Loading Efficiency and Stiffness of the Lea Park Formation at Borden Bridge

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ABSTRACT
The research demonstrates the effect of temporal changes in subsurface pore pressures at the Borden Bridge landslide which are shown to impact the landslide's rate of movement. A historical perspective of the area shows the failure is being driven along a weakened layer of residual clay shale roughly 40 m below surface. Changing pore pressure dynamics due to seasonal fluctuations influence the modulus and shear strength of the residual clay shale. These changes have the potential to affect the stability and contribute to delayed failure mechanisms for the landslide. Several vibrating wire piezometers and a barologger were recently installed on site. Using the collected dataset, the deformation modulus can be interpreted within the Lea Park Shale. The data presented in this paper includes the stiffness of the residual shale formation below the groundwater level based on historic laboratory results and from in-situ methods.

RÉSUMÉ
La recherche démontre l'effet des changements temporels dans les pressions interstitielles du sous-sol au glissement de terrain du pont Borden, dont on sait qu'ils influent sur le taux de déplacement du glissement de terrain. Une perspective historique de la région montre que l'échec est entraîné le long d'une couche affaiblie de schiste argileux résiduel à environ 40 m sous la surface. La modification de la dynamique de la pression interstitielle due aux fluctuations saisonnières influence le module et la résistance au cisaillement du schiste argileux résiduel. Ces changements ont le potentiel d'affecter la stabilité et de contribuer à retarder les mécanismes de défaillance du glissement de terrain. Plusieurs piézomètres à fil vibrant et un barologger ont été récemment installés sur le site. En utilisant l'ensemble de données collectées, le module de déformation peut être interprété dans le schiste de Lea Park. Les données présentées dans cet article incluent la rigidité de la formation de schiste résiduel au-dessous du niveau de la nappe phréatique basée sur les résultats de laboratoire historiques et des méthodes in-situ.

1 INTRODUCTION

Approximately 12 km south-east of Borden, Saskatchewan three bridges for Highway 16 cross the North Saskatchewan River as shown in Figure 1. The three bridges were constructed in 1937, 1985, and 1997 respectively (Panesar et al., 2014). The original bridge was opened in 1937 and decommissioned in 1985 as construction was completed for the new Borden Bridge, approximately 70 m downstream to the original (Wilson et al, 1989). The final bridge span was completed in 1997, located between the original and second Borden Bridge, resulting in 35 m approximate spacing between bridges (Panesar et al., 2014). In 1937, the river channel was narrowed to reduce the length of the bridge structure (Wilson et al., 1989). In 1983, movements of the east abutment required the construction of a toe berm to increase the factor of safety during bridge construction (Wilson et al., 1989). The berm extends 20 m into the river channel. In 1995, the eastern abutment and the berm were widened for the third bridge. The eastern abutment showed evidence of movement, whereas the west abutment appeared to be stable (Panesar et al., 2014).

Figure 1. Map showing Borden Bridge location (Antunes, 2001).

1.1 Preliminary Stiffness of Lea Park Shale

The bridges were built on landslide debris on the eastern abutments. The North Saskatchewan River Valley in this area has many landslides caused by high plasticity clay shales, which have a high montmorillonite content. Some other notable slides in the area include the Denholm Slide and the Maymont Slide. 1 km downstream of the Borden Bridge a CN railway bridge is also impacted by a landslide. The landslide activity is exacerbated by the potential for pre-sheared zones within the clay shale from glacial thrust, previous mass wasting or interstitial erosion and weathering.

When unweathered, the clay shales exhibit a relatively high cohesion and friction angle. Weakening of the clay shales occurs from a combination of weathering and shear stresses. Minor strains (< 10%) can reduce the clay shales to their residual strength resulting in rapid failure and potentially high consequence mass wasting (Clifton et al., 1995). The failure mechanism in the Cretaceous clay shales typical to the western Prairies historically results in progressive sliding on a near horizontal shear zone which originates from a common slip surface. Changes in pore pressures have been shown to influence this landslide's rate of movement (Clifton Associates, 2014).

This paper's objective is to obtain preliminary stiffness values for the Lea Park formation in-situ, using barometric compensation and its corresponding loading efficiency (van der Kamp & Gale, 1983). This study will look to compare lab stiffness values from oedometer testing with in-situ determination of stiffness. Further study of this data will be used to help correlate the reduction in stiffness and pore-water dynamics to the landslide rate of movement.

1.2 Regional Geology

The Borden Bridge area is part of the Saskatchewan Plains characterised by a westerly thickening deposit of Cretaceous sedimentary clay shales and limestones. The geology is typical to the Interior Plains physiographic region of western Canada. The overburden was formed through glacial activity during the Quaternary glaciations and the advance and subsequent melt-out of the Laurentide ice sheet approximately 12,000 years ago (Christiansen, 1979). The fluvial lacustrine deposits consist of silty clay and sandy clay on the eastern shore of the bridge crossing. On the west side of the Borden Bridges (where the CN line crosses Highway 16). Christiansen (1997) reports that the western flood plain appears stable and consists of aeolian dunes. The slide on the east side is believed to be resulting from river downcutting into the bedrock shales in the last deglaciation (Wilson et al., 1989). Downcutting and bank erosion is ongoing and is considered to be a contributing factor to current bank instabilities (Panesar et al., 2014)

1.3 Local Geology

The eastern side of the river was infilled with varying depths of alluvial sand from the Empress groups over glacial drift from Sutherland and Saskatoon groups (Christiansen, 1996). Below the glacial drift is landslide debris with evidence of slickensides, brecciation, large disturbance, and fracturing at depth. The debris regions contain clays, silt and sand interbedding and are believed to be part of the Judith River Formation (Christiansen, 1996). This feature is absent on the western wall of the valley (Christiansen, 1997). The Judith River Formation overlies the Lea Park Formation shale (Christiansen, c1996). The Lea Park Shale is typically silty downward grading into very high plasticity, bentonitic, marine clay shale, (Christiansen, 1996, Clifton Associates, 1983). The contact between the upper Lea Park Shale and the Judith River Formation is a sharp unconformity and is believed to indicate sliding on the eastern valley wall (Wilson et al., 1989, Clifton Associates, 1983).

1.4 Site Stratigraphy

On the eastern bridge abutments, the site stratigraphy is slightly variable and dependant on the topography of the east valley wall. The basic stratigraphy from 5 boreholes drilled in 2014 is shown in Table 1.

Table 1. Site stratigraphy (Panesar et al., 2014)

Stratigraphic Unit	Formation	Approximate Top Elevation Above Sea Level
Alluvial Sands	Post Glacial	Variable
Glacial Drift	Glacial Deposits	Variable
Disturbed Shale	Judith River	~ 420 m – 450 m
Intact Highly Plastic Clay Shale	Lea Park	~ 410 m
Sandy Clay Shale	Lea Park	~ 408 m

1.5 Groundwater Regimes

Evidence from saline patches along Highway 16 from Langham to the Borden Bridge area is indicative of high groundwater levels (Christiansen, 1996). The high groundwater levels are believed to be originated from the Dalmeny Aquifer (Christiansen, 1996). However, from site investigations conducted by E.A Christiansen Consulting Ltd. (1996) it appears as if the Dalmeny aquifer may not be present at the Borden Bridge site. The two groundwater systems found at the east side of the valley are an upper system in the surficial drift, and a lower system located within the shales at depth (Clifton Associates, 1995). The upper system is within 10 m of surface, and the lower system is located within pre-glacial sediments 3 to 5 m above the base of the valley (Clifton Associates, 1995). Both groundwater regimes are being drawn down by the presence of the North Saskatchewan River (Clifton Associates, 1995).

1.6 Historical Site Monitoring

The site has undergone extensive drilling and monitoring programs in the past to investigate the deep landslide. Prior to bridge construction in the 1980s, a combination of geophysical logs including E-logs, natural gamma logs, and caliper logs were taken with drilling samples at 0.6 m intervals (Clifton Associates, 1982 and 1983). Additionally, undisturbed tube sampling was completed at approximately 1.5 – 3.0 m intervals. Selected laboratory tests were carried out including soil classification and grain size analysis. Piezometric head and slope movement was monitored during and after the second bridge construction (Clifton Associates, 1982 and 1983).

In 1995, a report by Clifton Associates was completed detailing similar laboratory testing and installation of more extensive slope movement instrumentation (Clifton Associates, 1995). The additional instrumentation included slope inclinometers to monitor the movement of the slope, before and after construction (Clifton Associates, 1995). The average rate of movement at this time was 2 to 3 mm per year (Clifton Associates, 1995).

Due to a relatively rapid increase in movement around 2009 that impacted the existing bridge abutments, a follow up site investigation was completed by Clifton Associates in 2014. The purpose of the investigation was to explore remediation options to slow or stop the slope movements. The 2014 report detailed the monitoring results from slope inclinometer and piezometer instrumentation. Based on information presented in both reports, the primary failure surface of the landslide was determined to be located between 412 masl and 415 masl (Clifton Associates, 2014). The landslide is believed to be moving on a softened pre-sheared zone at the contact between the upper Lea Park Formation and the Judith River landslide debris (Clifton Associates, 1995).

The 2014 report intended to detail any correlation between increased movement in the landslide and increased pore-water pressures in the slope (Clifton Associates, 2014). The report found that prior to 2009 the east abutment showed only minor slope movement with an average rate of 0.8 mm to 2.0 mm per year, though some years showed no movement (Clifton Associates, 2014). In 2010, the rate of movement increased to 26.4 mm per year before slowing down again in 2011. Prior to 2012 the total displacement of the SI's were approximately 35 mm (Clifton Associates, 2014). In 2012, large displacements similar to 2010 were observed and caused the slope inclinometers to shear off (Clifton Associates, 2014). The piezometric head was compared to the slope inclinometer data and revealed a spike in pore-water pressures that corresponded with movement in the slide.

1.7 Historical Lab Testing

Testing throughout this site's history has focused on shear strength parameters and soil classification.

Throughout 1982, the testing done for Borden's Lea Park formation shale was not sufficiently detailed and back analysis was used to establish values for cohesion c' and internal friction angle, ϕ'. In 1983, further testing was performed to gather shear strength parameters including unconfined compression, consolidated undrained triaxial, direct shear, and consolidation tests. The consolidation testing reported values for initial void ratio e_0 (0.77-1.14) and specific gravity G_s ~2.72 and were used to calculate the associated oedometric parameters. (Clifton Associates, 1982 and 1983)

With respect to testing at Borden, the Clifton Associates consulting report from 1995 only added some visual inspections, soil classifications, and dry unit weight tests (Clifton Associates, 1995).

2 METHODOLOGY

2.1 Theory

Barometric pressure change can be used to determine loading effects and hydraulic properties of an aquitard (Jacob, 1940). The in-situ elastic properties of soil formations have been determined in previous studies using the pore pressure response from surface loading (van der Kamp & Schmidt, 1997; Barr et al., 2000; Smith et al., 2013; Smith et al., 2016). A change in atmospheric pressure has been shown to cause pore pressure fluctuations in both aquifers and aquitards (Bardsley & Campbell, 2007; Barr et al., 2000; van der Kamp & Schmidt, 1997; Anochikwa et al., 2000; Rojstaczer & Agnew, 1989; Sophocleous et al., 2006). To find the instantaneous pore pressure response due to surface loading, sensitive pore pressure transducers are coupled with dataloggers as part of a long-term monitoring plan. As the measured piezometric surface responds to barometric pressure changes from atmospheric changes, the in-situ compressibility m_v for the Lea Park Shale can be calculated (Weeks, 1979).

Recent studies have confirmed changes in groundwater pressure correspond closely to changes in mechanical loading (Skempton, 1954; Anochikwa et al., 2012). In the following study, understanding loading efficiency (γ) is valuable because it represents the ratio of pore-water pressure change to vertical stress change in a laterally constrained condition. The loading efficiency ranges from 0 to 1.0 and is related to an infinitely stiff or soft material respectively. Horizontal loads can be neglected because the vertical load will not result in horizontal flow (van der Kamp & Gale, 1983). Vertical stresses in a soil formation are more easily quantified and enable the use of loading efficiency in field applications (van der Kamp & Gale, 1983).

The loading efficiency represents the portion of the barometric induced load change that is supported by water (van der Kamp & Schmidt, 1997). It has been shown that for clay-rich aquitards, the compressibility of the material is high relative to water and would yield values slightly below 1.0 (van der Kamp & Schmidt, 1997). Finding the loading efficiency for a selected formation may be completed through analysis of barometric loading response (van der Kamp & Schmidt, 1997)

The loading efficiency and one-dimensional compressibility m_v of the Lea Park Shale was determined using the method of van der Kamp and Gale, (1983).

$$m_v = \frac{gnb}{1- g} \qquad [1]$$

The porosity of the formation is denoted by n and was assumed based on the initial void ratio of the shale of 0.88. The assumption is based on the understanding that the in-situ void ratio is subject to change due to sample disturbance. The bulk compressibility of water, β is taken as 4.6 x 10^{-7} kPa^{-1}. The m_v can be converted to a constrained modulus M by using $M = 1/m_v$. Using an assumed undrained Poisson's ratio of 0.38, similar to that of Pierre shale (Smith et al., 2018), the constrained modulus can be used to estimate a deformation modulus, E. A brief sensitivity analysis found that when the Poisson's ratio was decreased from 0.38 to 0.35 it showed a deformation modulus increase of 16.7%, whereas when the Poisson's ratio was increased to 0.41 the deformation modulus decreased by 19.5%. Considering these not insignificant differences, it is clear that accurate determination of the undrained Poisson's ratio will be required in the future. The range of Poisson's ratio for the Lea Park Shale was chosen in accordance with the Pierre shale range found by Smith et al., (2018). Further determination for the Poisson's ratio of Lea Park Shale will be conducted in the continuation of this study.

Estimating the loading efficiency included correcting for the barometric pressure with Equation 2.

$$p^* = \left[p_t - B_{ave}\right] - g\left(B - B_{ave}\right) \qquad [2]$$

where p^* is the corrected pore pressure, and p_t is the uncorrected pore pressure measured on site.

Pore pressure responses related to the fluctuation of atmospheric pressure display an inverse relationship with open standpipe piezometers and directly with sealed piezometers (Smith et al., 2013). The piezometers on site were sealed with grout and may be assumed to directly respond to the barometric loading.

An estimation of oedometric modulus E_{oed} was found from the recompression (loading) curves of the oedometer testing completed in the Lea Park Shale (Clifton Associates, 1983). The oedometric modulus can be converted to a deformation modulus using the assumed Poisson's ratio.

3 PROCEDURE

3.1 In-situ Data Acquisition

The pressure data for this research was acquired using a weather station and 6 nests of 3 piezometers each at varying depths. The deepest piezometer in each borehole was installed in the Lea Park Shale. The boreholes were designated BH501 to BH506. BH501 and BH506 have piezometric data dating to September 2014. BH503, BH504, BH505 have data extending back to October 2017. Unfortunately, the data from BH502 was not recorded due to an unknown issue. The weather station was installed in September 2017 and as a result, the data for the loading efficiency calculations are limited to this period.

Figure 2. BH504 data acquisition

3.2 Data Manipulation

The barometric data from the weather station was correlated to pore pressure readings from the boreholes. The pore pressure results were then corrected using Equation 2 for time intervals of one week. The loading efficiency for the Lea Park shale was determined based on visual data comparison with the corrected pore pressures. Once a loading efficiency was estimated, the temporal changes in compressibility, m_v could be calculated. Inconsistency in the dataset caused considerable scatter when compensated for barometric changes and therefore, significant noise preventing accurate data interpretation.

The data points for each datalogger were logged at differing time intervals every 4 hours. The original intent of this pore pressure data was not to be used for in-situ loading efficiency calculations but for pore pressure monitoring on site. BH501 and BH506's data were recorded on a 4-hour interval with each point being recorded at the beginning of the hour, whereas the other borehole's points were not recorded at the beginning of the hour. For this reason, BH 501 and BH506 are most suitable to match with the hourly logged data of the barometer.

Of the two suitable boreholes, the data from BH506 appeared to have more noise and therefore BH501 was used as the main focus for the preliminary results of this study. The oedometer testing done on the Lea Park Shale was taken from samples drilled within the river for the design of the bridge piers. BH501 was close to the river and for this preliminary study of stiffness the in-situ and laboratory areas where the data were gathered are assumed to be representative.

Figure 3. Borehole location plan

At some intervals, the datalogger was unable to record any pressure readings and those points were removed from the dataset to eliminate potentially erroneous data points. The points without pressure readings appeared to be due to unknown error within the dataloggers and appeared randomly throughout monitoring. Due to the 4-hour timesteps between pore pressure measurements, the subtle differences in barometric pressure change are less evident.

3.3 Lab Data

The lab data used for this study were taken from previous oedometer testing (Clifton Associates, 1983) conducted as part of the first Borden Bridge twinning. The samples taken for the consolidation testing in 1983 were taken from drilling located on the river bed parallel to BH506, BH502, and BH501 as shown in Figure 3 (Clifton Associates 1983). Each oedometer test in the Lea Park Shale was used to find a oedometric modulus which was then converted to deformation modulus *E*.

4 PRELIMINARY RESULTS

Each borehole was equipped with a vibrating wire piezometer installed into the Lea Park Shale. The calculated stiffnesses determined along the assumed slide plane within the Lea Park Shale are shown in Table 2.

Table 2 – Mechanical in-situ properties of Lea Park Shale at Borden from BH501 – Depth 414.7 masl (potential slip surface)

Date Range	Loading Efficiency γ	Compressibility m_v (kPa^{-1})	Coefficient of Determination R^2
10/31/17 to 11/07/17	0.36	1.19 x10^{-7}	0.91
11/14/17 to 11/21/17	0.63	3.60 x10^{-7}	0.72
1/11/18 to 1/17/18	0.54	2.48 x10^{-7}	0.84
2/07/18 to 2/14/18	0.68	4.50 x10^{-7}	0.81
2/22/18 to 3/01/18	0.58	2.91 x10^{-7}	0.88

Figure 4 displays the corrected pore pressure change over time at the current time intervals that were available. Visual interpretation of the loading efficiency, γ required an iterative approach and judgement until the barometric fluctuation is minimized for the corrected pore pressure data (Smith et al., 2013). The analysis was performed based on weekly intervals starting at the onset of the available barometric data. The fitted values for loading efficiency based on the data available are shown in Table 2.

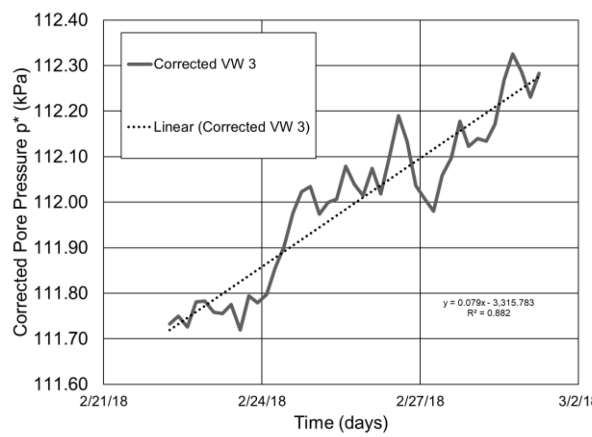

Figure 4. BH501 corrected pore pressure vs. time. Determining loading efficiency visually while correcting for barometric loading in Lea Park Shale

The data acquisition and analysis for the Lea Park Shale could change as more data is collected and the sample frequency is increased. The current change in the in-situ m_v with time is shown in Figure 5.

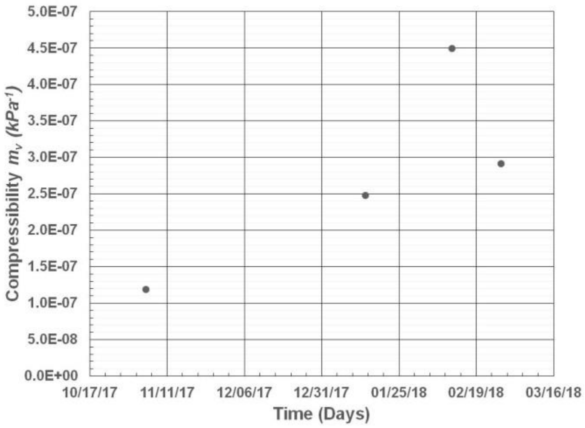

Figure 5. Change in In-situ compressibility m_v due to barometric loading over the time of monitoring

The general trend of in-situ compressibility from Figure 5 shows an increase over the time it was monitored. An increase in compressibility suggests a decrease in stiffness. The stiffness of the Lea Park Shale was expected to decrease with increased shear strains. However, without more data the trend for in-situ stiffness reduction at the Borden Bridge site may not be fully representative.

The data from the oedometer tests completed in 1983 by Clifton Associates were used to find the constrained modulus of the Lea Park Shale. This was done by finding the change in stress over the change in strain for the recompression (loading) line of each oedometer test that was located in the Lea Park Shale. The results of a consolidation test done on sample of the upper Lea Park Shale taken from beneath the river bed at an elevation of 423.1 masl is shown in Figure 6 (Clifton Associates, 1983).

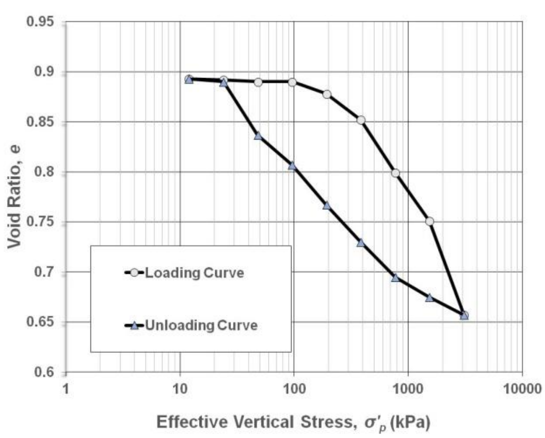

Figure 6. 1-D consolidation test results of upper Lea Park Shale from beneath the North Saskatchewan River bed. (Clifton Associates, 1983)

Table 3 – Laboratory stiffness properties of Lea Park Shale at Borden Bridge

Elevation (masl)	Oedometric Modulus E_{oed} (MPa)	Deformation Modulus E (MPa)
419.3	11.3 – 30.7	6.0 – 16.4
434	3.7 – 8.5	2.1 – 4.5
423.3	8.4 – 148.9	4.5 – 79.5
423.1	24.6 – 410.9	13.1 – 220.0
411.5	592.0	316.0
429.3	18.0 – 60.0	9.6 – 45.6
418	42.5 – 100.0	22.7 – 53.5
422.5	11.7 – 150.1	6.3 – 80.2

The laboratory values for deformation modulus *E were* found to vary between 2.1 MPa to 316 MPa. The deformation modulus *E* for both lab and in-situ measurements were converted using an assumed Poisson's ratio (0.38).

The use of oedometer and in-situ testing for producing a stiffness degradation curve is directly comparable. This is because both samples are laterally confined and the degree of horizontal stress increase (in a perfectly undisturbed lab test) is constrained by the coefficient of lateral earth pressure at rest, K_0. It is important to understand that the in-situ method is considered to have a higher degree of reliability as it is expected that upon recovery of the shale, the sample would undergo considerable unloading, resulting in yielding of the sample and complete destruction of any diagenetic bonds that may have been in place in-situ. In this same vein, it would be expected that a borehole would have the same impact in the near field rock surrounding the borehole. However, flow into and out of a well pack is generally constrained by the intact rock outside of the damaged zone. It is therefore expected that the damage around a borehole has little to no influence on the predicted stiffness values using barometric compensation. As a result, data logging concerns aside, provided that the loading efficiency was found to have a coefficient of determination greater than 0.8, then it is expected that the data is reliable and the calculated stiffnesses reasonable.

For Figure 7, the oedometer stiffness (green triangles) represents the lab data used to find the deformation modulus of the soil. The stress increments in the elastic zone of the recompression line for each oedometer test were used with the associated strain to find an oedometric modulus, E_{oed}. The oedometric modulus was then converted using Poisson's ratio to a deformation modulus, E. The barometric loading data (orange circles) represent the in-situ deformation modulus of the soil using the maximum stress applied over each week of loading with the deformation modulus to find the associated strain increment.

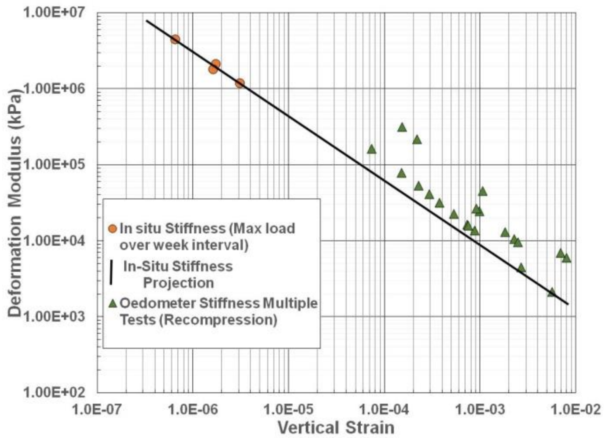

Figure 7. Relationship of deformation modulus with respect to vertical strain increment

Figure 7 illustrates the degradation of stiffness with strain in the Lea Park Shale using both in-situ and laboratory methods to establish stiffness. Clearly since the laboratory testing plots above the in-situ line, there are concerns with the in-situ results. It is suspected that the results of the in-situ moduli are a function of lower sample rates. These results are only preliminary and will be updated in the future when more information becomes available.

5 DISCUSSION

5.1 Results

The relationship shown in Figure 7 is the preliminary stiffness degradation curve that may be subject to change as more data is collected.

With respect to data acquisition, the reduction of noise for the loading efficiency calculations can be improved by increasing the frequency of data collection and synchronizing the barometer and piezometer readings.

The determination of degradation of a soils stiffness with increased strain can help determine an appropriate modulus used in a numerical environment based on the anticipated strain increments (Clayton, 2011). Figure 7 illustrates the degradation of stiffness in the Lea Park Formation with increased strain as well as the reliability of the laboratory testing. Any points below the projected line of best fit through the in-situ data indicate a level of damage to the test sample (Smith et al., 2018). However, as shown in Figure 7, all laboratory tests are above the projected line which either indicates that the in-situ or the laboratory determination of stiffness is not fully representative. It is more likely that the in-situ stiffness is correct because there is no yielding of the soil during the in-situ determination of stiffness.

5.2 Future work

Possible future expansion of the work may include the correlation of the in-situ stiffness change to slope movement monitoring. Slope movement monitoring may be accomplished either with marked survey points or photogrammetry. Using the monitoring data, the change in stiffness for the slip surface material could be linked to the slope’s movement.

Changing the data acquisition on site could increase in the accuracy and estimation of the in-situ stiffness and modulus at Borden. Over time, more data will be gathered involving seasonal changes and pore pressure fluctuations which will help to further identify and correlate the rate of movement with the change in stiffness.

It would be helpful to gather shear strain data to relate the stiffness to shear strain. Additional testing would be valuable to determine a Poisson’s ratio and a more representative void ratio for the Lea Park Shale. Further laboratory testing will be conducted to better establish the lower end of the stiffness relationship shown in Figure 7. In-situ data is beginning to show the degradation of stiffness, but improved data collection and analysis may help characterize the subtle changes caused by barometric loading which could impact the stability of the landslide.

Additional methods for determining the loading efficiency in the shale using barometric loading such as linear regression approaches or barometric response functions also exist and may be employed for this study (Tipman & Barbour 2017). The validity of other methods is further discussed in Tipman and Barbour (2017).

The data gathered, and work done to date is considered preliminary with the intent to establish a rigorous method for studying the impact of changes in stiffness in the future.

6 CONCLUSION

The use of in-situ testing to determine engineering properties of soil is proving to be a valuable tool in civil, geological, and environmental engineering practices. Combining field and laboratory tests can further increase the understanding of soils on can potentially be used to correlate seasonal and weather events to slope movement. At Borden, through the use of barometric and pore pressure monitoring a preliminary stiffness degradation curve has been produced. The early trend appears to be a decrease in stiffness over time. Further studies and data collection will attempt to correlate the stiffness degradation to the slope’s movement.

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