelowna has been developed as presented in Table 1.

Table 1. Summary of Sites Preloaded in Kelowna.

Site	Location	Maximum Preload Height (m)	Duration of Preload (Days)	Maximum Surface Settlement (mm)
1.	Bernard St	11.0	211	840
2.	Ellis St Site 1	9.5	350	1192
3.	Ellis St Site 2[1.]	3.6	198	162
4.	Gordon Dr[2.]	3.0	168	470
5.	K.L.O. Rd Site 1[2.]	3.3	62	77
6.	K.L.O. Rd Site 2	3.6	306	75
7.	Lakeshore Rd[3.]	4.5	217	507
8.	Lawrence Ave	4.0	103	312
9.	Pandosy St	5.8	301	500
10.	William R. Bennett Bridge East Approach	2.2	566	204
11.	William R. Bennett Bridge West Approach[4.]	2.9	483	1110
12.	Water St	10.0	150	790

1. No shallow settlement plate situated near centre of preload mass

2. Site treated by R.I.C. prior to placement of preload

3. Preload still in place at time of writing

4. Preload placed shortly after completion of causeway.

3 SETTLEMENT PREDICTION USING THE HYPERBOLIC METHOD

The hyperbolic method enables the prediction of total settlement of embankments from field measurements, once sufficient data is available during the early stages of settlement to reach the hyperbolic line. As the field data simultaneously includes primary and secondary consolidation application of the hyperbolic method is considered to give a reasonable approximation of ultimate settlement.

Tan et al (1991) postulated that the relationship between consolidation settlement (s) and time (t) approaches a hyperbolic curve as defined by Equation 1.

$$s_i = \frac{t_i}{\alpha + \beta t_i} \qquad [1]$$

Unique values of α and β can be obtained by plotting t/s versus t where α is defined as the y-axis intercept and β the inverse of the slope of the linear portion of the data as illustrated by Figure 2.

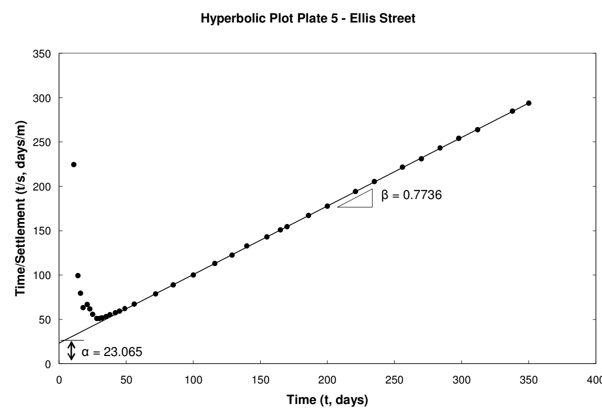

Figure 2. Example of a hyperbolic plot

As time approaches infinity α becomes infinitesimal relative to βt and therefore the ultimate settlement s_u is equal to 1/β.

As the percentage of consolidation U_i is given by s_i/s_u Equation 1 can be re-written to determine the time t_i required to achieve any given percentage of consolidation U_i along the hyperbolic line as shown by Equation 2.

$$t_i = \frac{\alpha s_u U_i}{1 - U_i} \qquad [2]$$

Typically consolidation behaviour approaches the hyperbolic line between 30% to 40% degree of consolidation. Care should be taken when applying the hyperbolic method to field data from embankments that induce relatively small settlements, as these cases can be susceptible to the tolerances of the surveying technique.

The ultimate settlement of the preloaded sites presented in Table 1 has been calculated using the hyperbolic method as presented in Table 2.

Table 2. Calculated Ultimate Settlement of Sites Preloaded in Kelowna by the Hyperbolic Method.

Site	Location	Maximum Preload Height (m)	Calculated Ultimate Settlement (mm)
1.	Bernard St	11.0	966
2.	Ellis St Site 1	9.5	1293
3.	Ellis St Site 2	3.6	182
4.	Gordon Dr	3.0	539
5.	K.L.O. Rd Site 1	3.3	81
6.	K.L.O. Rd Site 2	3.6	98
7.	Lakeshore Rd	4.5	566
8.	Lawrence Ave	4.0	342
9.	Pandosy St	5.8	540
10.	William R. Bennett Bridge East Approach	2.2	236
11.	William R. Bennett Bridge West Approach	2.9	1200
12.	Water St[1.]	10.0	746

[1.] Settlement plate situated beneath centre of preload mass.

4 FINITE ELEMENT ANALYSIS

The Finite Element analysis package PLAXIS 2D v9 (PLAXIS BV) was used to undertake the finite element settlement analysis of the three sites. Two constitutive soil models, namely the Mohr-Coulomb Model and the Plaxis Hardening-Soil Model were used in the analysis.

The soil stratigraphy and engineering parameters utilized in the analysis were estimated from the results of cone penetration tests and boreholes undertaken at each of the sites.

Comparison of predicted versus actual settlement was only undertaken at the centre of the preload mass to minimize the influence of 3D effects.

4.1 The Mohr-Coulomb Model

The Mohr-Coulomb constitutive model simulates elastic soil behaviour until a yield criterion is met at which point perfectly plastic soil behaviour occurs parallel to the yield surface (bi-linear stress strain behaviour) as illustrated by Figure 3.

It is commonly utilized in soil mechanics as it only has five basic parameters that can be easily approximated from data obtained from a typical site investigation. The five basic parameters are, Young's modulus E^{ref}, Poisson's ratio ν, the cohesion intercept c, the friction angle φ, and the dilatancy angle ψ.

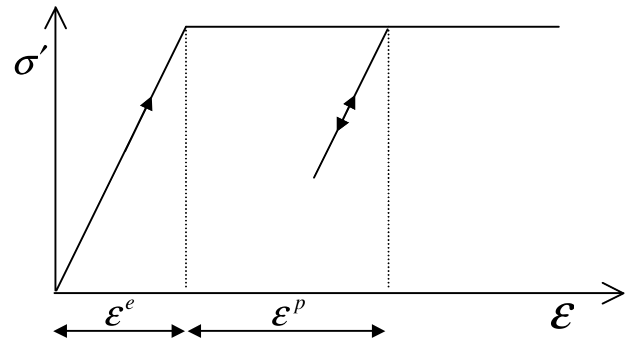

Figure 3 Elastic plastic stress strain behaviour from Brinkgreve et al (2008)

The majority of soils exhibit non-linear stress strain behaviour from the commencement of loading. Subsequently Brinkgreve et al (2008) have recommended the use of the secant modulus $E_{50,}$ which occurs at 50% strength, as opposed to the tangent modulus $E_{0,}$ which is represented by the initial slope of line, as appropriate in the selection of Young's modulus for most problems in soil mechanics, as illustrated by Figure 4.

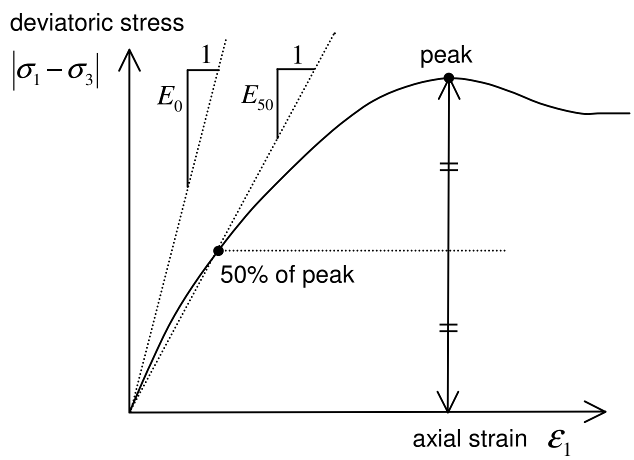

Figure 4. Definitions of E_0 and E_{50} for a standard drained triaxial test result from Brinkgreve et al (2008)

4.2 The Hardening-Soil Model

The Hardening-Soil constitutive model, as described in detail by Schanz et al (1999), simulates non-linear (hyperbolic) stress strain behaviour by permitting plastic strain hardening to occur for both virgin consolidation and unloading/reloading as illustrated by Figure 5. It however does not take into account viscous secondary consolidation effects such as creep and stress relaxation.

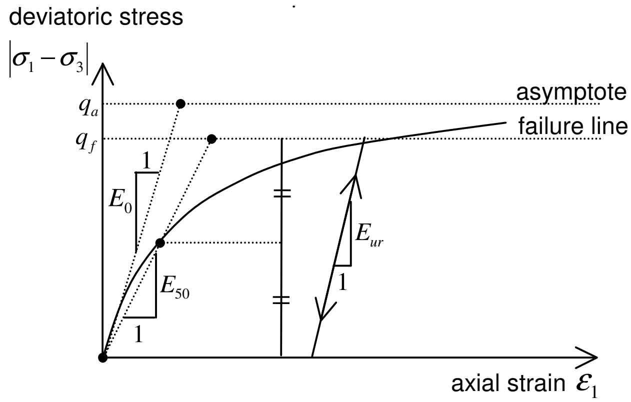

Figure 5. Hyperbolic stress strain behaviour for a standard drained triaxial test result from Brinkgreve et al (2008)

In addition to the Mohr-Coulomb model strength parameters c, φ, and ψ, five stiffness parameters are now required to formulate the Hardening-Soil Model, namely E_{50}^{ref} (the secant stiffness in the drained triaxial test), E_{oed}^{ref} (the tangent stiffness for primary oedometer loading), E_{ur}^{ref} (the unloading reloading stiffness), m the power exponent for the stress level dependency of stiffness and ν_{ur} Poisson's ratio for unloading reloading. The stress strain behaviour for primary loading is simulated using Equation 3, the oedometer stiffness for primary loading using Equation 4 and unloading and reloading stiffness using Equation 5.

$$E_{50} = E_{50}^{ref}\left(\frac{\sigma_3 + c\cot\varphi_p}{\sigma^{ref} + c\cot\varphi_p}\right)^m \qquad [3]$$

$$E_{oed} = E_{oed}^{ref}\left(\frac{\sigma_1 + c\cot\varphi_p}{\sigma^{ref} + c\cot\varphi_p}\right)^m \qquad [4]$$

$$E_{ur} = E_{ur}^{ref}\left(\frac{\sigma_3 + c\cot\varphi_p}{\sigma^{ref} + c\cot\varphi_p}\right)^m \qquad [5]$$

Where σ^{ref} is the reference stress for stiffness and φ_p is the angle of friction at the reference stress.

The power function m ranges between 0.5 for sandy soils and 1.0 for clayey soils and the unloading reloading Poisson's ratio ν_{ur} ranges between 0.15 and 0.25, Brinkgreve et al (2008).

In the absence of any applicable laboratory testing data it is standard practice to adopt a value of E_{50}^{ref} & E_{oed}^{ref} that equate to E^{ref} in the Mohr-Columb model and E_{oed}^{ref} of three times the value of E^{ref} at the reference stress.

4.3 Calculation of Initial Stresses

The calculation of the initial in-situ stresses for each of the analyses was undertaken in accordance with the K_0 procedure. For a normally consolidated soil, the value of K_0 is assumed to be related to the friction angle and is determined with the application of Equation 6.

$$K_0 = 1 - \sin\varphi \qquad [6]$$

5 ESTIMATION OF SOIL PARAMETERS FROM CPT DATA

5.1 Effective Strength Parameters

Numerous methods have been proposed in the estimation of effective stress parameters from CPT data, such as empirical or semi-empirical correlations based on calibration chamber tests, bearing capacity theory and cavity expansion theory.

Senneset et al (1982, 1989) in Lunne et al (1997) developed an effective stress bearing capacity interpretation method that can be used for the development of effective strength parameters of both fine and coarse grained soils. The bearing capacity formula in terms of effective overburden stress is expressed as Equation 7.

$$q_t - \sigma_{vo} = N_m(\sigma'_{vo} + a) \qquad [7]$$

Where:

$$N_m = \frac{N_q - 1}{1 + N_u B_q}$$

$$B_q = \frac{\Delta u}{(q_t - \sigma_{vo})}$$

$$N_q = \tan^2(45 + \varphi'½)e^{(\pi - 2\beta)\tan\varphi'}$$

$$N_u \approx 6\tan\varphi'(1 - \tan\varphi')$$

Δu = excess pore pressure

a = attraction

β = angle of plastification

Typical values of soil attraction and peak secant friction for various soil types are presented in Table 3.

Table 3. Typical values of soil attraction and friction.

Soil type	Shear strength parameters			
	a (kPa)	φ' (°)	N_m	B_q
Clay, soft	5-10	19-24	1-3	0.8-1.0
Clay, medium	10-20	19-29	3-5	0.6-0.8
Clay, stiff	20-50	27-31	5-8	0.3-0.6
Silt, soft	0-5	27-31		
Silt, medium	5-15	29-33	5-30	0-0.4
Silt, stiff	15-30	31-35		
Sand, loose	0	29-33		
Sand, medium	10-20	31-37	30-100	<0.1
Sand, dense	20-50	35-42		
Hard, stiff soil, OC, cemented	>50	38-45	100	<0

5.2 Dilatancy of Sands

The dilatancy of sands typically range from -2° for very loose sands to 14° for very dense sands, Bolton (1986).

Lee et al (2008) proposed a direct correlation method to estimate the dilatancy of sands as expressed by Equation 8.

$$\psi = \frac{1}{a}\left(\frac{q_c / \sigma'_{h0}}{b}\right) \qquad [8]$$

Where:

$$a = 0.135 K_0^{-0.115}$$

$$b = 64.09 K_0^{-0.17}$$

5.3 Elastic Parameters

Various authors have published correlations to estimate constrained modulus values, M, directly from the cone resistance, q_c.

The elastic modulus, E can be obtained from the constrained modulus by the use of Equation 9.

$$E = \frac{(1+\nu)(1-2\nu)}{(1-\nu)} M \qquad [9]$$

Mitchell and Garner (1975) in Lunne et al (1997) developed correlations for cohesive soil as presented in Table 4.

Table 4. Estimation of constrained modulus for cohesive soils.

Soil type	q_c (MPa)	M (MPa)
Clays of low plasticity (CL)	$q_c < 0.7$	$3 q_c < M < 8 q_c$
	$0.7 < q_c < 2.0$	$2 q_c < M < 5 q_c$
	$q_c > 2.0$	$1 q_c < M < 2.5 q_c$
Silts of low plasticity (ML)	$q_c < 2.0$	$3 q_c < M < 6 q_c$
	$q_c > 2.0$	$1 q_c < M < 3 q_c$
Highly plastic silts and clays (MH,CH)	$q_c < 2.0$	$2 q_c < M < 6 q_c$

Lunne and Christopersen (1983) in Lunne et al (1997) developed correlations for silica sands as presented in Table 5.

Table 5. Estimation of constrained modulus for silica sands.

q_c (MPa)	M (MPa)
$q_c < 10$	$M = 4 q_c$
$10 < q_c < 50$	$M = 2 q_c + 20$
$q_c > 50$	$M = 120$

Senneset et al (1988) as in Lunne et al (1997) developed correlations for intermediate soils as presented in Table 6.

Table 6. Estimation of constrained modulus for intermediate soils.

q_c (MPa)	M (MPa)
$q_c < 2.5$	$M = 2 q_c$
$2.5 < q_c < 5$	$M = 4 q_c - 5$

Where a range of values is given for a particular soil type to estimate the constrained modulus, the average value was adopted for the sites considered in this paper.

The estimation of elastic parameters from CPT results for soils in the Central Okanagan Valley is discussed in more detail in Catana and Laws (2009).

5.4 Hydraulic Conductivity and Unit Weight

In the absence of site specific data, such as from adjacent boreholes or local experience, Lunne et al (1997) provides estimates of soil hydraulic conductivity and units weights based on the soil behaviour type classification system proposed by Robertsen et al (1986) in Lunne et al (1997) as shown in Table 7.

Table 7. Estimation of hydraulic conductivity *k* and unit weight based on soil behavior.

Zone (Soil Behaviour type)	Approximate unit weight (kN/m^3)	Range of hydraulic conductivity *k* (m/s)
1 (Sensitive fine grained)	17.5	3×10^{-9} to 3×10^{-8}
2 (Organic material)	12.5	1×10^{-8} to 1×10^{-6}
3 (Clay)	17.5	1×10^{-10} to 1×10^{-9}
4 (Silty clay to clay)	18	1×10^{-9} to 1×10^{-8}
5 (Clayey silt to silty clay)	18	1×10^{-8} to 1×10^{-7}
6 (Sandy silt to clayey silt)	18	1×10^{-7} to 1×10^{-6}
7 (Silty sand to sandy silt)	18.5	1×10^{-5} to 1×10^{-6}
8 (Sand to silty sand)	19	1×10^{-5} to 1×10^{-4}
9 (Sand)	19.5	1×10^{-4} to 1×10^{-3}
10 (Gravelly sand to sand)	20	1×10^{-3} to 1
11 (Very stiff fine grained[1.])	20.5	1×10^{-9} to 1×10^{-7}
12 (Sand to clayey sand[1.])	19	1×10^{-8} to 1×10^{-6}

[1.] Overconsolidated or cemented.

Comparison of soil behaviour type interpreted from CPT results with adjacent boreholes for soils in the Central Okanagan Valley is discussed in more detail in Catana and Laws (2009).

6 CASE HISTORIES

6.1 Bernard Street

The Bernard Street site preload mass comprised a truncated L shaped pyramid with a basal width of approximately 45.0 m, a length of approximately 75.0 m in both directions and a height of 11.0 m. It was surcharged for a duration of 211 days with a maximum settlement of 840 mm recorded near the centre of the preload mass. Limited time history data of the construction of the preload mass was available to the authors. For the purposes of modelling it was assumed that construction of the preload mass commenced 8 days after the installation of the settlement plates and was constructed at a constant rate, achieving the maximum height at 25 days.

One CPT was advanced near the centre of the site to a maximum depth of 39 m with a total of 5 distinct soil types identified as summarized in Table 8.

Table 8. Summary of Subsurface Conditions Encountered in Bernard Street Site

Unit	Description	Depth Zone (m)	Cone Resistance q_c (MPa)
Unit 1	SAND	0.0 to 3.0, 7.0 to 8.75, 10.25 to 13	1.5 to 8.5
Unit 2	Sandy GRAVEL	3.0 to 7.0	> 15.0
Unit 3	Clayey SILT/ CLAY	8.75 to 10.25, 13.0 to 14.0	0.7 to 3.0
Unit 4	Silty SAND	14.0 to 17.75, 37.0 to 39.0	5.0 to >15.0
Unit 5	SILT	20.5 to 30.5	1.5 to 4.5

The geometry of the preload mass and soil profile is illustrated in Figure 6.

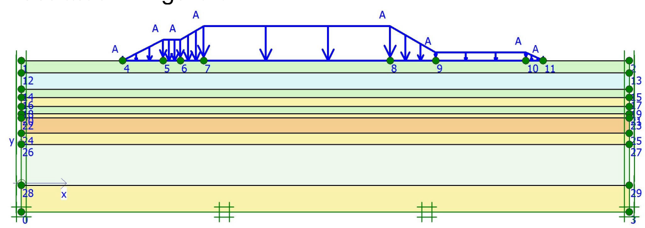

Figure 6. Preload geometry and soil profile Bernard Street

6.1.1 Hyperbolic Method Analysis

A hyperbolic method analysis of the data obtained from the settlement plate beneath the centre of the preload mass resulted in α and β values of 27.008 and 1.0352 respectively, and a predicted ultimate settlement of 967 mm as illustrated by Figure 7.

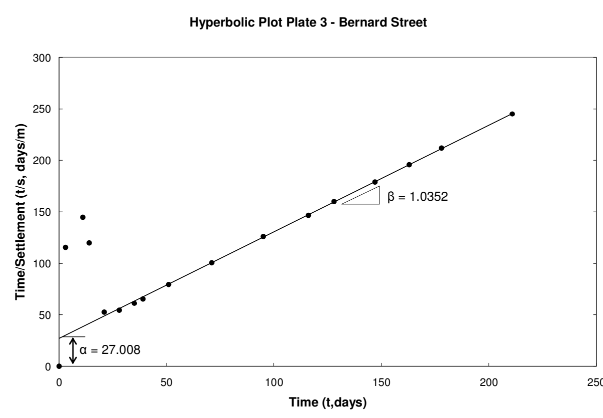

Figure 7. Hyperbolic plot Bernard Street

6.1.2 Finite Element Soil Parameters

The parameters utilized in the finite element analysis of the Bernard Street site are summarized in Tables 9 & 10.

Table 9. Summary of Mohr-Coulomb Model Soil Parameters Bernard Street Site

Parameter	Unit 1	Unit 2	Unit 3	Unit 4	Unit 5
γ_{unsat} (kN/m^3)	17.0	18.0	16.0	16.0	16.0
γ_{sat} (kN/m^3)	19.0	20.0	18.5	18.0	18.0
c (kPa)	0.5	0.5	2	0.5	3
φ (°)	28	38	28	32	30
ψ (°)	2	2	0	2	0
E^{ref} (MPa)	7	25	2.25	16	2.75
ν	0.3	0.3	0.3	0.33	0.33
k (m/day)	100	100	0.0005	0.1	0.0005

Table 10. Summary of Hardening-Soil Model Soil Parameters Bernard Street Site

Parameter	Unit 1	Unit 2	Unit 3	Unit 4	Unit 5
γ_{unsat} (kN/m^3)	17.0	18.0	16.0	16.0	16.0
γ_{sat} (kN/m^3)	19.0	20.0	18.5	18.0	18.0
c (kPa)	0.5	0.5	2	0.5	3
φ (°)	28	38	28	32	30
ψ (°)	2	2	0	2	0
E_{50}^{ref} (MPa)	7	25	2.25	16	2.75
E_{oed}^{ref} (MPa)	7	25	2.25	16	2.75
E_{ur}^{ref} (MPa)	21	75	6.75	48	8.25
m	0.5	0.5	1.0	0.6	0.9
ν_{ur}	0.2	0.2	0.2	0.2	0.2
k (m/day)	100	100	0.0005	0.1	0.0005

6.1.3 Comparison of Analyses

Time-settlement data was obtained at the location of the settlement plate beneath the centre of the preload mass for both soil models considered and plotted against the actual field measurements as presented in Figure 8.

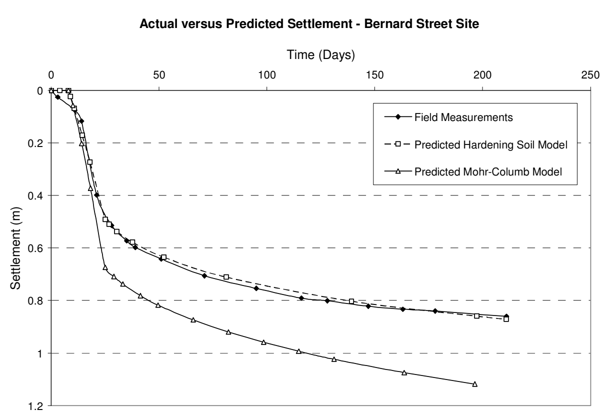

Figure 8. Comparison of predicted finite element settlement versus actual field measurements Bernard St.

The finite element analyses were run until the completion of settlement for comparison with the ultimate settlement predicted by the hyperbolic method as summarized in Table 11.

Table 11. Summary of Predicted Settlement Bernard Street

Method	Settlement at 211 Days (mm)	Ultimate Settlement (mm)
Hyperbolic Method	860	966
Mohr-Coulomb Model	1136	1396
Hardening-Soil Model	871	961
Field Measurements	861	n/a

6.2 Ellis Street

The Ellis Street site preload mass comprised a truncated rectangular shaped pyramid with a basal width of approximately 45.0 m, a length of approximately 48.0 m and a height of 9.5 m. It was surcharged for a duration of 350 days with a maximum settlement of 1192 mm recorded near the centre of the preload mass. Limited time history data of the construction of the preload mass was available to the authors. For the purposes of modelling it was assumed that construction of the preload mass commenced 10 days after the installation of the settlement plates and was constructed at a constant rate, achieving the maximum height at 28 days.

One CPT was advanced near the centre of the site to a maximum depth of 46 m with a total of 5 distinct soil types identified as summarized in Table 12.

Table 12. Summary of Subsurface Conditions Encountered at Ellis Street Site

Unit	Description	Depth Zone (m)	Cone Resistance q_c (MPa)
Unit 1	CLAY (FILL)	0.0 to 2.8	0.8 to 2.9
Unit 2	Sandy SILT/SAND	2.8 to 8.0	1.0 to 15.0
Unit 3	Clayey SILT	8.0 to 20.8	0.5 to 1.0
Unit 4	Sandy SILT/ Clayey SILT	20.8 to 34.0	1.4 to 3.8
Unit 5	Silty SAND	34.0 to 46.0	2.4 to 13.5

The geometry of the preload mass and soil profile is illustrated in Figure 9.

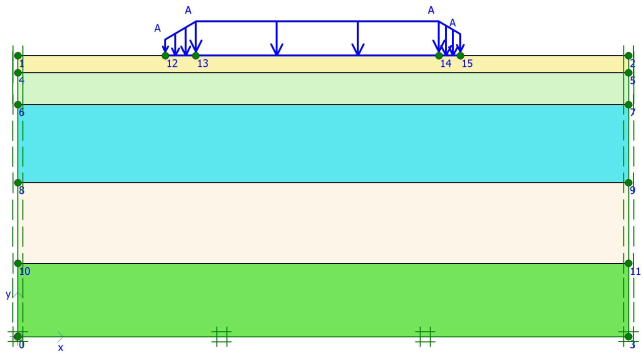

Figure 9. Preload geometry and soil profile Ellis Street Site

6.2.1 Hyperbolic Method Analysis

A hyperbolic method analysis of the data obtained from the settlement plate beneath the centre of the preload mass resulted in α and β values of 23.065 and 0.7736 respectively, and a predicted ultimate settlement of 1293 mm as illustrated by Figure 10.

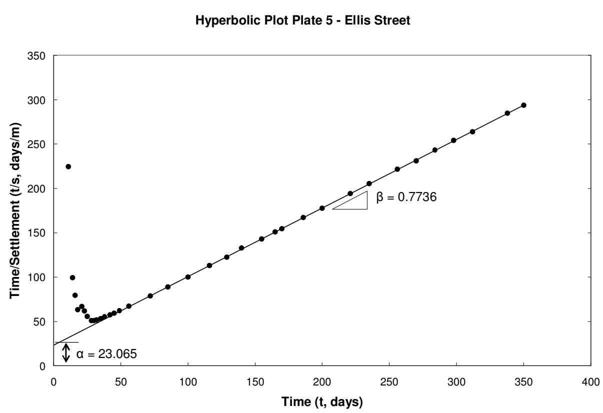

Figure 10. Hyperbolic plot Ellis Street

6.2.2 Finite Element Soil Parameters

The parameters utilized in the finite element analysis of the Ellis Street site are summarized in Tables 13 & 14.

Table 13. Summary of Mohr-Coulomb Model Soil Parameters Ellis Street Site

Parameter	Unit 1	Unit 2	Unit 3	Unit 4	Unit 5
γ_{unsat} (kN/m^3)	16.0	18.0	15.0	16.5	18.0
γ_{sat} (kN/m^3)	18.0	20.0	17.0	18.5	20.0
c (kPa)	8	0.5	8	5	0.5
φ (°)	32	32	28	32	34
ψ (°)	0	2	0	0	2
E^{ref} (MPa)	10	16	2.5	5	8
ν	0.33	0.3	0.33	0.33	0.3
k (m/day)	0.0005	0.5	0.0005	0.005	0.1

Table 14. Summary of Hardening-Soil Model Soil Parameters Ellis Street Site

Parameter	Unit 1	Unit 2	Unit 3	Unit 4	Unit 5
γ_{unsat} (kN/m^3)	16.0	18.0	15.0	16.5	18.0
γ_{sat} (kN/m^3)	18.0	20.0	17.0	18.5	20.0
c (kPa)	8	0.5	8	5	0.5
φ (°)	32	32	28	32	34
ψ (°)	0	2	0	0	2
E_{50}^{ref} (MPa)	10	16	2.5	5	8
E_{oed}^{ref} (MPa)	10	16	2.5	5	8
E_{ur}^{ref} (MPa)	30	48	7.5	15	24
m	1.0	0.6	0.8	0.8	0.6
ν_{ur}	0.2	0.2	0.2	0.2	0.2
k (m/day)	0.0005	0.5	0.0005	0.005	0.1

6.2.3 Comparison of Analyses

Time-settlement data was obtained at the location of the settlement plate beneath the centre of the preload mass for both soil models considered and plotted against the actual field measurements as presented in Figure 11.

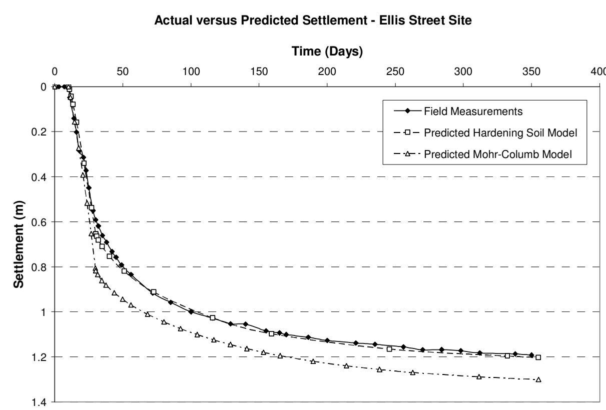

Figure 11. Comparison of predicted finite element settlement versus actual field measurements Ellis Street

The finite element analyses were run until the completion of settlement for comparison with the ultimate settlement predicted by the hyperbolic method as summarized in Table 15.

Table 15. Summary of Predicted Settlement Ellis Street

Method	Settlement at 350 Days (mm)	Ultimate Settlement (mm)
Hyperbolic Method	1191	1293
Mohr-Coulomb Model	1301	1332
Hardening-Soil Model	1203	1224
Field Measurements	1192	n/a

6.3 Water Street

Howie (1994) presents the case history of the ground improvement undertaken for the Delta Grand Okanagan Resort on Water Street. which included a preload mass comprising a truncated rectangular shaped pyramid with a basal width of approximately 65.0 m, a length of approximately 80.0 m and a height of 10.0 m. Placement of the preload was completed in 18 days and it was left in place for a total of 150 days. Upon initial completion of the preload it was discovered that it was partially occupying an incorrect footprint and it was subsequently readjusted. A maximum settlement of 790 mm was recorded, however, only 680 mm occurred at the centre of the preload mass.

Several CPT's were advanced within the proposed building footprint to a maximum depth of approximately 40 m with a total of 5 distinct soil types identified as summarized in Table 16.

Table 16. Summary of Subsurface Conditions Encountered at Water Street Site

Unit	Description	Depth Zone (m)	Cone Resistance q_c (MPa)
Unit 1	SAND (FILL)	0.0 to 4.0	3.5 to 11.0
Unit 2	SAND	4.0 to 16.0	2.0 to 8.0
Unit 3	SAND/Silty SAND	16.0 to 24.3	4.0 to 10.0
Unit 4	CLAY	24.3 to 29.0	1.5 to 2.0
Unit 5	SILT	29.0 to 40.0	3.5 to 8.0

The geometry of the preload mass and soil profile is illustrated in Figure 12.

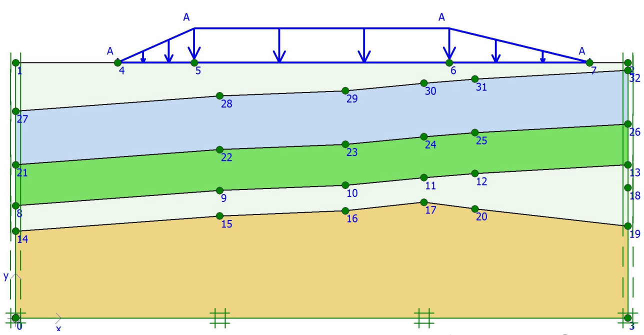

Figure 12. Preload geometry and soil profile Water Street Site

6.3.1 Hyperbolic Method Analysis

A hyperbolic method analysis of the data obtained from the settlement plate beneath the centre of the preload mass resulted in α and β values of 16.964 and 1.341 respectively, and a predicted ultimate settlement of 746 mm as illustrated by Figure 13.

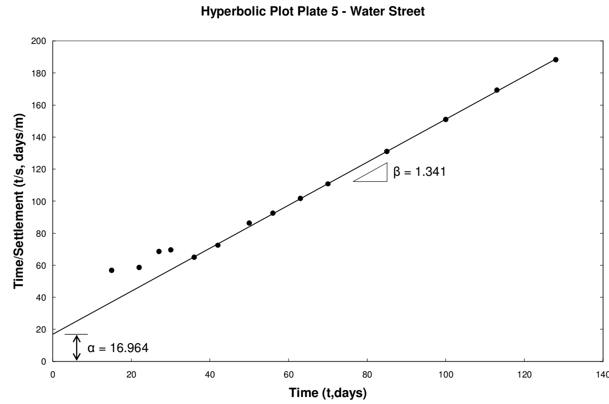

Figure 13. Hyperbolic plot Water Street

6.3.2 Finite Element Soil Parameters

The parameters utilized in the finite element analysis of the Water Street site are summarized in Tables 17 & 18.

Table 17. Summary of Mohr-Coulomb Model Soil Parameters Water Street Site

Parameter	Unit 1	Unit 2	Unit 3	Unit 4	Unit 5
γ_{unsat} (kN/m^3)	18.0	17.0	16.0	16.0	16.0
γ_{sat} (kN/m^3)	20.0	19.0	18.0	18.5	18.0
c (kPa)	0.5	0.5	0.5	8	3
φ (°)	34	28	32	28	30
ψ (°)	2	2	2	0	0
E^{ref} (MPa)	12	6	8	2.5	4.5
ν	0.3	0.3	0.3	0.33	0.33
k (m/day)	100	100	0.1	0.0005	0.0005

Table 18. Summary of Hardening-Soil Model Soil Parameters Water Street Site

Parameter	Unit 1	Unit 2	Unit 3	Unit 4	Unit 5
γ_{unsat} (kN/m^3)	18.0	17.0	16.0	16.0	16.0
γ_{sat} (kN/m^3)	20.0	19.0	18.0	18.5	18.0
c (kPa)	0.5	0.5	0.5	8	3
φ (°)	34	28	32	28	30
ψ (°)	2	2	2	0	0
E_{50}^{ref} (MPa)	12	6	8	2.5	4.5
E_{oed}^{ref} (MPa)	12	6	8	2.5	4.5
E_{ur}^{ref} (MPa)	36	18	24	7.5	13.5
m	0.5	0.5	0.6	1.0	0.8
ν_{ur}	0.2	0.2	0.2	0.2	0.2
k (m/day)	100	100	0.1	0.0005	0.0005

6.3.3 Comparison of Analyses

Time-settlement data was obtained at the location of the settlement plate beneath the centre of the preload mass for both soil models considered and plotted against the actual field measurements as presented in Figure 14.

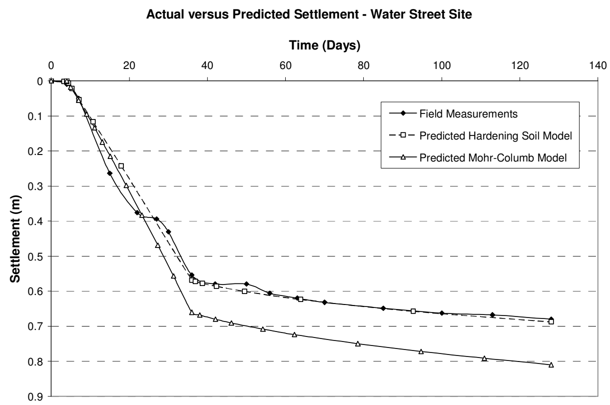

Figure 14. Comparison of predicted finite element settlement versus actual field measurements Water St.

The finite element analyses were run until the completion of settlement for comparison with the ultimate settlement predicted by the hyperbolic method as summarized in Table 19.

Table 19. Summary of Predicted Settlement Water Street

Method	Settlement at 128 Days (mm)	Ultimate Settlement (mm)
Hyperbolic Method	679	746
Mohr-Coulomb Model	810	1025
Hardening-Soil Model	688	780
Field Measurements	680	n/a

7 CONCLUSIONS

In general the time settlement history plot for the Hardening-Soil constitutive model provides good agreement with the observed field measurements and the use of this constitutive model when predicting the behaviour of preload embankments in the Central Okanagan is recommended.

Discrepancies between the initial part of the predicted settlement curve and observed field measurements can be likely attributed to the non-linear application of the preload mass and 3D effects.

For the three case histories considered the predicted ultimate settlement calculated from the Hardening-Soil analysis was within 6% of that predicted when applying the hyperbolic method to the field data. This could be attributed to the 'linearity' of the later time settlement data for both the Hardening-Soil analysis and hyperbolic plot. Consequently the hyperbolic method is considered to provide a good prediction of the ultimate settlement of preload embankments in the Central Okanagan.

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REFERENCES

Bolton, M.D. 1986. The Strength and Dilatancy of Sands, *Geotechnique* 36, No. 1, 65-78.

Brinkgreve R.B.J., Broere W. and Waterman D. 2008. Plaxis 2D Material Models Manual Version 9, *A.A. Balkema,* Rotterdam.

Catana, M.C., and Laws, M.J. 2009. Settlement Prediction Using CPT Data in the Central Okanagan Valley, Kelowna, B.C. *Proceedings 62nd Canadian Geotechnical Conference,* Halifax.

Howie, J.A., Jinks, A.R. and Sladen, J.A. 1994. The Grand Okanagan Lake Front Resort and Conference Centre, Kelowna, B.C., *Proceedings 47th Canadian Geotechnical Conference,* Halifax 200-206.

Lee, J., Eun, J., Lee, K., Park, Y. and Kim, M. 2008. In-Situ Evaluation of Strength and Dilatancy of Sands Based on CPT Results, *Soils and Foundations* Vol. 48, N0. 2, 255-265, Japanese Geotechnical Society.

Lunne, T., Robertson, P.K. and Powell, J.J.M. 1997. Cone Penetrations Testing in Geotechnical Practice, Spon Press, London.

Nasmith, H. 1962. Late Glacial History and Surficial Deposits of the Okanagan Valley British Columbia, *British Columbia Department of Mines and Petroleum Resources*, Bulletin No. 46.

Roed, M.A. and Greenough, J. D., Editors 2004. Okanagan Geology British Columbia, *Kelowna Geology Committee,* Ehnmann Printworx Ltd, Kelowna BC V1Y 7S3.

Schanz, T., Vermeer, P.A. and Bonnier, P.G. 1999. The Hardening-Soil model: Formulation and verification, *Beyond 2000 in Computational Geotechnics – 10 Years of PLAXIS*, Balkema, Rotterdam.

Tan, T.S. and Lee, S.L. 1991. Hyperbolic Method for Consolidation Analysis, *Journal of Geotechnical Engineering*, ASCE, 117: 1723-1737.