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A UNIFYING FRAMEWORK FOR MODELING SWELLING SOIL BEHAVIOUR

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ABSTRACT

The destructive effect of expansive soils on infrastructure is one of the classical issues associated with unsaturated soils. Given the continuing challenges associated with swelling soils there is still a need for a practical analysis method. The Swelling Equilibrium Limit (SEL) provides a concept for modeling expansive soil behaviour. The SEL is an upper bound swelling limit, which depends on the soil's initial state and the boundary conditions upon wetting. The SEL is obtained experimentally from swelling tests performed by applying a range of swelling scenarios from rigid boundary condition to free swelling without confinement. In this paper, improvements to the triaxial swelling apparatus are reported, SELs for two new swelling soils are characterized, and a framework for SEL prediction is developed. The SEL framework is developed to relate SEL parameters to index properties as well as the soil's initial volume. SEL parameters are correlated to liquid limit, plasticity index, free swell potential, and specific volume. The relationships are used to estimate the SEL for Regina clay, which is shown to agree with experimental data.

Keywords: expansive soil, experimental, unifying framework, unsaturated soil

INTRODUCTION

Swelling-induced damage to infrastructure costs billions of dollars annually. On the positive side, applications such as landfills rely on the same swelling potential to create self-healing environmental barriers. Significant research is focused on challenges associated with swelling soils (Komine and Ogata 1994; 2004; Miller et al. 1995; Yilmaz 2006; Nagaraj et al. 2009; Puppala and Cerato 2009; Siemens and Blatz 2009; Houston et al. 2011; Kayabali and Demir 2011; Rao et al. 2011; Kodikara 2012; Powell et al. 2013). Vanapalli and Lu (2012) identified twenty-five different methods for heave prediction grouping them into empirical, oedometer, and suction methods. As challenges associated with swelling soils are still being encountered and the swelling potential is required for environmental barriers, a unifying framework for swelling behaviour is a valuable tool for practitioners and researchers. Ideally a unifying framework would link experimental results to a practical numerical modeling tool.

Much effort has been invested in development of advanced laboratory testing for swelling soils (ASTM D4546; Wiebe et al. 1998; Yong 1999; Al-Shamrani and Al-Mhaidib 2000; Aversa and Nicotera 2002; Sánchez et al. 2008; Wang et al. 2012; Cui et al. 2014; Liu et al. 2014). Typically, swelling tests are conducted in one-dimensional oedometer apparatuses (ASTM D4546) where a disc-shape specimen is placed inside a rigid ring. The specimen is given access to water to activate swelling. Expansion or swelling-induced pressure is measured with a displacement sensor or a loading frame (Ferber et al. 2009; Baille et al. 2010; Mohajerani et al. 2011; Powell et al. 2012a; Powell et al. 2012b; Abbas et al. 2015). One of the drawbacks of oedometric apparatuses is the rigid lateral constraint imposed on the soil specimen (Al-Shamrani and Al-Mhaidib 2000). The rigid boundary condition confines the specimen to deform only in vertical direction, which may or may not be representative of field applications. Al-Shamrani and Dhowian (2003) concluded that oedometer test overestimated measured field data but a triaxial swelling test gave better prediction. Swelling soil testing in a triaxial

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apparatus has the advantages of allowing more generalized conditions and direct measurement of state variables (Al-Mhaidib 1998; Al-Shamrani and Al-Mhaidib 2000; Al-Shamrani and Dhowian 2003). Siemens and Blatz (2007) reported a new triaxial swelling test that included the capacity to automatically apply boundary conditions as well as allow for suction measurement (Wheeler and Sivakumar 1995; Rampino et al. 1999; Sivakumar and Wheeler 2000; Lloret et al. 2003; Sun et al. 2007).

Adem and Vanapalli (2015) reviewed the methods for modeling expansive soils and divided models into three categories: consolidation theory-based methods, water content-based methods, and suction-based methods. Modeling analysis contributes to the applicability and practicality of advanced analysis methods for use in practical engineering problems (Fredlund 1983; Rees and Thomas 1993; Miller et al. 1995; Briaud et al. 2003; Masia et al. 2004; Ito and Hu 2011; Puppala et al. 2014). Puppala et al. (2014) developed a model that is based on seasonal moisture content to predict swelling and shrinkage deformations. Constitutive models have also been developed that capture swelling soil behaviour while only requiring nominally more parameters compared with other advanced constitutive models (Shuai and Fredlund 1998; Sheng et al. 2008).

A common approach to interpreting general trends in experimental results and constitutive models is to relate engineering parameters to basic soil properties. For example, Nagaraj and Srinivasa Murthy (1986) and Burland (1990) related soil compressibility to index properties, thus allowing for interpretation of soil structure. Additionally, swelling potential has been related successfully to the soil index properties (Shi et al. 2002; Komine and Ogata 2003; Prakash and Sridharan 2004; Rao et al. 2004; Ito and Azam, 2010; Cui et al. 2012; Ito and Azam 2013). Swelling potential is generally related to the volume state (specific volume, void ratio, $V=e+1$), liquid limit, plasticity index, and free swell potential.

Despite much effort to characterize swelling soil behaviour, general agreement on a practical analysis tool for use by researchers and practitioners has not been found. Siemens and Blatz (2009)

proposed a concept, known as the Swell Equilibrium Limit (SEL). The underlying notion is that expansive soils have a swelling potential that is realized as expansion or swelling-induced stresses. How the swelling potential is realized depends on the mechanical boundary conditions, which range from constant stress to constant volume. An application to illustrate the SEL is a retaining wall constructed using swelling soil that is subjected to infiltration conditions (Figure 1). Three conditions are highlighted on the retaining wall and their stress-volume paths are plotted in specific volume-mean stress (V-p) space. Swelling potential is activated by water infiltration. Near the surface (soil element 1), unconfined swelling conditions prevail as the soil element is at shallow depth. On the plot (Figure 1b), the stress-volume path starts at zero stress at the initial specific volume and proceeds vertically until the swelling potential is satisfied. The second condition (soil element 2) is swelling at depth under a constant mean stress (CMS) equal to the stress from the retaining wall and overlying soil. In V-p space (Figure 1b), the soil element is initially at a stress associated with its overburden and then swells upwardly along a vertical stress-volume path until its swelling potential is satisfied. Under CMS conditions, no additional confinement is provided during swelling. The elevated initial stress along with the CMS boundary causes less swelling compared with the unconfined case (i.e. reduced final specific volume and less change in volume during swelling). The third condition is behind the retaining wall (soil element 3). As the soil swells, volumetric confinement is provided by the retaining wall. Thus, as the soil swells it encounters the stiff retaining wall, which deflects in response to the lateral stress. In V-p space (Figure 1b), an upward sloping stress-volume path results in increases in both volume and stress until the soil's swelling potential is realized. The stiffer the wall, the flatter the stress-volume path. For perfect confinement a horizontal stress-volume path is followed, termed constant volume (CV). For each of the three conditions the soil swells until its swelling potential is satisfied. Connecting the 'swelled state' points on Figure 1b forms the SEL.

The SEL is an upper limit to swelling that defines the end-of-swelling state. The SEL concept provides a basis to model swelling soil behaviour. The end-of-swelling state is determined from the initial specific volume (related to dry density) and mean stress as well as the boundary conditions during swelling. Siemens and Blatz (2007) developed a triaxial swelling test, which allows for general conditions to be applied during a swelling experiment. Lim and Siemens (2013) developed an unconfined swelling test that incorporates non-contact method to measure soil deformations. Thus, the range of field boundary conditions presented in Figure 1 can be applied experimentally. Currently SELs for a compacted 50:50 bentonite-sand mixture (named bentonite-sand-buffer) and natural Bearpaw shale have been characterized (Siemens and Blatz 2009; Powell et al. 2013). Of interest is the potential to relate SEL parameters to fundamental expansive soil properties. This would allow for estimation of SELs for new soils for use in preliminary analysis. If significant swelling issues are anticipated, an experimental program could then be initiated. In this paper, the swelling results for two swelling soils, Lake Agassiz clay and Bearpaw clay, are reported and their SELs are interpreted. The SEL parameters for the characterized soils are successfully related to expansive soil index properties and initial specific volume. Finally the SEL for Regina clay is successfully estimated and compared with published data.

MATERIALS AND METHODS

Materials

The swelling properties of three expansive soils: Lake Agassiz clay, Bearpaw clay and bentonite-sand-buffer, are examined in this study. Index properties of the soils are summarized in Table 1 and Figure 2. The swelling soils examined for this study were selected to have a range of clay fraction, montmorillonite fraction, consistency limits, free swell potential, and activity (Table 1).

Lake Agassiz clay is a high plasticity freshwater clay (Graham et al. 1983) of glacial origin from southern Manitoba, Canada. The consistency limits are $LL=85\%$, $PL=34\%$, and $PI=51\%$ (ASTM

D4318) and the grain size distribution (Figure 2a) consisting of 75% clay and 20% silt. Modified optimum dry density and water content are 1.62 Mg/m^3 and 20% (ASTM D1557). Soil specimens were prepared to modified optimum dry density and water content.

Bearpaw clay, taken from the Bearpaw formation, consists of predominately marine silty clays and sands. The consistency limits of the Bearpaw clay were $LL=67\%$, $PL=26\%$ and $PI=41\%$. The $LL=145\%$ and $PI=122\%$ of natural Bearpaw shale were higher than the soil tested due to drying of samples prior to specimen preparation (Rao et al. 1989). The clay fraction and montmorillonite content for Bearpaw shale is 39% (Figure 2a) and 72% (Bryne and Farvolden 1959), respectively. Bearpaw shale, in its natural state, was previously tested for its SEL (Powell et. al. 2013) and the same soil was re-used in this research. The Bearpaw shale was dried, pulverized, and compacted to a target dry density and moisture content of its insitu state: 1.50 Mg/m^3 dry density and 30% gravimetric moisture content. The preparation procedures removed depositional and aging effects (Powell et al. 2012b) and allowed for comparison with the other materials in this study.

Bentonite-sand-buffer consists of 50% sand and 50% Wyoming bentonite by mass (Dixon et al., 1994). The consistency limits of bentonite-sand buffer are $LL=265\%$, $PL=21\%$, $PI=244\%$. Specimens were compacted to target dry density and gravimetric water content of 1.65 Mg/m^3 and 19.4%, which are the specifications for the Canadian concept of a deep geological repository (Dixon et al. 1994).

Triaxial Swelling Specimen Preparation, Apparatus, and Test Method

Test specimens were prepared to a target dry density and water content condition following procedures detailed in Siemens (2006). A required amount of dry mass of pulverized soil was weighed. The dry pulverized soil was moistened thoroughly with a targeted amount of de-ionized water within a mixing bowl. The moist soil was kept in a double-layered sealed plastic bag and placed in the refrigerator for 24 hours for moisture equilibrium. After 24 hours, the moist soil was compacted in 5-

layers in a 50.8mm diameter compaction mold. The compacted triaxial specimen was installed in the modified triaxial apparatus (see Figure 3).

The triaxial swelling apparatus, previously used by Siemens and Blatz (2007, 2009) and Powell et al. (2013), was modified to allow for suction measurement device installation within the triaxial pedestal (Figure 3). Internal suction measurement provides information on the internal moisture response during a swelling test, data to be used in future numerical simulations, and is one of the end-of-test criteria. Previously a suction measurement probe was installed at the center of the specimen, however, the sensor was no longer commercially available. An extensive search for a replacement did not turn up any sensors that met small size and high accuracy requirements. Blatz and Graham (2000) noted that total suction measurements taken near the top cap gave similar readings as those taken at the center of the specimen, however, locating a sensor within the top cap still has size-limiting constraints. No notable effect on mechanical or suction behaviour during isotropic compression or shear loading was found due to compaction of sensors within triaxial specimens (Blatz et al. 1999). Locating the sensor within the pedestal allowed for probe installation prior to the test, which was, operationally, a significantly more efficient method compared with compaction at the center of test specimens. A grooved pedestal cap replaced the lower porous stone installation. The water flowed through the fanning groove at the pedestal cap, through the geotextile and infiltrated the specimen. The chamber beneath the cap contained the relative humidity sensor. On the top of the pedestal cap, a perforated cylindrical tip protruded from the cap. The triaxial specimen was installed on the pedestal with the tip inserted into a pre-bored hole at the bottom of the specimen. The specimen equilibrated with the vapor inside the confined chamber where the relative humidity sensor was housed. Total suction was calculated using the Kelvin equation:

$$\Psi_t = -\frac{RT}{\nu_{w0}\omega_v} \ln(RH) \quad [1]$$

where Ψ_t = total suction, R = universal gas constant (8.314 J/mol K), T = absolute temperature (K), ν_{w0} = specific volume of water (m^3/kg), ω_v is the molecular mass of water vapor (18.016 kg/kmol) and RH = relative humidity.

A typical triaxial swelling test consisted of three phases including isotropic compression at constant water content, suction sensor equilibration, and swelling. Isotropic compression and equilibration occurred as the cell pressure was raised up to a target stress level and held constant. After 24 hours, suction equilibration at the specimen was achieved. To initiate the swelling phase, the back pressure was raised and the plumbing and geotextile were flooded. The initially unsaturated specimen accepted water until the specimen equilibrated with the moisture and stress regimes. The completion of a swelling phase was indicated by less than $\pm 2\%$ changes in water content, total suction, cell pressure, and volumetric strain over a one-day period.

During the swelling phase, specimens were subjected to simulated field conditions by controlling the boundary conditions (Figure 1). The specimens consist of initially unsaturated expansive soil and, therefore, had a swelling potential prior to being given access to water. A constant mean stress (CMS) stress-volume path was one where the specimen was allowed to swell under a constant cell pressure to measure the volumetric swelling potential at a particular stress level. The stress-volume path was set to move vertically along y-axis in a V-p plot (Figure 1b). In the second boundary condition, a constant volume (CV) test, measured the swelling-induced-stress potential at a constant volume. During the CV test, the initial volume of the specimen was maintained constant by increasing the confining pressure. The stress-volume path followed during a CV test moved horizontally in V-p space (Figure 1b).

Unconfined Swelling Test Method

An unconfined swelling test (Lim and Siemens 2013) represented the scenario where the swelling soil was free to expand under zero confinement (notwithstanding the self-weight of the specimen). The results from an unconfined swelling test defined the maximum volumetric swelling

potential. The unconfined swelling tests were carried out in a humidity-controlled chamber. The specimens were given direct access to water through spraying as well as wicking action from a reservoir. Deformations were measured with a non-contact method incorporating digital image analysis (GeoPIV, White et al. 2003).

RESULTS

Typical Results

Triaxial Constant Volume (CV) Swelling Test

Results from a typical CV swelling test for Lake Agassiz clay (CV150) are presented in Figure 4 as mean stress, volumetric strain, total suction, and gravimetric water content versus time (Figure 4a-d respectively). Following preparation and installation, the isotropic compression phase was initiated with cell pressure raised to 150 kPa. After 24 hours, total suction of 9.1 MPa and volumetric strain was 0.72%. The test continued with the CV swelling phase. When water infiltrated into the specimen, it expanded a small, but detectable, amount. The stress-volume path control algorithm activated and increased cell pressure to suppress expansion. The cell pressure increased from 150 kPa up to 750 kPa to maintain initial volume. During infiltration, the water content (Figure 4d) increased asymptotically to 29.0%. The total suction response (Figure 4c) showed a slower rate of dissipation compared to the mean stress and water content plots since the measurement is at the center of the specimen rather than at the perimeter. Total suction dissipated after 5 days of infiltration. The end-of-test criteria were satisfied after 25 days.

Triaxial Constant Mean Stress (CMS) Swelling Test

Results from a typical CMS swelling test for Lake Agassiz clay (CMS150) are presented in Figure 5 as mean stress, volumetric strain, total suction, and gravimetric water content versus time (Figure 5a-d respectively). The mean stress was raised to 150 kPa for the constant water content

isotropic compression and suction sensor equilibration phases. After 24 hours, total suction and volumetric strain values were measured at 6.5 MPa and 0.45%. During the CMS swelling phase, the volumetric strain and gravimetric water content versus time graphs (Figure 5b and 5d) visually resemble each other. As the specimen took on water it also expanded under the constant stress boundary condition. The rate of change for both curves was relatively higher at the beginning of the swelling test phase and then tapered off asymptotically. The end-of-test criteria were satisfied after 35 days.

Unconfined Swelling Test

Figure 6 and Table 3 presents typical unconfined swelling test results for Lake Agassiz clay and Bearpaw clay. The Lake Agassiz clay swelled at a high rate for the first two days after water application. By the second day, the volumetric strain obtained 80% of the total change over the test. Subsequently, the swelling rate leveled off over time. For Lake Agassiz clay, the final total volumetric strain was -70% (in swelling). The Bearpaw clay specimen behaved similarly to the Lake Agassiz clay but at reduced magnitudes for both swelling strain and swelling rate. At the end of the unconfined swelling test the Bearpaw clay specimen had expanded to -46% volumetric strain.

Lake Agassiz Swelling Tests

A series of five triaxial swelling tests were performed on the Lake Agassiz clay. The test series consisted of CMS150, CMS300, CMS600, CV150, and CV300 tests (detailed results for CV150 and CMS150 appear in Figure 4 and 5). Figure 7a plots mean stress versus time for the five tests. Mean stress in CMS600, CMS300 and CMS150 was kept at a constant level, e.g., at 600 kPa, 300 kPa and 150 kPa, throughout the swelling phase. Specimen CV300 required the highest mean stress of 1200 kPa to counteract the swelling potential during the constant volume swelling phase. The second highest mean stress is measured in CV150 which required mean stress of 750 kPa to maintain the constant volume.

A summary of the volumetric strain versus time results for the Lake Agassiz clay swelling tests is plotted in Figure 7b. The maximum swelling strain was measured in the CMS150 as it had the lowest applied mean stress. Volumetric swelling decreased with increasing applied mean stress in the CMS300 and CMS600 experiments. In the two constant volume tests, acceptable control of volume was attained during swelling for both CV150 and CV300 as indicated by the horizontal plots of volumetric strain versus time.

The triaxial swelling and unconfined swelling test results are plotted in Figure 8 in specific volume versus mean stress (V - p) space to allow for interpretation of the SEL for Lake Agassiz clay. The test paths started at a target specific volume of specimen of 1.73 and zero mean stress. The unconfined swelling test swelled along a vertical stress-volume path to the equilibrium value of $V=3.0$. The triaxial swelling tests began with the isotropic compression stage to 150, 300, or 600 kPa mean stress. Results from the isotropic compression stage provided information on the compressibility of the material. During infiltration, the stress-volume path followed the specified boundary condition resulting in a vertical path for CMS and a horizontal path for CV tests. CMS150, CMS300, and CMS600 terminated at specific volumes of 1.99, 1.88, and 1.80, respectively. The CV150 and CV300 swelling tests followed horizontal stress-volume paths during swelling. The test results indicated that a higher mean stress was required in CV300 ($p=1200$ kPa) than CV150 ($p=750$ kPa) to maintain constant volume. Finally, the SEL was formed by optimizing a function to coincide with the end-of-test states from triaxial and unconfined swelling tests.

Bearpaw Clay Swelling Tests

A series of triaxial and unconfined swelling tests were performed on Bearpaw clay (5 tests). Triaxial swelling test results are plotted as mean stress and volumetric strain versus time in Figure 9a and Figure 9b (with detailed results in Lim, 2014). Five (5) triaxial swelling tests are shown performed at isotropic pressures of 200 and 400 kPa. Summaries of the initial and end-of-test conditions are given

in Table 1 and Table 2. At 200 kPa, volumetric swelling strains of -0.1 to -1.1% are measured. The CV400 test only required an additional 10 kPa to maintain the constant volume conditions. Collapse was observed in the two CMS400 tests. In both CMS400 tests, the specimens were brought to 400 kPa isotropic stress and then the swelling phase was initiated. As water infiltrated, the specimens compressed to volumetric strains of 4.1% and 6.7%. Thus 400 kPa was greater than the swelling potential of Bearpaw clay for the conditions tested.

DISCUSSION

In this section the experimental results are interpreted from a SEL perspective and a swelling pressure perspective and then the new unifying framework for modeling swelling behaviour is proposed. The SELs for the Lake Agassiz clay, Bearpaw clay and bentonite-sand-buffer are plotted in Figure 10a-c. The plots include end-of-test mean stress and specific volume states from triaxial swelling tests (open symbols) and unconfined tests (closed symbols), SELs, and the isotropic compression lines. The isotropic compression line denotes the compressibility of the soil when subjected to isotropic compression at constant water content. The SELs were evaluated from a spreadsheet solution assuming a natural logarithm equation in the form of:

$$V_{SEL} = A + B \ln p \quad [2]$$

where 'A' represents the y-intercept at $p=1$ and parameter 'B' reflects the curvature. A spreadsheet was used to minimize the square of the difference between the measured value and the predicted value. A nominal stress of 0.1 kPa is assumed for the unconfined tests to make equation [2] calculable. SEL parameters are listed on Figure 10 and in Table 4. The SELs are comparable with the experimental data with specific volume modeled within ± 0.01 for Lake Agassiz and ± 0.02 for Bearpaw clay.

In addition to the tests reported here, Figure 10 includes a summary of triaxial swelling tests performed on bentonite-sand-buffer (Siemens and Blatz, 2009) in Figure 10c. The plot includes the SEL,

isotropic compression curve, and the swelling potential. The parameters for the optimized SEL parameters are listed on the figure as well as in Table 4. The bentonite-sand-buffer tests were performed prior to development of the unconfined test methodology (Lim and Siemens 2013) so only triaxial swelling results are shown on Figure 10c. The results for bentonite-sand-buffer indicate the SEL captures the observed swelling behaviour. The results also show that the soil is expansive over a wider range of stresses than those examined. At 1500 kPa mean stress, expansion was measured under CMS boundary conditions and an additional 400 kPa of swelling-induced stress was required to maintain the initial volume for the CV1500 test.

The swelling test results in Figure 10 provide insight into the variable swelling potential of each expansive soil. The area between the isotropic compression line and the SEL is highlighted as the swelling potential in Figures 10a-c. The isotropic compression phase sets the starting point for the swelling test. During the swelling phase, the swelling potential is realized as volumetric expansion, swelling-induced stress, or a combination of both depending on the boundary conditions. The results indicate that Lake Agassiz clay is expansive over the range of stresses tested while the maximum swelling potential of Bearpaw clay was achieved. In Figure 10a the SEL for Lake Agassiz clay is approaching the isotropic compression curve, however, the soil still has swelling potential at the maximum stress tested. The CV300 test required 1200 kPa of mean stress to suppress expansion and volumetric expansion was observed in the CMS600 test. Similarly the isotropic compression line and SEL for bentonite-sand-buffer are converging at 2000 kPa (Figure 10c). The SEL for Bearpaw clay connects with the isotropic compression line at approximately 300 kPa (Figure 10b). Swelling tests performed at stresses greater than 300 kPa had nominal swelling-induced-stress or collapse. Thus at stresses below 300 kPa Bearpaw clay is expansive and at stresses above 300 kPa the swelling potential is overcome. This is the first time in which collapse has been observed during this type of triaxial swelling test.

Comparing the size and shape of the swelling potential (indicated by the grey shaded area in Figure 10a-c) gives insight into swelling behaviour from the SEL perspective. A straightforward observation is that a larger shaded area indicates higher swelling potential. Thus bentonite-sand-buffer has the highest swelling potential followed by Lake Agassiz clay and then Bearpaw clay. The shape of the shaded area is a function of the SEL slope and the compressibility of the soil during isotropic compression. The SEL has a steeper slope than the isotropic compression line for the three soils. At the intersection of the two lines the swelling potential is overcome, as in the case of Bearpaw clay. For bentonite-sand-buffer and Lake Agassiz clay, the SEL is asymptotically approaching the isotropic compression line and they will intersect at a mean stress above the available test results.

Interpreting Swelling Pressure with EMDD

Traditionally expansive soils are characterized by measuring the swelling pressure utilizing one or more of the available methods (ASTM D4546; Wiebe et al. 1998; Yong 1999; Al-Shamrani and Al-Mhaidib 2000; Aversa and Nicotera 2002; Sánchez et al. 2008; Ferber et al. 2009; Baille et al. 2010; Mohajerani et al. 2011; Powell et al. 2012a; Wang et al. 2012; Powell et al. 2013; Cui et al. 2014; Liu et al. 2014; Abbas et al. 2015). The major clay constituent that contributes to the swelling potential is the montmorillonite content of the material (Mitchell and Soga 2005). The majority of swelling tests found in the literature are one-dimensional (1D) swelling tests performed in an oedometric apparatus. In order to compare the triaxial swelling test results with results from the literature, two calculations are required. First, as the 1D swelling pressure database is for laterally confined tests, the stresses in the triaxial swelling tests must be converted to equivalent vertical swelling pressures before plotting on Figure 11. Siemens and Blatz (2009) developed a conversion equation to convert 1D swelling pressures to mean stress (or vice versa). The derivation used Hooke's law and the relationship took the following form:

$$p_{equil} = \frac{p_{swell}}{3} \left(\frac{1 + \nu}{1 - \nu} \right) \quad [3]$$

where p_{equil} is the equilibrium mean stress from end of the triaxial swelling test, p_{swell} is the 1D swelling pressure and ν is the Poisson's ratio of the soil.

Second, the montmorillonite component of each material is normalized using the Effective Montmorillonite Dry Density equation (EMDD, Dixon et al. 2002). EMDD is defined as the mass of montmorillonite divided by the volume of swelling materials and voids. An equivalent definition is the mass of montmorillonite divided by the volume not occupied by non-clay and non-swelling materials. EMDD is calculated as:

$$EMDD = \frac{f_m f_c \rho_d}{\left[1 - \frac{(1-f_c)\rho_d}{G_s \rho_w} - \frac{(1-f_m)f_c \rho_d}{G_n \rho_w} \right]} \quad [4]$$

where f_m is the montmorillonite fraction of clay, f_c is the fraction of clay, ρ_d is the dry density of soil, ρ_w is the density of water, G_s is the specific gravity of non-clay soil, G_n is the specific gravity of non-swelling clay. Calculating EMDD allows for comparison of different materials with variable clay content, montmorillonite content, and dry density. The converted swelling pressure for Lake Agassiz clay, Bearpaw clay, and bentonite-sand-buffer as well as swelling pressure results taken from Dixon et al. (2002) are plotted in Figure 11.

Dixon et al. (2002) proposed an exponential relationship between swelling pressure and EMDD, which is:

$$p_{swell} = C * e^{(D * EMDD)} \quad [5]$$

where 'C' is the theoretical value of p_{swell} at $EMDD=0$ 'C' and 'D' is the slope of the line in log-linear space. A spreadsheet solution was implemented to minimize the difference between the experimental data plotted in Figure 11 and Equation [5] for each soil. The output was optimized values of 'C' and 'D' for each soil, which are listed in Table 4. The optimized lines show visual and quantitative agreement with the experimental data as swelling pressures are predicted within an average of 11% of the actual.

The bentonite-sand-buffer data also shows general agreement with the database of 1D swelling pressures with the triaxial swelling test results show less scatter. Comparing the 'C' and 'D' parameters for each soil (Table 4) shows they follow anticipated trends based on their swelling potential. Ranking the materials in order of greatest to least values of parameter 'C' is bentonite-sand-buffer, Lake Agassiz clay, and Bearpaw clay. Ranking the materials in increasing order for parameter 'D' is bentonite-sand-buffer, Bearpaw clay, and Lake Agassiz clay. The highest swelling potential is represented by the highest 'C' (7.609×10^{-3}) value with lowest 'D' (4.56) value in bentonite-sand-buffer. Lake Agassiz and Bearpaw clays have a steeper slope compared to bentonite-sand-buffer (Figure 11). The EMDD lines for the clays are located at lower EMDD owing to their reduced montmorillonite content and dry density. The steeper slope in the clays is an indication that the swelling pressure is more sensitive to increases in EMDD.

Unifying Framework for SEL Prediction

In this section a unifying framework for predicting SELs from basic soil properties is proposed. The SEL concept has been utilized to characterize swelling potential of a number of expansive soils, however, the length of time for each test is 2-5 weeks per test. Therefore the question arises whether SEL equation parameters could be related to more fundamental soil properties as performed by numerous researchers (Shi et al. 2002; Komine and Ogata 2003; Prakash and Sridharan 2004; Rao et al. 2004; Ito and Azam 2010; Cui et al. 2012; Ito and Azam 2013). In a similar focus, Nagaraj and Srinivasa Murthy (1986) and Burland (1990) developed relationships between index properties and compressibility curves. The SEL curves are represented with equation [2] in a V-p plot (Figure 10) or equation [5] for a p_{swell} -EMDD plot (Figure 11). The parameters for these equations for the three expansive soils are listed in Table 4. Thus a parametric study was performed to explore empirical relationships between soil properties and SEL parameters. Liquid limit, plasticity index, free swell potential, and initial specific volume (related to dry density) were found to correlate well to SEL

parameters. Figure 12 and Figure 13 plot the relationships between parameters 'A' and 'B' with the correlated soil properties. The SEL parameters 'A' and 'B' show a visual and quantitative correlation relationship to initial specific volume, liquid limit, free swell, and plasticity index. Using the equations displayed on the figures parameters 'A' and 'B' calculated to within an average of 1.0% and 4.2% respectively. The relatively wide range of values, especially those related to plasticity gives increased confidence to the validity of the relationship. At this point a straight-line equation is utilized to relate the soil properties to the SEL parameters. However, as more soils are characterized a higher order relationship may be justifiable.

The benefit of this framework is to allow for estimation of SEL curves for soils that have not been subjected to advanced triaxial swelling tests. This concept could be used in preliminary design calculations where swelling issues are anticipated. Table 1 lists the index properties and initial volume state of Regina clay, which was characterized by Fredlund (1975). These properties of Regina clay are used to calculate the SEL parameters 'A' and 'B' as detailed in Table 5. The predicted SEL from each pair of parameters varies by just 1-3%. Thus the average of each parameter was used to estimate the SEL for Regina clay ($A=2.644$ and $B=-0.133$) and the preliminary SEL is $V_{\text{Regina}}=2.644-0.133\ln(p)$. The SEL for Regina clay is plotted in Figure 14 compared with the test data from Fredlund (1975). The estimated SEL of Regina clay shows visual and quantitative agreement with the experimental data. Specific volume is predicted within $\pm 20\%$ for the entire range and within $\pm 5\%$ for stresses above 75 kPa thus giving confidence to the unifying framework for the SEL concept.

CONCLUSIONS

The destructive effect of expansive soils on infrastructure is one of the classical issues associated with unsaturated soils. Given the continuing challenges associated with swelling soils the need for a practical analysis method is apparent. The SEL provides a conceptual basis for modeling behaviour of expansive soils. It is an upper bound swelling limit, which depends on the soil's initial state and the

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stress-volume paths upon wetting. The SEL curve obtained from triaxial and unconfined swelling tests encompasses the range of potential swelling scenarios from a rigid boundary to free swell. This paper presents improvements to the triaxial swelling apparatus, SELs for Lake Agassiz clay and Bearpaw clay, and proposed a framework to relate SEL parameters to basic soil properties. The newly characterized SELs allow for development of a unifying framework and preliminary estimation of SEL parameters for yet uncharacterized expansive soils. Using this framework the SEL for Regina clay was successfully estimated. At this time the framework is based on the number of soils characterized (i.e. three) and therefore, further testing is required to build-up the SEL database to increase the range of applicability and refinement of the relationships.

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1 Table 1. Properties of soils used in this study.

Soil	Specific gravity, G_s (-)	Clay fraction, f_c (%)	Montmorillonite fraction of clay, f_m (%)	Liquid Limit, LL (%)	Plastic Limit, PL (%)	Plasticity Index, PI (%)	Free Swell (mL/2g) ⁵	Activity, $A=PI/f_c$	Initial dry density, ρ_d (Mg/m ³)	Initial specific volume, V_o (-)	Poisson's ratio, ν (-)
Lake Agassiz clay	2.76	75	30 ¹	85	34	51	6.5	0.64	1.62	1.73	0.33
Bearpaw clay	2.75	39 ²	72 ³	67	26	41	5.0	1.05	1.50	1.89	0.34
Bentonite –sand-buffer	2.70	50 ⁴	90 ⁴	265	21	244	29	4.88	1.65	1.62	0.18
Regina clay (Fredlund 1975)	2.83	51	77	76	26	50	N/A	1.0	1.54	1.84	0.35

2 Notes: ¹ Dixon et al., 2002.
3 ² Powell et al., 2013.
4 ³ Bryne and Farvolden, 1959.
5 ⁴ Dixon, 1994.
6 ⁵ Measured using ASTM, 2011.

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8 Table 2. Initial and end-of-test measurements for triaxial swelling tests on Lake Agassiz and Bearpaw clays.

Soil	Test ID	Initial (as-compacted)			End of triaxial swelling test						
		Gravimetric water content, w_o (%)	Specific volume, V_o (-)	Degree of saturation, S_r (%)	Gravimetric water content, w (%)	Specific volume, V (-)	Volumetric strain ¹ , ϵ_v (%)	Mean stress, P_{equil} (kPa)	Water uptake (mL)	Degree of saturation, S_r (%)	Dry density, ρ_d (Mg/m ³)
Bearpaw clay	CMS200	29.3	1.87	93	31.9	1.90	-1.1	200	23	98	1.45
	CMS400	27.8	1.88	87	31.6	1.87	0.98	400	1.8	100	1.47
	CMS200 ²	31.8	1.95	92	33.5	1.95	-0.1	200	8.9	97	1.41
	CMS400 ³	30.9	1.95	89	31.3	1.83	6.7	400	2.3	97	1.46
	CV400	29.6	1.91	90	30.8	1.83	4.1	410	1.82	102	1.50
Lake Agassiz clay	CV150	21.6	1.76	79	29.0	1.76	-0.06	750	25	105	1.57
	CMS150	20.3	1.74	77	39.1	1.99	-14.5	150	62	109	1.39
	CV300	19.0	1.72	73	26.3	1.70	1.2	1200	25	103	1.62
	CMS300	21.1	1.75	78	32.4	1.88	-7.3	300	37	102	1.47
	CMS600	19.3	1.75	71	29.2	1.80	-3.8	600	33	101	1.53

9 Notes: All triaxial swelling tests are performed with water pressure at 100 kPa.

10 ¹ negative value represents expansion; positive value represents compression.

11 ² test begins with CMS200, then load up with 200 kPa at each stress increment up till 800kPa under CMS condition.

12 ³ test begins with CMS400, then unload to 200 kPa under CMS condition.

14 Table 3. Initial and end-of-test measurements for typical unconfined swelling tests on Lake Agassiz and Bearpaw clays.

Soil Type	Initial (as-compacted)			End of unconfined swelling test				
	Gravimetric water content, w_o (%)	Specific volume, V_o (-)	Degree of saturation, S_r (%)	Water content, w (%)	Specific volume, V (-)	Volumetric strain ¹ , ϵ_v (%)	Degree of saturation, S_r (%)	Dry density, ρ_d (Mg/m ³)
Lake Agassiz clay	20.6	1.77	74.1	67.3	3.00	-70	93	0.92
Bearpaw clay	30.0	1.88	93.8	61.8	2.74	-46	98	1.00

15 Notes: ¹ negative value represents expansion; positive value represents compression.

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Table 4. Optimized SEL and EMDD- p_{swell} parameters for bentonite-sand-buffer, Lake Agassiz clay, and Bearpaw clay.

Soil	$V = A + B * \ln(p)$	$p_{swell} = C * e^{(D*EMDD)}$
Bentonite-sand-buffer	$V = 3.023 - 0.214 \ln p$	$p_{swell} = 7.609 \times 10^{-3} * e^{(4.56*EMDD)}$
Lake Agassiz clay	$V = 2.691 - 0.141 \ln p$	$p_{swell} = 4.966 \times 10^{-4} * e^{(12.27*EMDD)}$
Bearpaw clay	$V = 2.596 - 0.127 \ln p$	$p_{swell} = 1.566 \times 10^{-4} * e^{(11.73*EMDD)}$

Table 5. Estimated SEL for Regina clay.

Index properties of Regina clay (Fredlund 1975)	Equation	'A'	Equation	'B'
$V_o = 1.838$	Figure 12a	2.638	Figure 13a	-0.133
$LL = 76$	Figure 12b	2.641	Figure 13b	-0.132
$PI = 50$	Figure 12c	2.653	Figure 13c	-0.135
	Average	2.644	Average	-0.133

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Figure 3. A a) schematic and b) photograph of the suction measurement instrument located within the triaxial pedestal.

Figure 10. Swell equilibrium Limits (SELs) for: a) Lake Agassiz clay, b) Bearpaw clay and c) Bentonite-sand-buffer (BSB).