Geotechnical Properties of a Weak Mudstone in Downtown Calgary

Kwan Yee Lo & Silvana Micic
Geotechnical Research Centre, University of Western Ontario, London, Ontario
Thomas Lardner & Matthew Janes
Isherwood Associates, Mississauga, Ontario

ABSTRACT
A comprehensive laboratory research program was carried out to investigate strength and deformation properties, both elastic and time-dependent, of weak mudstone from the Porcupine Hills Formation. The mudstone samples were recovered from the deep excavation for The Bow office tower located in the heavily built-up area of downtown Calgary. The fractured and highly fissile nature of weak mudstone renders block sampling and preparation of specimens unusually difficult. The strength test results indicate that the horizontal uniaxial compression strength of the mudstone is around 830 kPa and the elastic modulus is from 85 to 180 MPa. The results of multistage shear tests suggest that the mudstone effective peak strength parameters are c'=340 kPa and $\phi'=24^o$, while the residual strength parameters are c'_r=0 kPa and $\phi'_r=15^o$. A limited number of swell tests performed indicates that the weak mudstone possesses significant swelling behaviour that is consistent with post-excavation displacements recorded by inclinometers. The program also included a study of mineralogical, physical and chemical properties of the mudstone to delineate the mechanism(s) of swelling.

RÉSUMÉ
Un programme exhaustif de recherche en laboratoire a été mené dans le but d'étudier la compressibilité et les propriétés de déformation, autant élastiques que variables dans le temps, de la roche argileuse de la Formation Porcupine Hills. Les échantillons de roche argileuse ont été extraits de l'excavation profonde de la tour à bureau Bow, dans un secteur densément construit du centre-ville de Calgary. La nature fracturée et hautement fissurable de la roche argileuse rend l'échantillonnage en bloc et la préparation de spécimens particulièrement difficiles. Les résultats d'essais de compressibilité démontrent que la résistance en compression uniaxiale horizontale de la roche est d'environ 830 kPa et que le module d'élasticité varie entre 85 et 180 MPa. Les résultats des essais de cisaillement à plusieurs niveaux suggèrent que les paramètres de résistance effective maximale sont c'_r=340 kPa et $\phi'_r=24^o$, alors que les paramètres de résistance résiduelle sont c'_r=0 kPa et $\phi'_r=15^o$. Un nombre restreint d'essais de gonflement ont démontré que la roche argileuse possède une tendance significative au gonflement, cela allant de pair avec les déplacements post-excavation observés par les inclinomètres. Le programme comptait aussi une étude des propriétés minéralogique, physique et chimique de la roche argileuse afin de déterminer le mécanisme de gonflement.

1 INTRODUCTION

In 2007, the construction of The Bow tower was commenced in downtown Calgary. The 54-storey tower will be the headquarter office building of EnCana Corporation. An image showing The Bow under construction, surrounded by adjacent buildings, is shown in Figure 1. The site, located about 400 m south of the Bow River, encloses a two city block area of approximately 17,000 m^2. The 6th Avenue SE splits the site into two sections, including The Bow tower on the north and a proposed cultural centre on the south. The Bow borders two heritage buildings, the Royal Canadian Legion No.1, which is a provincial historic resource, and the facade of the York Hotel. The depth of excavation is approximately 20.5 m for the construction of the six storeys of underground parking. In a such heavily built-up area, the horizontal movements during this deep excavation may affect the adjacent structures and buried utilities.

The site stratigraphy is composed of 6 to 7 m thick unconsolidated deposits underlain by bedrock of the Porcupine Hills Formation, which is mainly layered weak mudstones, siltstones and sandstones. The mudstone is highly fissile and susceptible to weathering. In spite of experience gained in several previous deep excavations in downtown Calgary, the mechanical properties of the bedrock are still poorly understood It has been reported that the Porcupine Hills Formation exhibits significant lateral in-situ stresses (K_o=2) as well as susceptibility to shear band propagation in weak mudstone layers (AMEC, 2006 and Lardner et al., 2008). However, uncertainties still remain in predicting rock deformation during excavation, in particular, (a) the magnitudes and directions of the maximum and minimum horizontal stresses in rock, (b) the existence and location of shear bands, and (c) the strength and deformation properties of the weak mudstone.

To support the deep excavation for The Bow, a comprehensive shoring system was designed and constructed. It consisted of a continuous caisson wall installed 2 m into the sound bedrock and an anchored shotcrete wall in the excavated bedrock below (Lardner et al., 2008). The excavation support construction was incorporated with an extensive monitoring program which included 12 inclinometers, 8 extensometers, and precision target monitoring of the shoring wall and surrounding buildings. During excavation, the monitoring program proved to be successful in providing early warning of horizontal movements of the soil and rock mass. It also showed that the shear band phenomenon is an important factor for large horizontal deformations in deep, downtown Calgary excavations (Lardner et al., 2008).

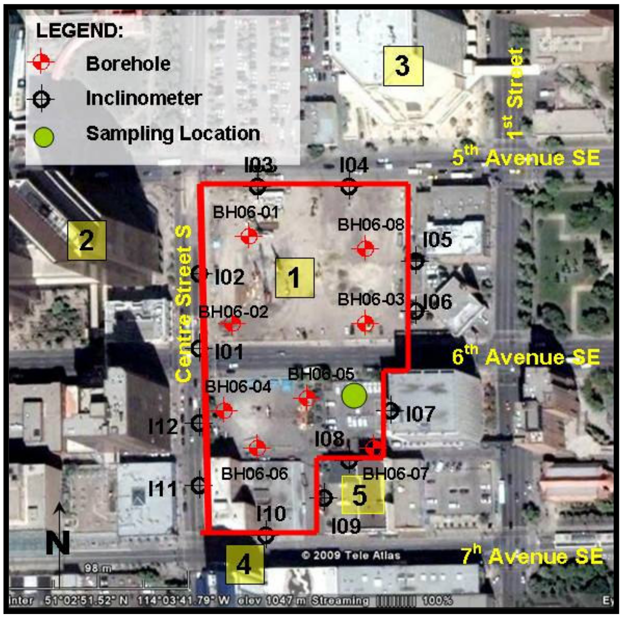

Figure 1. The Bow site: (1) The Bow; (2) Petro-Canada Centre; (3) Telus, (4) Calgary's Light Rail Transit and (5) The Royal Canadian Legion No.1

Field observations during construction of The Bow indicated the necessity to characterize the geotechnical properties of the mudstone which would allow a better assessment of the horizontal displacements induced by rock excavations. The objective of this study is to investigate the strength and deformation properties, both elastic and time-dependent, of the weak mudstone through a comprehensive laboratory test program including uniaxial compression tests, hydrostatic compression test, multistage direct shear tests, and swell tests. A study of mineralogical, physical and chemical properties of rock samples was also performed. The strength and deformation properties of the mudstone are presented. The swelling behaviour of the rock measured is confirmed with field measurements.

2 SITE GEOLOGY AND SUBSURFACE CONDITIONS

Geotechnical investigations at The Bow site were performed in 2005 and 2006 by AMEC Earth and Environmental from Calgary. Their supplementary geotechnical field investigation performed in 2006 consisted of eight boreholes extending to depths of 19 to 21 m below ground surface (AMEC, 2006). Bedrock, that was encountered at a depth of about 7 m, was continuously drilled using a NQ size tube barrel. The locations of the boreholes are shown on Figure 1.

The investigations show that the overburden condition consists of 0 to 0.5 m of fill and 5.5 to 7 m of well graded sandy gravel with variable inclusions of cobbles and boulders. The overburden is underlain by bedrock that was encountered between elevations 1039 and 1041 m. The bedrock consists of a horizontally bedded sequence of grey mudstone, siltstone, and sandstone layers known geologically as the Porcupine Hills Formation of Cretaceous or Paleocene age (Jackson and Wilson, 1987 and Osborn and Rejewicz, 1998). The formation was formed as lacustrine deposits with clay as a cementing agent of the mudstones and siltstones giving these formations a more soil-like quality than the sandstone lenses. According to rock descriptions in the borehole logs, the bedrock is weathered, weak and fractured to a depth of about 2 m. Rock Quality Designation (RQD) ranged from 0 to 95% with an average value of 45%, which classifies these rock cores as poor quality rock. The mudstone is prone to rapid weathering upon excavation.

The ground surface elevation of The Bow site lies between 1045.5 to 1046.3 m. The groundwater level is located in the gravel soils and varies seasonally from 1 m to 3.5 m in depth above the rock surface.

3 LABORATORY TESTS FOR THE EVALUATION OF MUDSTONE PROPERTIES

3.1 Test Program

A comprehensive laboratory test program was carried out to study strength-deformation, swelling, mineralogical, physical and chemical properties of the Porcupine Hills Formation mudstone samples recovered from The Bow excavation. The test program consisted of (a) uniaxial compression tests; (b) multistage direct shear tests; (c) hydrostatic compression test; (d) semi-confined swell tests; (e) null-swell tests; (f) XRD tests; (g) glycol retention tests; (h) water content tests; (i) Atterberg limits tests; (j) unit weight tests; (k) calcite content tests; and (l) rock salinity tests. All uniaxial compression and swell tests were performed on samples with loading parallel to the bedding planes.

3.2 Recovery of Rock Samples and Test Specimen Preparation

The rock samples were collected just south of 6th Avenue SE, on the east side of the site at the approximate elevation of 1028 m, as shown on Figure 1. Sampling preparation was done by identifying the location of the required rock layer on an exposed face and then excavating. Sampling was extremely difficult due to the highly fissile nature of the rock. Several attempts to recover large samples were unsuccessful. Any use of powered machinery to aid in the removal of samples resulted in the rock disintegrating. All successful samples were removed by hand. Each irregular shaped block sample was immediately wrapped in plastic cling-wrap, and waxed to prevent any loss of moisture. However, the rock samples received in the lab were weak and highly fissured, and were prone to rapid deterioration upon unpacking from the waxed wrap.

To obtain proper samples in shape and length for the proposed testing program was extremely difficult. Any coring, cutting, trimming or polishing of mudstone samples was exceptionally challenging due to the rock disintegrating. For example, coring of specimens from the block samples to obtain a circular-section specimen for uniaxial, hydrostatic, and swell tests was practically impossible. Therefore, square-section specimens were cut carefully using a band saw without any lubricant, as shown on Figure 2 (a). However, many samples were broken during cutting action which significantly limited the number of appropriate samples for testing.

(b)

(a) (c)

Figure 2. Sample preparation: (a) cutting of samples; (b) a typical sample tested in compression tests; and (c) a typical sample tested in direct shear tests

4 RESULTS OF LABORATORY TESTS

4.1 Uniaxial Compression Tests

Five uniaxial compression tests were carried out on square-section mudstone specimens with loading parallel to the bedding planes. Figure 2 (b) shows a typical sample tested in uniaxial compression tests. Table 1 summarizes the results of the tests. The stress-strain relationships of uniaxial compression tests are illustrated on Figure 3. It can be seen that the uniaxial compressive strength of the mudstone lies in the range of 560-1230 kPa. Variation in strength could be a result of variability in sample size and slenderness ratio due to the difficulty in obtaining proper samples. It has been reported that the compressive strength of rock decreases as size and slenderness of the sample increase, especially in fissured rocks (Hudson and Harrison,1997). The larger the specimen, the greater the likelihood of having larger and more fissures, and hence, the greater the likelihood of lower strength.

The initial tangent modulus E_t and modulus E_{50}, which is the conventional secant modulus taken at up to 50% of the failure stress, are determined from the stress-strain curves, and the results are listed in Table 1. The values of both elastic moduli are very similar. The initial elastic modulus of the mudstone, E_t, varies from 85 to 180 MPa, while the modulus E_{50} varies from 80 to 170 MPa.

Table 1. Results of the uniaxial compressive tests

Specimen #	Dimensions L x W x H (mm)	Uniaxial Compressive Strength (kPa)	Modulus* (MPa)	
			E_t	E_{50}
UCT 1	43 x 44 x 76	720	138	122
UCT 2	37 x 45 x 69	1070	180	170
UCT 3	50 x 50 x 71	570	85	80
UCT 4	42 x 39 x 106	560	135	132
UCT 5	38 x 33 x 67	1230	168	160

* E_t= initial tangent modulus
E_{50} = secent modulus at 50% σ_{max}

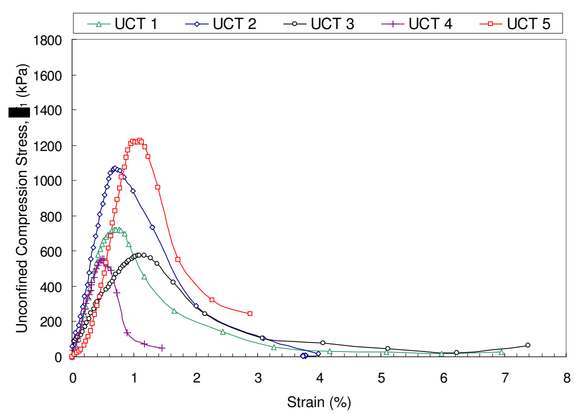

Figure 3. Results of the unconfined compression tests

4.2 Multistage Direct Shear Tests

Multistage direct shear tests were performed on five mudstone samples. Figure 2(c) shows a typical sample tested. The tests performed were consolidated drained tests at different consolidation pressures. The limited availability of appropriate test samples required that the same sample be used for different applied normal stresses. Samples were consolidated and sheared in stages by gradually increasing the load until the maximum horizontal movement of the box was reached. Each sample was first consolidated for approximately 24 hours to a predetermined vertical stress, and then sheared until a peak was reached. After the first stage, the sample was consolidated and sheared again under the new, higher normal stress. After two or three stages of consolidation and shearing, the sample was brought back to its initial preshear position by reversing the direction of movement manually until the two halves of the shear box matched. The sample was then sheared again to assess the residual strength of the mudstone. The constant rate of horizontal movement was set at 0.0162 mm/min.

The typical results are shown on Figure 4. As can be seen from Figure 4, three stages were performed on this particular sample to measure peak shear strengths. The sample first consolidated to 150 kPa, and then sheared until the failure. After failure the machine was stopped and the sample was loaded to the second stage of 300 kPa and allowed to consolidate over 24 hours. The sample was then sheared again until the peak strength was reached. The shearing stopped again and the normal stress of 600 kPa was applied for consolidation and shearing. The residual strengths were then measured on the same specimen.

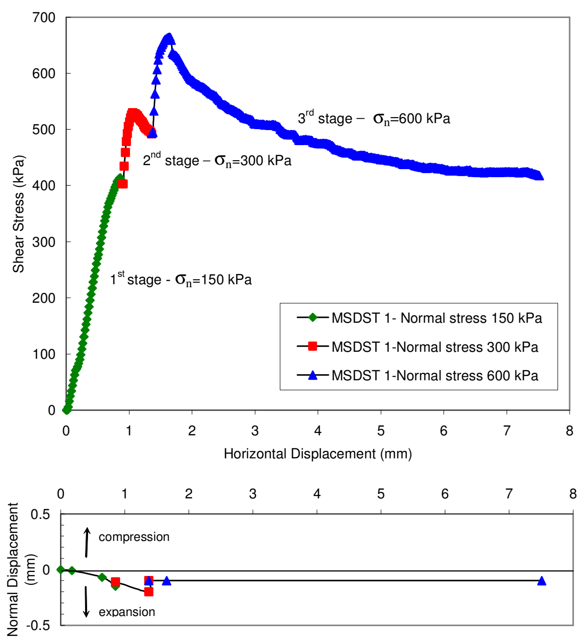

Figure 4. Results of the multistage direct shear test MSDST 1—Peak shear tests

The summary of all results of five series of tests, including peak and residual shear strengths, are shown on Figure 5. The figure shows the range of peak strength results. The scattering of the results could be due to inevitable sample disturbance caused by preparing specimens from material in various fissured and fractured conditions. In general, the results of peak shear strength may be represented by a linear Mohr-Coulomb envelope giving c'=340 kPa and ϕ'=24°, while the results of residual shear strength may be represented by linear Mohr-Coulomb envelope giving c'_r=0 kPa and ϕ'_r=15°.

The results of four direct shear tests performed by Golder Associates on samples from the same rock formation are also shown on Figure 5. Both the peak and residual strengths measured are consistent with the results obtained in this test program. In addition, the results from several triaxial compression tests on the same rock are also plotted on Figure 5, indicating that the strength parameters from two types of tests are consistent with each other.

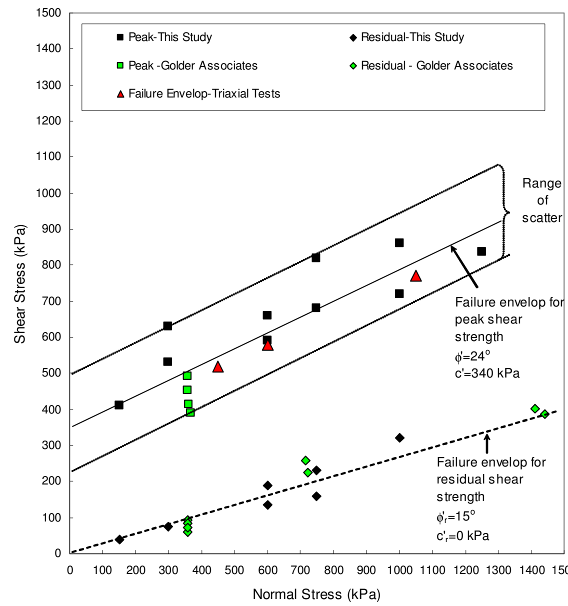

Figure 5. Summary of the results of multistage direct shear tests

4.3 Hydrostatic Compression Test

A hydrostatic compression test was carried out on a specimen of square prism shape trimmed from the block sample of the Porcupine Hills Formation mudstone. The test was performed in a triaxial compression cell gradually applying all round pressure and keeping the deviator stress equal to zero. A constant pressure rate of 30 kPa/hour was applied. Volume changes of the specimen were monitored during the test and recorded.

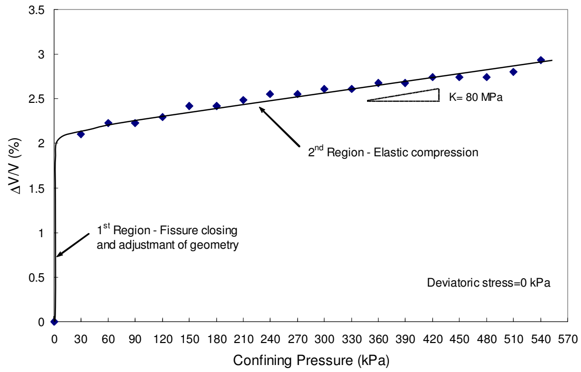

Figure 6. Results of the hydrostatic compression test

Figure 6 illustrates the results of the hydrostatic compression test. As can be seen, the pressure-volumetric curve shows two distinct regions. In the first region, the volume changed due to geometry adjustment and the closing of pre-existing fissures. In the second region, the volume change is representative of bulk compression of rock (Goodman, 1980). The slope in the volumetric strain-pressure curve in the second region yields the bulk modulus, K. For this specimen of the weak mudstone, the value is approximately 80 MPa. Using the elastic relationship between K and E, it can be shown that the values of two measured moduli are consistent with each other in the range of Poisson's ratio of 0.15 to 0.3.

4.4 Swell Tests

Two semi-confined and two null swell tests were performed on cubes trimmed out of mudstone samples. The method of semi-confined swell test used was described in Lo et al. (1978). For this test, the rock specimen was submerged in water and the strain changes in only one direction were monitored. A small constant pressure of 0.01 MPa was applied to the rock sample in the direction of measurement while deformations in perpendicular directions remained unrestricted. Test data were analysed by plotting strain vs. the logarithm (to the base of 10) of elapsed time. The average slope between 10 and 100 days of such a plot gives an indication of tendency of the rock to expand upon stress relief and serves as an index termed the "swelling potential" of the rock being tested. Null swell tests were included in the testing program to measure the critical pressure required to completely suppress swelling in the horizontal direction. The procedure and method of interpretation for the null swell tests have been discussed in Lo (1989) and Lo and Lee (1990).

Table 2. Results of swell tests

	Semi-Confined Swell Tests		Null Swell Tests	
Specimen #	SCST 1	SCST 1	NST 1	NST 2
Dimensions L x W x H (mm)	34 x 34 x 34	55 x 57 x 57	39 x 39 x 44	55 x 57 x 51
Calcite Content (%)	0.9	1.02	< 0.9%	< 0.9%
Salinity (mg/g of rock)	0.95	1.2	1.06	0.95
Applied/Seating Pressure (MPa)	0.01	0.01	0.2	0.2
Horizontal Swelling Potential (%/log cycle)	0.24	0.53	n/a	n/a
Horizontal Swell Suppression Pressure (MPa)	n/a	n/a	1.25	0.7

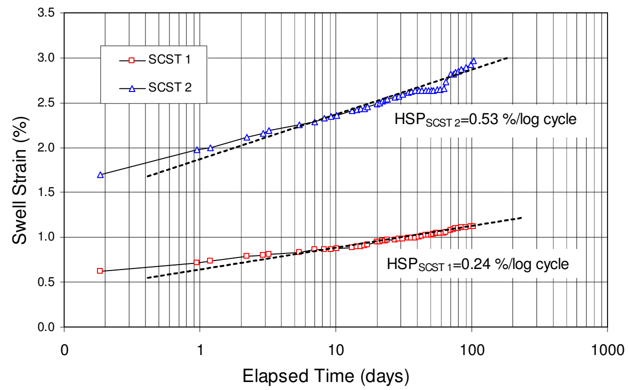

Figure 7. Results of the semi-confined swell tests

The results of semi-confined swell tests are summarized in Table 2, while Figure 7 shows the swelling strain-time curves for the two samples tested. The results show that the Porcupine Hills Formation mudstone exhibits swelling behaviour in horizontal direction. The horizontal swelling potentials (HSP) of the specimens tested were 0.24 and 0.53 %/log cycle under an applied pressure of 0.01 MPa. The calcite contents of the mudstone specimens measured were 0.9% and 1.02%.

The summary of results of the null swell tests is presented in Table 2. Figure 8 shows the stress-time relationships of the two specimens. The results of the two tests yielded suppression pressures of 0.7 and 1.25 MPa in the horizontal direction. The calcite contents of these two samples were less than 0.9%.

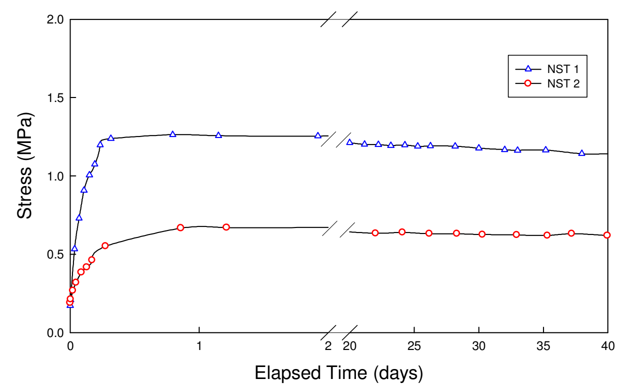

Figure 8. Results of the null swell tests

4.5 X-Ray Diffraction Tests

In order to explore the swelling mechanism of the mudstone, the mineralogy was analyzed by X-ray diffraction (XRD) using a Rigaku RTP 300 RC rotating anode diffractometer with CoKα radiation. The diffraction patterns of random and preferred oriented particles were recorded, and minerals were identified as described by Brendley (1951) and Moore and Reynolds (1997).

The results of the X-ray diffraction analysis on randomly oriented powder pattern of the mudstone, using converted CuKα radiation for clay mineral identification, is shown on Figure 9. As can be seen, the dominant non-clay minerals are quartz and feldspar. The X-ray diffractograms of the oriented less than 2 µm fines of the mudstone shown on Figure 10 are interpreted to show that the primary clay minerals in the mudstone are kaolinite and mixed-layered illite/smectite. The presence of kaolinite is confirmed by disappearance of the 7.2 Å peak (see top scale) after the heat treatment at 550^{o}C. Even though X-ray diffraction patterns of mixed-layered (or interstratified) clay minerals pose some of the most difficult problem of interpretation, the low-angle shift of illite 10 Å peak in Figure 10 in different treatments could suggest interstratification of the illite layers typical of illite/smectite (Moore and Reynolds, 1997). The peak shift toward 16.8 Å in the glycolated sample also suggests the presence of smectite in the interlayers. Smectite is known as a swelling clay mineral.

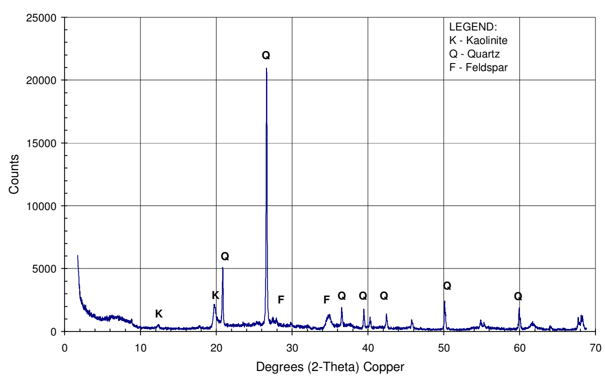

Figure 9. X-ray diffraction traces of the Porcupine Hills Formation mudstone

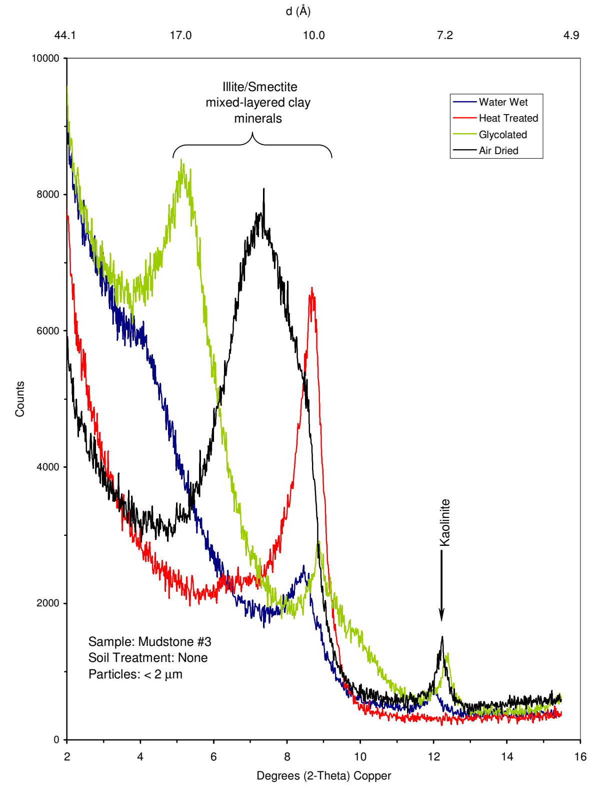

Figure 10. X-ray diffraction traces of oriented < 2µm fines of the Porcupine Hills Formation mudstone

4.6 Glycol Retention Tests

Five glycol retention tests following the method described by Martin (1955) were performed to verify potential presence of swelling clay minerals in the Porcupine Hills Formation mudstone. The surface area of the mudstone clay fraction was estimated using this method. The results of the glycol retention tests on five samples are given in Table 3. As can be seen from the table, the surface area of the mudstone samples ranges between 200.6 and 241.2 m^2/g of rock. This surface area is higher than those typically measured in clay with kaolinite (10 to 20 m^2/g) or illite (70 to 150 m^2/g) as a dominant clay mineral (Mohamed et al., 1998). The higher surface area could be attributed to interstratification of illite with smectite, that is consistent with the findings of the X-ray diffraction analyses.

Table 3. Results of glycol retention, index, calcite content and salinity tests

Test Performed	Samples*				
Glycol Retention (mg/g of rock)	61.32	74.46	72.09	77.35	70.17
Surface Area (m^2/g of rock)	200.6	241.22	220.54	241.05	230.54
Water Content (%)	10.6	9.3	12.46	12.1	11.01
Unit Weight (kN/m^3)	21.68	22.12	22.92		
Liquid Limit (%)	62				
Plastic Limit (%)	33.4				
Salinity (mg/g of rock)	0.95	1.2	1.06	0.95	1.24
Salinity** (g/L of pore water)	8.59	10.92	8.48	8.6	11.26
Calcite Content (%)	0.9	1.02	< 0.9%	< 0.9%	0.9

*- Listed tests were performed on the individual samples
* *- Water content of the rock ~ 11%

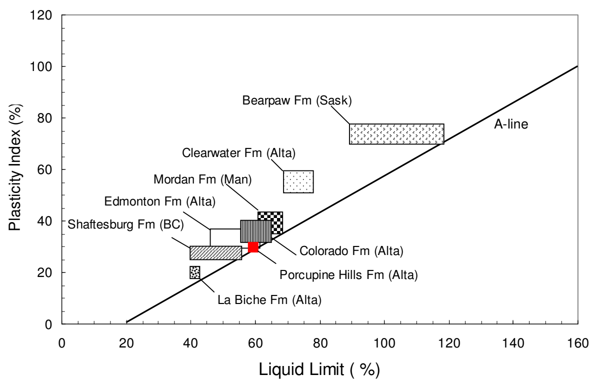

Figure 11. Plasticity of various shale formations in western Canada (modified from Wong, 1998)

4.7 Index Tests

The results of water content, unit weight and Atterberg limits tests are listed in Table 3. The water content tests performed on five mudstone samples measured upon opening the waxed wrap revealed that the water content of the mudstone was between 9.3% to 12.5%. The average unit weight was 22.3 kN/m^3. Using the standard ASTM procedure (ASTM D 4318, 2005) the plastic and liquid limits of the mudstone were determined to be about 33% and 62%, respectively. The results of the Atterberg limits tests on the Porcupine Hills Formation mudstone are plotted on Figure 11, along with the reported results of Atterberg limits of various shales in western Canada (Wong, 1997) for comparison. As can be seen from the figure, the Porcupine Hills Formation mudstone has a liquid limit and plasticity index similar to the shales of the Edmonton and Colorado Formations.

4.8 Calcite Content and Salinity Tests

In general, there is an increasing trend in swelling potential of shales (mudstones) with decreasing calcite content (Lo et al., 1987). The gasometric method using the Chittick apparatus (Dreimanis, 1962) was adopted to estimate the amount of calcite. The results of testing are shown in Table 3. The calcite content of Porcupine Hills Formation mudstone from five measurements is around 0.9%.

The magnitude and rate of swelling also increase with increasing outward salt concentration gradient from the pore fluid of the rock to the ambient fluid. The salinity of rock samples was determined to see if its salinity was higher than salinity of the tap water used in all swell tests. Both processes of osmosis (liquid flows from low to high concentration region) and diffusion (ion transfers from high to low concentration region) can be responsible for the dilution of pore water salt concentration (Lee and Lo, 1993). The results of the tests are shown in Table 3. The salinity of Porcupine Hills Formation mudstone, by weight of rock, was in the range of 0.95-1.24 mg/g of rock. This is equivalent to the salinity of the pore fluid of 8.5-11.7 g/L of pore water, for the water content of the rock of 11%. The salinity of tap water was less than 0.1 g/L.

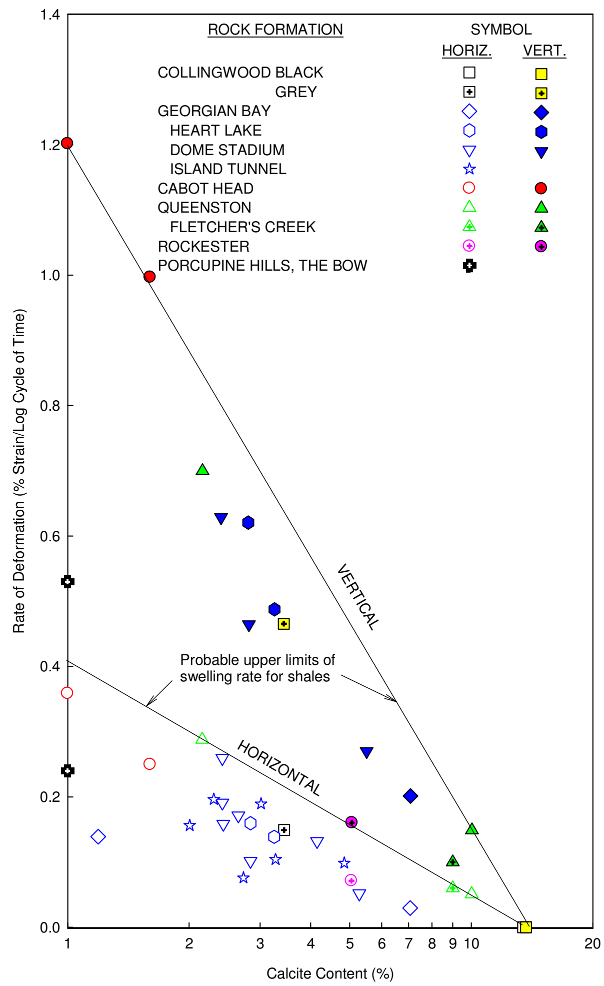

Figure 12. Relationship between swelling potential and calcite content of various shale formations in Canada (modified from Lo et al, 1986)

5 DISCUSSION

The horizontal swelling potentials and calcite contents measured on the Porcupine Hills Formation mudstone were plotted on Figure 12, along with the reported values of these two parameters for various shales in southern Ontario (Lo et al., 1987) for comparison. As can be seen from the figure, the relationship of these two parameters measured on the Porcupine Hills Formation mudstone is consistent with the relationships established for other shales. From the results of mineralogical and chemical tests, it is considered that probable causes of swelling of the Porcupine Hills Formation mudstone include low calcite content, presence of smectite, and salt concentration in the pore water in the rock.

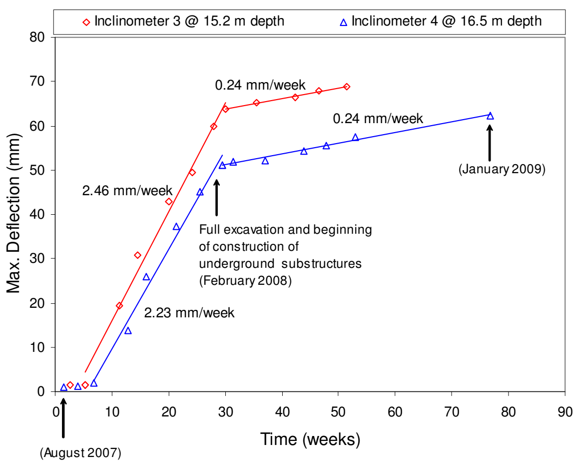

Figure 13. Maximum deflection vs. time recorded in two inclinometers at the north wall of The Bow excavation

The records of inclinometer measurements were examined for field evidence of swelling. The maximum horizontal deflection recorded over almost 80 weeks in two inclinometers (Inclinometer 3 and 4; see Figure 1) installed in the north wall of the excavation are shown on Figure 13. At this location the excavation sequence was relatively straight forward, resulting in relative plane strain conditions. As can be seen, a deflection rate of approximately 2.3 mm/week occurred during excavation. After the excavation was completed at the north wall, the movement continued with the slower rate of 0.24 mm/week, indicating the presence of time-dependent deformation most likely due to swelling of mudstone. A theoretical estimate of post-excavation, horizontal, time-dependent deformation using an average laboratory rate of 0.4 %/log cycle of time, would be approximately 15 mm in a year. This value may be compared with the field record of 12.5 mm in the same period.

6 CONCLUSIONS AND RECOMMENDATIONS

A program of field sampling and laboratory testing of the weak mudstone of Porcupine Hills Formation at The Bow deep excavation in downtown Calgary has been carried out. The main objective of the investigation was to characterize the geotechnical behaviour of the rock, so as to provide relevant information on deformation and strength parameters for detailed analyses of field behaviour at the site, and for similar projects in the future. From the results of this study, the following conclusions may be drawn:

(a) The fractured and highly fissile nature of the weak mudstone renders block sampling and preparation of specimens unusually difficult. Specimens prepared were disturbed to various degrees. Alternative techniques should be considered. An approach could be to use a thin wall, low area ratio Shelby tube, axially and circumferentially split. This special sampler should be pushed step by step into the weak rock, with manual progressive excavation around its cylindrical surface. In the laboratory, specimens of required geometry can be readily retrieved from the sampling tube.

(b) The elastic modulus from five uniaxial compression tests ranges from 85 to 180 MPa. A value of 150 MPa may be considered to be representative.

(c) The uniaxial compressive strength varies from 560 kPa to 1230 kPa from the five tests, averaging 830 kPa.

(d) Results of five multistage drained shear box tests indicated c'=340 kPa and $\phi'=24^{o}$ within a range of scatter. The residual strength measured from the five shear box tests yielded $c'_r=0$ kPa and $\phi'_r=15^{o}$.

(e) Results of two swell tests indicated the weak mudstone possesses significant swelling behaviour. The horizontal swelling potentials and calcite contents measured are consistent with the relationships of these two parameters established for other shales. From the results of mineralogical and chemical tests, it is considered that probable causes of swelling include salt concentration in the pore water in the rock, presence of smectite and low calcite content.

It should be emphasized that the number of tests performed are limited in this study and the findings need to be verified before they are applied to future projects.

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