Comparison of the nonlinear modulus of the Cambridge high-resolution pressuremeter in Clearwater Formation Clayshales

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ABSTRACT
This paper presents the results of thirteen pressuremeter tests conducted in Clearwater Formation Clayshale at Syncrude's oil sands operations near Fort McMurray. This paper performs an interpretation of nonlinear stiffness data from undrained pressuremeter tests. The behavior of the secant shear modulus for several shear strain values is analyzed and a comparison of the nonlinear stiffness is made using London clay as a reference. In addition, the initial modulus obtained with shear wave velocity values derived from seismic tests is included. Finally, the stiffness parameters for the Clearwater clayshale are presented.

RÉSUMÉ
Cet article présente les résultats de treize essais pressiométriques menés dans les schistes argileux de la formation Clearwater, dans les exploitations de sables bitumineux de Syncrude, près de Fort McMurray. Cet article interprète les données de rigidité non linéaire provenant d'essais pressiométriques non drainés. Le comportement du module de cisaillement sécant pour plusieurs valeurs de déformation de cisaillement est analysé et une comparaison de la rigidité non linéaire est effectuée en utilisant l'argile de London comme référence. En plus, le module initial obtenu avec des valeurs de vitesse d'onde de cisaillement dérivées d'essais sismiques est inclus. Les paramètres de rigidité de l'argilite de Clearwater sont enfin présentés.

1 INTRODUCTION

Standard practice for analyzing the stability of mine pit walls within the oil sands is to run limit equilibrium analysis (LE) using conventional soil parameters along with application of the observational method. LE outputs do not permit the prediction of displacements associated with cut slopes that would be an indication of the onset of undesired performance. In addition, LE models are not capable of showing progressive failure within cut slopes or adequately deal with stress rotations that occur over time. The criteria that relate instrumentation readings to mitigative actions meant to minimize and control against poor performance were developed over the 35+ year history of mining in the oil sands and observations of behavior due to unloading. With the advancement of computational power, the ability to measure in-situ stress-strain characteristics, and mines advancing into locations markedly different than the pits that led to the development of the instrumentation criteria, application of finite element (FE) numerical models is the future.

The accuracy of calculated displacements for slopes and walls from FE models of is typically poor when compared to measured values. Inaccuracies have been attributed to the general simplicity of most linear elastic / perfectly plastic, constitutive models. Because deformations prior to yielding can be considerable, the assumption of linear elasticity is often invalid and tends to underestimate the pre-yield deformations. As such, Burland (1989), Simpson (1992), Benz et al (2009) and Clayton (2011) demonstrate that the use of non-linear elasto-plastic constitutive models, tend to better predict lateral deformations when compared to conventional linear elastic-perfectly plastic models. The conventional non-linear models like the Hardening Soil (HS) model (Schanz et al, 2009) however, require inputs from advanced triaxial testing. These data are easily acquired for most soils, however in HOC (heavily overconsolidated) soils, sampling and testing is not straight forward.

Stiff-fissured clays are typically characterised by their brittle behaviour during laboratory testing. However, in practice, the reliability of the measured strength and stiffness results can be greatly reduced through sample disturbance. Skempton (1964, 1970, 1977, 1985), Bjerrum (1967), Wroth (1984), Fahey (1998), Whittebolle (1982) and many more have shown for fissured HOCs it is difficult to obtain a representative sample that is reliable for laboratory strength measurements. Fissures represent discontinuities within the soil mass, and sample sizes may be inadequate to fully capture the overall macrostructure of the soil. If however a series of macro or micro fissures are captured in a given test specimen, then preparation of the sample for testing is often fraught with difficulties and prone to pre-mature failure. As a result, it is typical for one to select only intact samples for testing. The resulting strength values generally represent the upper bound for the soil mass, while the overall material strength is in actuality closer to the residual value and controlled by the spacing and persistence of the fissures.

The clay shales characteristic of the Canadian Prairies are generally closer in nature to HOC soils as opposed to typical shales that are generally indurated and of higher quality. Certainly, within the upper reaches of the shale within Alberta, the clayshale is considered a bedrock by age only and, if classified using ISRM standards based on Uniaxial Compressive Strength (UCS), would be considered an R0 to R1 material. An example of the distribution of soil undrained shear strength, s_u versus the UCS strength of various rocks is shown below in Figure 1.

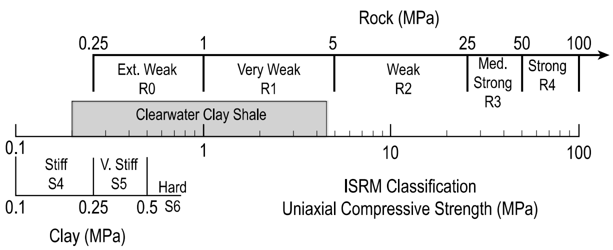

Figure 1. Distribution of soil shear strength to ISRM UCS classification of rocks.

Since stiff, fissured HOC clays present significant difficulties with sampling and testing, their nature is highly conducive to testing with various in-situ test methods. Wroth and Hughes (1973) first presented the development of the Cambridge style Self-Boring pressuremeter (SBPM). Since its inception, Cambridge In-situ has advanced the design and resolution of the probe to be capable of accurately measuring the shear modulus of a soil directly. Bellotti et al. (1989), Muir Wood (1990), Jardine (1991 & 1992), and Bolton and Whittle (1999) all demonstrate the use of the SBPM for evaluation of the non-linear shear modulus of a soil based on unload-reload cycles.

Syncrude has recently carried out an extensive field program within the Clearwater Shale using a high resolution pressuremeters operated by Cambridge Insitu. Each test was evaluated for both undrained strength parameters and linear secant and non-linear secant shear moduli. In addition to the PM testing, downhole seismic testing (DST) was carried out to determine the very small strain shear modulus at each PM test interval. DST was conducted following completion of the pressuremeter program and consisted of vertical seismic profiling using 70 mm diameter slope indicator casings installed into the pressuremeter test holes. All DST data acquisition and evaluation of the corresponding shear wave velocities was conducted by ConeTec Investigations Ltd.

The assessment of displacement is associated with soil stiffness and loading rate for a given soil. Research conducted originally by Hardin and Drnevich (1972) and later supported by Jardine et al. (1986), Jardine (1991) and Burland (1989), show that HOC stiffness decreases with increasing strain, showing a non-linear behaviour for shear strains generally less than 1%. Atkinson & Sallfors (1991) illustrate the use of non-linear elasticity in typical geotechnical engineering practice. Figure 2 provides a summary of typical stiffness-strain behaviour of soils with typical strain ranges for laboratory tests and structures.

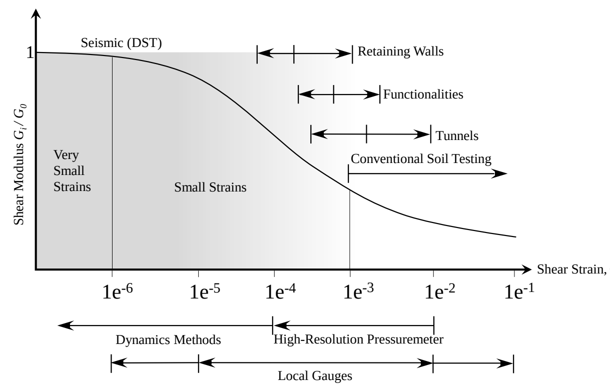

Figure 2. Non-linear stiffness-strain behaviour (adapted from Atkinson & Sallfors, 1991)

This paper presents the analysis of Cambridge PM data within the Clearwater Shale at the Aurora North mine located north of Fort McMurray, Alberta. The analysis focuses on the determination of the non-linear shear modulus and compares the results with the London Clay. Based on the interpretation of the DST and PM data, stiffness-strain curves for the Clearwater are developed between strain increments of 10^{-6} and 10^{-2}.

2 BACKGROUND INFORMATION

2.1 Site Information

The Clearwater Formation has a thickness of between 30 and 40 m based on the borehole information. The Clearwater consists of transgressive/ regressive sequences of moderately bioturbated clay, silty clay and glauconitic silt/sand. Figure 3 shows the distribution and typical thicknesses identified in the study area.

2.2 High-Resolution Pressuremeter Testing

The pressuremeter instruments used were designed and manufactured by Cambridge Insitu Ltd. The Self-Boring Pressuremeter Testing (SBPM) has an outside diameter of 88 mm and has a test interval length of approximately 0.5 m. The Reaming Pressuremeter (RPM) is a rigid cylindrical probe a little less than 1 m long and 47 mm in diameter. Using these dimensions, the volume of soil sheared during a given test is approximately 3 orders of magnitude more than a conventional 38 mm diameter triaxial sample.

The pressuremeter test produces an expansion and contraction curve assuming a right cylindrical cavity. The Cambridge probe measures the radial displacement of the probe at the borehole wall for a given applied radial stress. Within clay soils, the PM test is performed assuming undrained conditions, therefore the shear strain is derived from circumferential strain. In this research it is assumed that the cavity expands as a right cylinder, therefore, the average radial (cavity) strain of the 6 probe arms has been used for analysis and stiffness anisotropy has not been considered.

A total of 13 pressuremeter tests were performed in 3 boreholes as shown in Table 1.

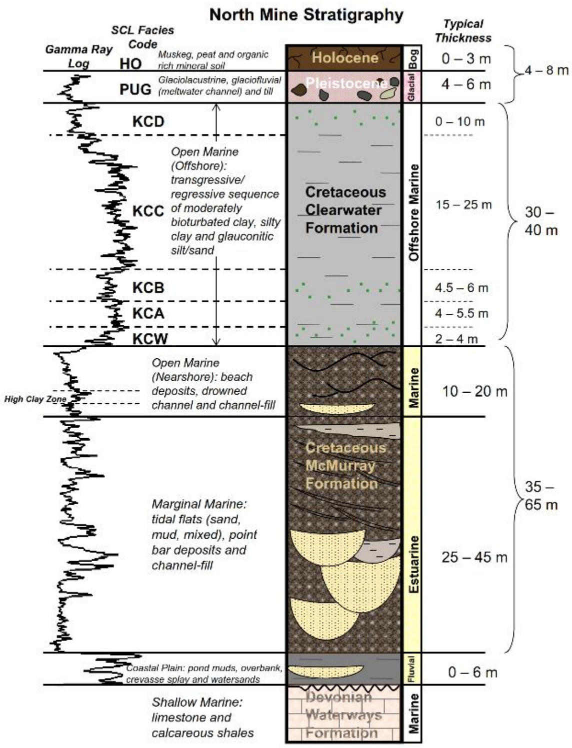

Figure 3. North Mine Stratigraphy (from Syncrude, 2022)

Table 1. Characteristics of tests performed.

Borehole	Test	# U-R Cycles	Material	Elevation [mASL]	Instrument
22-7030	Test 1	3	KCB	263.90	SBPM
	Test 2	3	KCB	262.13	SBPM
	Test 3	3	KCA	258.90	SBPM
	Test 4	3	KCA	256.94	SBPM
21-7002	Test 1	3	KCC	264.90	SBPM
	Test 2	4	KCC	260.60	RPM
	Test 3	3	KCB	256.40	RPM
	Test 5	4	KCA	248.50	RPM
	Test 6	3	KCA	247.60	SBPM
21-7004	Test 1	3	KCC	248.50	RPM
	Test 2	4	KCC	279.50	RPM
	Test 3	4	KCB	274.30	RPM
	Test 4	4	KCA	268.30	RPM

3 STIFFNESS ANALYSIS

Stiffness is a property that depends mainly on the soil mineralogy, structure, the degree of cementation, fissuring or discontinuities. It also depends on the stress path, strain path and strain rate as well as and the chosen strain increment (Sorensen, 2007). The stiffness at small deformations is typically assumed to be linear and is directly measured through seismic tests and generally assumed to be consistent with a strain increment of 10^{-6}. Figure 2 shows that at strain increments greater than the 10^{-6} value, the soil's stiffness becomes non-linear, decreasing with increased strain up to a residual (critical state) value.

3.1 Shear Modulus

Because the boundary conditions of the PM are well known and the soil is subjected to pure shear during borehole expansion, the soil shear modulus can be determined with minimal uncertainty. The simplest interpretation of shear modulus is the determination of the slope of the bisector extending through the top and bottom of each unloading-reloading cycle (G_{ur}). If the bisector is drawn through the radial stress-cavity strain field curve, the slope must be divided by 2 to result in a shear modulus. Because there are an infinite number of moduli that can be calculated from any given unload-reload cycle, the secant modulus through the top and bottom of a given cycle is the lowest shear modulus for that test and the highest strain increment. The initial shear modulus (G_0) is not typically calculated from the PM as it tends to overestimate the true value.

For this paper, the analysis of the minimum secant and non-linear shear moduli were calculated from 13 PM tests. At each of the test intervals, between 3 and 4 unload-reload cycles were performed during either the loading or unloading phase of each test. For each cycle, the non-linear secant shear modulus was also calculated using the method described by Bolton and Whittle (1999).

The tests within the Clearwater indicated strong similarity to those documented in the London Clay (Muir Wood, 1990; Whittle, 1999). The comparison to the London Clay could prove valuable for design in the Clearwater given the wealth of historical information on the strength and stiffness profiles. To assess whether there is a relationship between the London Clay and the Clearwater, a comparative analysis has been conducted with historical data obtained in triaxial and pressuremeter tests.

Figure 4 shows the shear modulus calculation for unload-reload cycle #3 of Test 4 in borehole 22-7030 and Table 2 shows in summary the minimum secant shear modulus values determined in each of the loops performed on the Clearwater Formation.

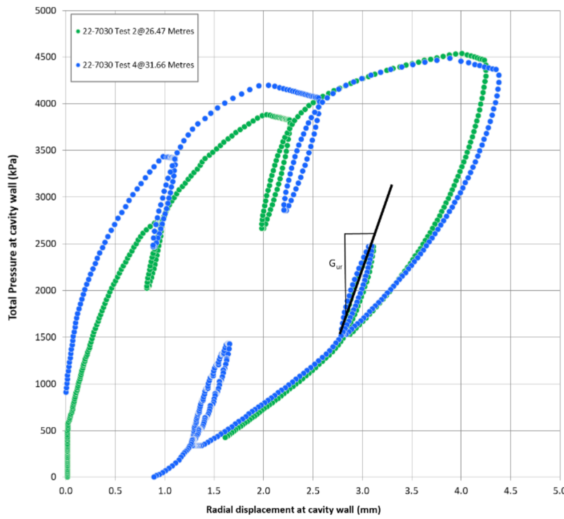

Figure 4. Determination of pressuremeter shear modulus unload/reload (G_{ur}) by linear regression (Tests 2 and 4 Borehole 22-7030).

Table 2. Shear moduli determined from unload-reload cycles.

Borehole	Test	G_{ur} [MPa]			
		U-R 1	U-R 2	U-R 3	U-R 4
22-7030	Test 1	28.6	25.7	36.5	-
	Test 2	104.3	87.4	74.6	-
	Test 3	75.0	69.5	86.8	-
	Test 4	93.8	73.0	72.0	-
21-7002	Test 1	74.0	68.0	53.0	-
	Test 2	28.0	27.0	26.0	39.0
	Test 3	23.0	23.0	28.0	-
	Test 5	32.0	31.0	35.0	32.0
	Test 6	90.0	89.0	97.0	-
21-7004	Test 1	80.0	70.0	67.0	-
	Test 2	47.0	52.0	92.0	60.0
	Test 3	112.0	76.0	64.0	82.0
	Test 4	81.0	77.0	74.0	83.0

3.2 Non-linear stiffness

As discussed above, the stiffness of a soil is not linear. In most cases however, most constitutive models assume that the modulus is represented linearly to simplify calculate deformations. The non-linear behaviour of the shear stress - shear strain for HOC has been extensively studied by Jardine et al. (1984, 1986), Burland (1989) and Hight et al. (2007). The variation of shear modulus (G_i) with increasing shear strain can be evaluated discretely (Muir Wood, 1990) from the small strain version of Palmer (1972) or fitted by the power law of Bolton and Whittle (1999) using equation 1.

$$G_s = \alpha\gamma^{\beta-1} \qquad [1]$$

Where α and β are material parameters determined from only the reloading portion of the unload-reload cycle in an PM test. Only the reload portion is selected, given the difficulty in determining an appropriate strain origin for analysis. Jardine (1992) discuss the impact of rate effects on strain origin determination from the unloading phase. Despite the pressure typically being held prior to unloading, the actual point where the borehole stops expanding from either creep or consolidation is not clear. Often, the 'start' of unloading is assumed where the reloading portion crosses the unloading branch. Though not necessarily incorrect, the accuracy of this assumption negates the evaluation of the small strain (10^{-4}) moduli. This level is the smallest value that is reasonably measured using the technology installed in a Cambridge style probe. It is also important to note that Jardine (1992) suggests that the cut-off strain of 10^{-4} is sufficient for assuming linear elasticity.

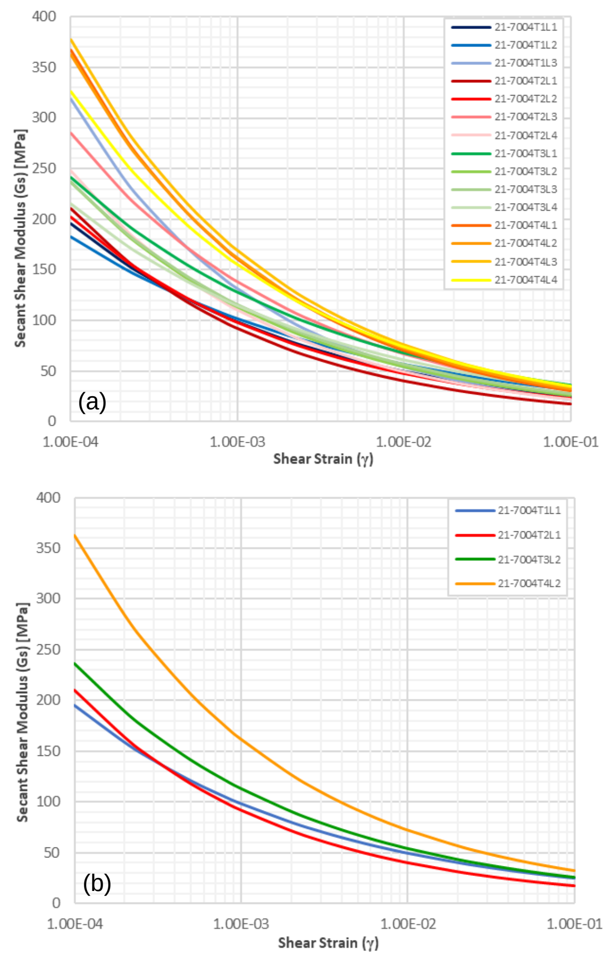

Figure 5. Secant shear modulus in Borehole 21-7004. a) each U-R cycle b) Simplified trend for each test.

Figure 5 shows the theoretical behaviour of the secant shear modulus for small deformations. These curves were created from the measurements of the α and β variables of each reloading loop shown in Figure 5a. To check the relative accuracy of the analysis, it would be expected that if the Clearwater was behaving as an undrained material, then the modulus degradation curves from a given test interval should plot on top of one another. The plots should be similar because if the soil is truly undrained, there is no consolidation that is possible (zero volume change) and therefore the effective stress at the borehole wall following yielding remains constant regardless of the stress state.

A representative reload cycle of each test interval shown in Figure 5a was chosen for further analysis as shown in Figure 5b. At first glance of the data in Figure 5b, it would appear that the material is quite different in each test as the shear modulus appears to increase for each test, however, if the calculated shear moduli are normalized to the vertical effective stress, the degradation curves are quite similar suggesting that the Clearwater's characteristics are consistent with depth and conducive to normalization as shown in Figure 6. Table 3 shows the selected non-linear modulus parameters for Clearwater Formation.

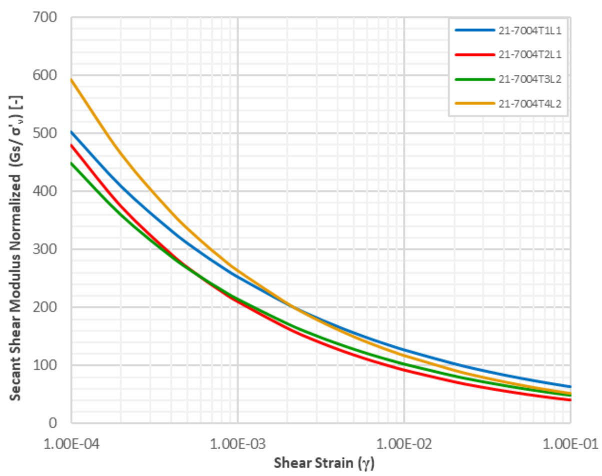

Figure 6. Secant shear modulus normalized in Borehole 21-7004

The secant shear moduli results obtained from the Clearwater Formation were compared to data provided by Jardine (1992) for the London Clay with an OCR of 3.0 and to the London Clay analyzed by Whittle & Liu (2013). Figure 7 shows the nonlinearity of the shear modulus for shear strains between 10^{-4} and 10^{-1}. From the figure it is observed that the Jardine (1992) clay presents a higher effective stress therefore the initial modulus is almost twice that of the Clearwater shale. In the case of Whittle & Liu (2013) clay, the modulus degradation curve is nearly identical to the Clearwater clay.

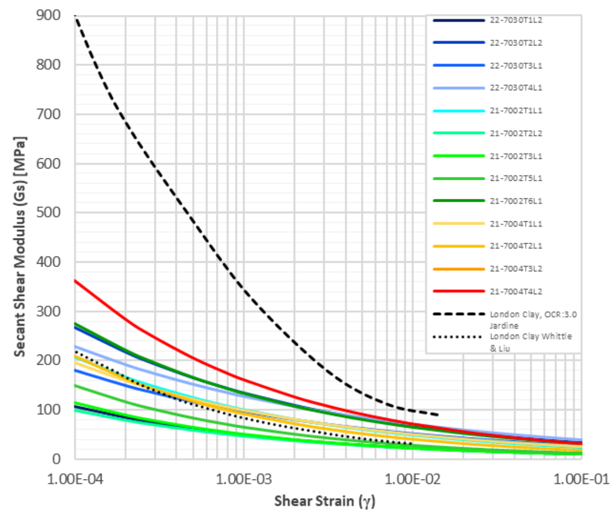

Figure 7. Comparison of secant shear modulus for Clearwater shale and London Clay.

In order not to bias the behaviour by the initial stress state, the secant shear modulus was normalized to the $G_{0.01\%}$ modulus as recommended by Jardine (1992). This strain was defined by Jardine as the cut-off between linear

Table 3. Selected non-linear modulus parameters.

Borehole	Test	α	β	$G_{s0.01\%}$	$G_{s0.1\%}$	$G_{s1\%}$	$E_{s0.01\%}$	$E_{s0.1\%}$	$E_{s1\%}$	G_y	G_{50}	G_{max}
		[MPa]	[-]	[MPa]	[MPa]	[MPa]	[MPa]	[MPa]	[MPa]	[MPa]	[MPa]	[MPa]
22-7030	Test 1	5.107	0.677	100	48	23	218	104	49	22	30	97
	Test 2	17.350	0.703	267	135	68	591	298	150	53	81	219
	Test 3	14.197	0.715	196	102	53	436	226	117	70	89	283
	Test 4	13.446	0.675	268	127	60	583	276	131	52	71	230
21-7002	Test 1	11.348	0.684	208	101	49	456	220	106	53	74	231
	Test 2	5.111	0.686	92	45	22	202	98	47	21	28	89
	Test 3	4.304	0.644	114	50	22	244	108	47	20	30	97
	Test 5	5.046	0.627	157	66	28	332	141	60	23	35	114
	Test 6	15.218	0.686	274	133	65	600	291	141	76	104	326
21-7004	Test 1	12.688	0.703	196	99	50	432	218	110	52	70	216
	Test 2	7.779	0.643	210	92	40	449	197	86	32	46	150
	Test 3	19.025	0.724	242	128	68	540	286	152	59	77	234
	Test 4	13.566	0.642	367	161	71	783	344	151	55	81	261

elasticity and non-linear elasticity. More importantly, as described above, this strain represents the smallest strain that can be reasonably measured with current pressuremeter equipment. Non-linear shear moduli normalized to the $G_{0.01\%}$ for the Clearwater and London Clays are shown in Figure 8. Once the stiffness degradation data is normalized, it is clear that the data from Jardine (1992) and Whittle & Liu (2013) is slightly lower.

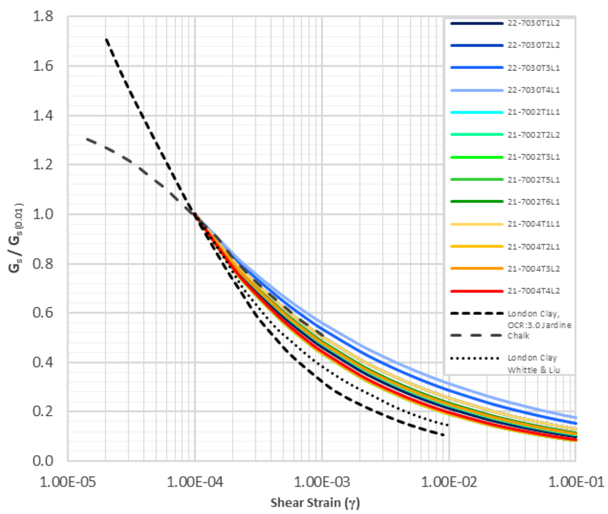

Figure 8. Comparison of normalized secant shear moduli for Clearwater shale and London Clay.

3.3 Small-strain Shear Modulus

The shear modulus for small strains was calculated from the shear wave velocity (V_s) obtained in the Downhole Seismic Test (DST). The small strain shear modulus may then be calculated based on the material density, ρ and the measured shear wave velocity at the test interval using equation 2.

$$G_0 = \rho V_s^2 \qquad [2]$$

A plot of the shear modulus at small deformations along the borehole shows the difference in stiffness between the Clearwater formation and the McMurray formation. In general, the modulus for small deformations of the clayshale tends to be homogeneous with mean values between 186 and 236 MPa. The minimum standard deviation was measured in Borehole 21-7004 at 20 MPa.

Figure 9 shows the shear modulus behaviour with depth in Borehole 21-7004, including the PM G_{ur} modulus and G_s modulus values for an equivalent strain of 0.0001. It is expected that the G_s values calculated from the unload-reload cycles will be less than those calculated by the DST given the change in strain increment. The G_0 obtained from the DST is considered truly linear elastic and therefore must be higher than values at higher strain increments. Table 4 presents a summary of the shear moduli determined from the DST for the 3 boreholes analyzed.

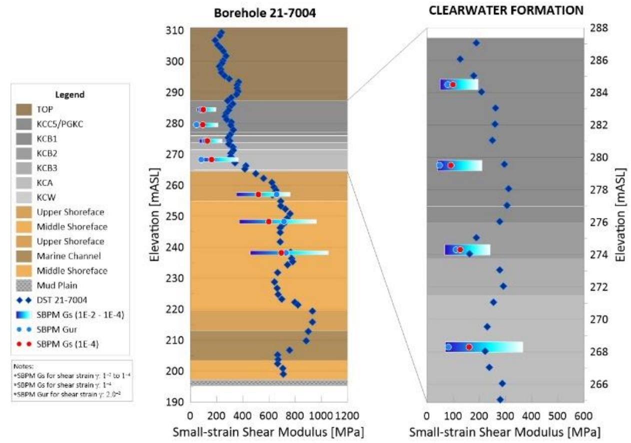

Figure 9. Small-strain Shear modulus from Downhole Seismic Testing (21-7004).

Table 4. Modulus from Shear Wave Velocity (Vs)

Borehole		Elevation	Yield Shear Modulus	G_0
		[mASL]	[MPa]	[MPa]
21-7002	Test 1	264.9	53	124.07
	Test 2	260.6	21	140.92
	Test 3	256.4	20	128.91
	Test 5	248.5	23	281.57
	Test 6	247.6	76	288.19
21-7004	Test 1	284.5	52	167.85
	Test 2	279.5	32	186.96
	Test 3	274.3	59	172.89
	Test 4	268.3	55	213.62
22-7030	Test 1	263.9	35	152.36
	Test 2	262.1	27	112.45
	Test 3	258.9	48	235.77
	Test 4	256.9	89	444.52

3.4 Shear modulus degradation

Having the initial shear modulus (G_0) and the secant shear modulus determined from the PM, a graphical coupling of the moduli was performed using a sigmoidal curve model. The purpose of this was to complete the stiffness degradation curve and provide a link between the in-situ test data and inputs required for non-linear elastic constitutive models like the hardening soil or small strain hardening soil model. The key unknown using the sigmoidal model, is the performance of the soil in strains between 10^{-6} (seismic) and 10^{-4} (PM). Figure 10 shows the field data from borehole 21-7004, fitted using a sigmoidal model. Where the transition in the curve from linear elasticity to non-linear behaviour is not clear and has been assumed. At this time, it is believed that the method described by Smith et al. (2018) will fill in the missing data and complete the small strain non-linear modulus profile.

Data has been acquired during this research program and is currently being processed.

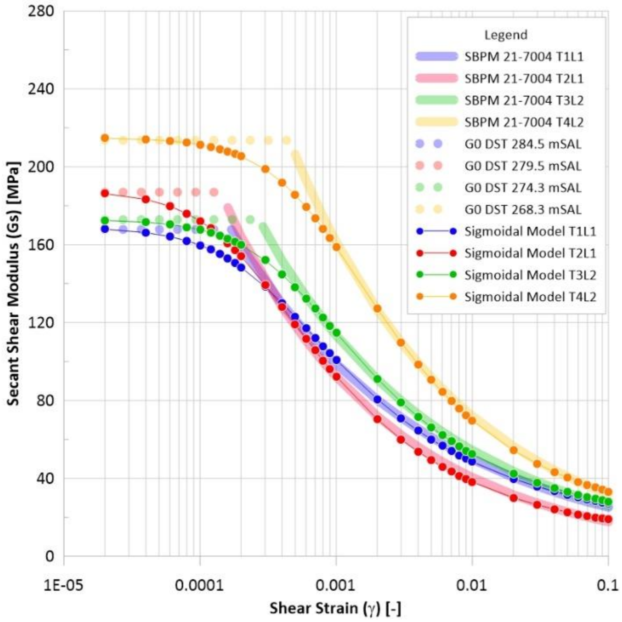

Figure 10. Shear modulus degradation borehole (21-7004).

4 SUMMARY AND CONCLUSION

As Jardine (1992) indicates "the behaviour over the nonlinear strain range 0.001%<ε<1% is crucially important in controlling the ground's response in a wide range of civil engineering problems".

To evaluate the stiffness and non-linear behaviour of the Clearwater Formation Clayshale of the Aurora North Mine north of Fort McMurray, 13 PM high-resolution pressuremeter tests were conducted. The nonlinear shear modulus behaviour for small strains was determined by Bolton and Whittle (1999) power law, a comparison was made with the degradation of the London Clay, and the initial shear modulus was determined from the shear wave velocities of the DST test.

The shear modulus and elastic modulus ranges of Clearwater Clayshale and its behavior for different strain levels were defined.

The graphical representation of modulus degradation generated using a conventional sigmoidal model.

It is important to complement this study at a more detailed level, evaluating the anisotropic behavior of the borehole stiffness and the identification of weak contacts.

5 ACKNOWLEDGMENTS

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