Swell-shrink-consolidation behaviour of expansive Regina clay

Maki Ito
Environmental Systems Engineering – University of Regina, Regina, SK, Canada
Shahid Azam
Environmental Systems Engineering – University of Regina, Regina, SK, Canada

ABSTRACT
Regina clay poses serious damages to various types of civil infrastructure due to periodic volume changes that, in turn, are brought about by seasonal climatic variations. This paper develops a method to correlate the swell-shrink-consolidation behaviour of expansive soils with initial saturation. The swelling potential and swelling pressure for a field sample of the native clay at *S* = 82% measured 1.5% and 3.5 kPa, respectively. The free swelling test data were used in conjunction with results from swell-shrink test to estimate both of the swelling parameters at any initial saturation.

RÉSUMÉ
L'argile de Regina pose des damages sérieux à divers types d'infrastructures civiles dus aux changements de volume ce, à leur tour, sont périodiques provoqués par des variations climatiques saisonnières. Cet article développe une méthode pour corréler le comportement de gonflement-contraction-consolidation des sols expansibles avec la saturation initiale. Le potentiel de gonflement et la pression de gonflement pour un échantillon de l'argile indigène à *S* = 82% ont donné les valeurs de 1.5% et 3.5 kPa, respectivement. Ces données du test de gonflement libre et les résultats obtenus forment tests de gonflement-contraction peuvent être utilisés pour prédire les deux paramètres de gonflement à n'importe à toute saturation initiale.

1 INTRODUCTION

Regina clay is a typical example of expansive soils in arid and semi-arid regions of the globe. The clay deposit evolved due to geologic weathering of glacial sediments under restrained leaching in a proglacial lake (Christiansen and Saure, 2002). This local soil exhibits swelling, shrinkage, and consolidation due to seasonal climatic variations. The resulting engineering problems are manifested by alternate heave and settlement affecting all types of lightly loaded structures in and around the city of Regina (Hu and Hubble, 2005). The ever-increasing failure rate in buried infrastructure as well as the distress and damage in residential and commercial buildings is a serious concern for the city. Most of the observable tribulations can be attributed to changes in the surface layer of the clay deposit that is directly affected by the atmospheric conditions prevalent in southern Saskatchewan (Wilson et al., 1997). To devise sustainable solutions for an aging infrastructure, the geotechnical engineering behaviour of indigenous expansive soils need to be clearly understood.

The main objective of this paper was to correlate the swell-shrink-consolidation behaviour of Regina clay with the degree of saturation. Representative field samples were retrieved from a depth of 0.5 m to 1.0 m in August 2007. The free swelling test was conducted to measure the swelling potential (*SP*) and the swelling pressure (*Ps*) at field saturation. To display volume changes over the entire saturation range, a swell-shrink test was conducted using a modified test procedure. The data from this test were used in conjunction with those from the free swelling test to estimate the swelling parameters at various degrees of saturation.

2 LITERATURE REVIEW

2.1 Swell-Consolidation Testing

The swell-consolidation behaviour of expansive soils is customarily determined using the ASTM Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils (D4546-08). This standard proposes three test methods for various applications:

Method A: The soil specimen is laterally restrained and axially loaded in a consolidometer with access to free water. The specimen is inundated and allowed to vertically swell at a nominal seating pressure (applied by the weight of the top porous stone and load plate) until primary swell is complete. The sample is incrementally loaded thereafter to cancel the deformation due to swelling. Also known as the free swelling test method, this two-stage procedure provides two important swelling parameters, namely: (i) *SP* or wetting-induced volume increase and (ii) *Ps* or loading-induced volume decrease.

Method B: The specimen is subjected to a vertical pressure (such as the *in situ* vertical overburden pressure or structural loading) exceeding the seating pressure. Thereafter, the specimen is given access to free water. This may result in (i) swelling; (ii) swelling then compression; (iii) compression; or (iv) compression then swelling. The magnitude of volume change is measured at the applied pressure after movement is negligible.

Method C: Also known as the constant volume test, this method requires the specimen height to be adjusted by consistently increasing the vertical pressures after inundation to obtain *Ps*. The specimen is subsequently loaded to determine the preconsolidation pressure and unloaded to use the rebound data for heave estimation.

These test methods are associated with several shortcomings. The first class of shortcomings comprises the ones that are unavoidable with the conventional apparatus such as the following: (i) neglecting lateral volume changes and not simulating specimen confinement; (ii) inefficient capture of soil fissures in the conventional odometer ring; and (iii) inadequate simulation of the actual availability of water to the soil. The first two limitations were taken care of by Azam and Wilson (2006) using a thoroughly instrumented large-scale odometer. However, the use of such an expensive apparatus and time-consuming analyses may or may not be justified by every project.

A second set of limitations includes shortcomings that can be rectified. For example, the measurement of *SP* in *Method A* at a token seating pressure (not the actual field overburden pressure) and the determination of *Ps* in *Method C* by applying pressure (not a constant field overburden pressure) while the specimen is inundated with water. *Method B* was designed to avoid this limitation. Likewise, secondary swelling can be added to primary swelling for soils exhibiting a significant amount of long-term volume change. Chemical composition of the inundating liquid can be included in the testing thereby measuring swelling parameters pertinent to certain geoenvironmental applications. Finally, the test data in all of the methods can be corrected for apparatus deformability.

Sample disturbance is unavoidable even with careful handling and transportation. The data obtained from *Method C* are corrected for sample disturbance using the method described by Fredlund and Rahardjo (1993). However, *Method B* and *Method A* do not utilize any corrections to account for discrepancies arising sample disturbance. This renders the results from these methods somewhat erratic, albeit at a lower degree when compared with *Method C*.

In summary, *Method B* and *Method C* can be considered to give reasonably accurate estimates of the swelling parameters either due to a test procedure representing field conditions and/or because of corrections applied to test data for minimizing discrepancies. In contrast, *Method A* still suffers from an inadequate capture of *in situ* conditions and a lack of corrections to apply to test data. This test has been traditionally used to estimate the swelling potential of intact field samples at the *in situ* degree of saturation. However, soil saturation depends on water availability that, in turn, is primarily governed by seasonal climatic conditions and depth in the profile. Consequently, the measured data correspond to a specific time and space only. Testing of air-dried or oven dried samples of expansive soils is quite difficult and does not represent the field setting because of shrinkage cracking and the associated fabric changes. Likewise, testing of compacted samples at various degrees of saturation is not representative of *in situ* conditions because of a complete change in soil microstructure. Further, the fabric of various compacted samples at wet and dry of optimum is known to be quite different. Therefore, supplementary tests can be conducted to correlate the swelling properties of intact field specimens of expansive clays with the degree of saturation.

2.2 Swell-Shrink Testing

Shrinkage occurs due to an increase in capillary stress causing a reduction in inter-particle distances thereby causing a decrease in the overall volume of soil. The determination of swell-shrink behaviour of expansive soils is based on shrinkage limit that is measured using the ASTM Standard Test Method for Shrinkage Factors of Soils by the Wax Method (D4943-08). Slurried, compacted, or undisturbed samples are desiccated from an initially saturated state without applying any surcharge pressure. The void ratio at different water contents is determined using the above method whereas the ASTM Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass (D2216-05) is used to measure the water content.

A typical swell-shrink test data depicted on a void ratio versus water content plot depicts an *S* - shaped curve (Tripathy et al. 2002). Theoretical lines for various degrees of saturation appear as straight lines with various slope angles emanating from the origin. This plot can be used to estimate both the degree of saturation and the void ratio of the soil for a given water content. Using these index properties as the initial conditions, the final void ratio can be obtained from the free swelling test as follows: (i) after the completion of swelling at the seating pressure to measure the *SP*; and (ii) after the cancellation of deformation due to swelling to measure *Ps*. This paper describes the above-mentioned procedure using Regina clay as a typical example of expansive soils.

3 MATERIALS AND METHODS

3.1 Sample Collection

The soil represented an extensive deposit of the glacio-lacustrine clay in the south of Regina. The samples were retrieved from a depth of 0.5 m to 1.0 m using the ASTM Standard Practice for Soil Investigation and Sampling by Auger Borings (D1452-2009) for disturbed samples and the ASTM Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes (D1587-08) for undisturbed samples. To preserve the field water content (*w*), the latter samples were wrapped with plastic sheets and painted with molten wax as per the ASTM Standard Practice for Preserving and Transporting Rock Core Samples (D5079-08). All of the samples were transported to the Geotechnical Testing Laboratory at the University of Regina and stored at 24 °C.

3.2 Geotechnical Index Properties

The geotechnical index properties were determined for preliminary soil assessment according to standard ASTM test methods as follows: (i) field water content (*w*) by the Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass (D2216-05); (ii) field dry unit weight (γ_d) by the Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method (D2937-04); (iii) specific gravity (*Gs*) by the Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer (D854-06); (iv) liquid limit

(w_l), plastic limit (w_p) and plasticity index (I_p) by the Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils (D4318-05); (v) shrinkage limit (w_s) by the Standard Test Method for Shrinkage Factors of Soils by the Wax Method (D4943-08); and (vi) clay size fraction (material finer than 0.002 mm) by the Standard Test Method for Particle-Size Analysis of Soils (D422-63(2007)).

3.3 Swell-Consolidation Test

The swell-consolidation test was conducted according to the ASTM Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils (D4546-08). A Dead Weight Consolidation Load Frame (S-449) containing a 64 mm internal diameter fixed ring consolidometer (S-455) manufactured by Durham Geo Slope Indicator Inc. were used. De-ionized water was used for specimen inundation and the sample was allowed to undergo swelling at a seating pressure of 1.5 kPa. After the completion of the swelling stage, the soil specimen was incrementally loaded to determine the consolidation properties of the expansive clay. This stage of the test was conducted in accordance with the ASTM Standard Test methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading (D2435-04). All of the volume changes were measured at regular intervals of time using a liner displacement transducer (E-311) manufactured by Durham Geo Slope Indicator Inc. The transducer was connected to a data logger and then to a portable computer for digital recording.

3.4 Swell-Shrink Test

The swell-shrink test was conducted in accordance with the ASTM Standard Test Method for Shrinkage Factors of Soils by the Wax Method (D4943-08). To obtain the void ratio, the volume of soil specimens was determined using the water displacement method. Each specimen was coated with molten microcrystalline wax (Gs = 0.91) and allowed to cool down at room temperature. After wax solidification, the sample was submerged in a 250 ml graduated cylinder that was filled with distilled water. The water height in the cylinder was carefully recorded using a Vernier calliper before and after sample submersion in the cylinder. A graduated syringe was used to remove the increased amount of water displaced by the sample thereby bringing the water height back to the initial reading. The displaced water mass was determined by weighing the graduated syringe before and after water filling and recording the difference. This quantity was readily converted to water volume representing the volume of wax-coated soil. The volume of soil was obtained from the difference of volume of the wax-coated sample and the volume of wax (mass / 0.91). A 7.4% correction was applied to account for the underestimation due to air entrapment at soil-wax interface in this method, as suggested by Prakash et al. (2008). The mass of the sample was also determined to estimate the bulk unit weight of the soil. Using basic phase relationships, the void ratio was determined from a knowledge of the bulk unit weight of the soil.

4 TEST RESULTS

4.1 Geotechnical Index Properties

Table 1 gives the geotechnical index properties of Regina clay. The field water content and the dry unit weight were determined to be 31.4% and 13.4 kN/m^2, respectively. From the measured specific gravity (Gs = 2.75) and degree of saturation (S = 82%), the field void ratio was calculated to be 1.05. The high liquid limit (w_l = 83%) and plastic limit (w_p = 30%) along with a low shrinkage limit (w_s = 15%) suggested the high water absorbing and retaining characteristics of the investigated clay. The clay size fraction (material finer than 0.002 mm) was found to be 66%. Using the plasticity index (I_p = 53%), the activity of the soil was found to be 0.8. These data indicate that Regina clay is a typical expansive soil (Azam, 2007). The authors' ongoing work has confirmed that the native soil is primary composed of expansive clay minerals such as smectite, hydrous mica, and chlorite.

Table 1 Geotechnical index properties of Regina clay

Property	Value
Field Water Content, w (%)	31.4
Field Dry Unit Weight, γ_d (kN/m^2)	13.4
Specific Gravity, G_s	2.75
Field Void Ratio, e*	1.05
Degree of Saturation, S (%)†	82
Liquid Limit, w_l (%)	83
Plastic Limit, w_p (%)	30
Plasticity Index, I_p (%)	53
Shrinkage Limit, w_s (%)	15
-0.002 mm, C (%)	66
Activity, $A = I_p / C$	0.8

* $e = (G_s/\gamma_d) - 1$
† $S = w\, G_s / e$

4.2 Swell-Consolidation Behaviour

Figure 1 gives the change in void ratio in relation to the applied pressure during the free swelling test. The test data was corrected for apparatus deformability. The sample increased from a void ratio of 1.05 to 1.08 at the seating pressure. The pre-consolidation pressure was determined to be 140 kPa whereas the overburden pressure was found to be about 10 kPa. This represents an over-consolidated clay deposit. According to Fredlund et al. (1980), clays in the study area have moderate compressibility owing to a long evaporation history and desiccation induced pre-consolidation. The compression index (Cc) was found to be 0.21 that fell within the range typical of inorganic silty clays, which is between 0.15 and 0.3 (Mitchell and Soga 2005). The swelling potential was computed using the following equation:

$$SP = \Delta e / (1 + e) \qquad [1]$$

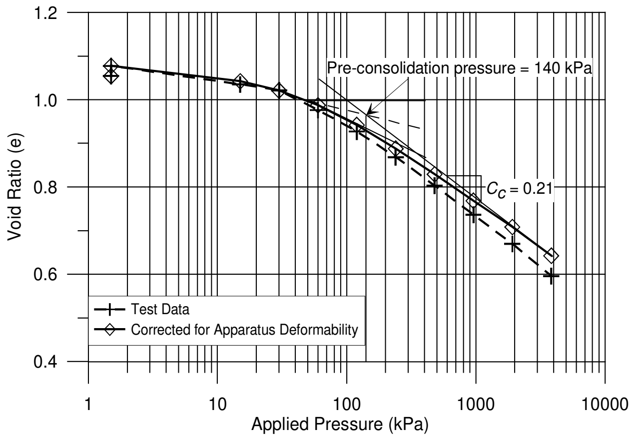

Figure 1: Void ratio versus applied pressure

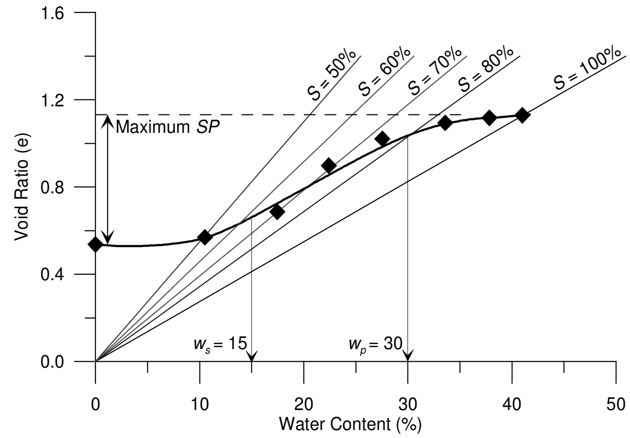

Figure 2: Void ratio versus water content

The *SP* was found to be quite low (1.5%). This value was confirmed by the actual volume change measurements over time data that showed the completion of swelling after five days. The low *SP* of the investigated sample is attributed to the field conditions (degree of saturation and fissuring) and unavoidable disturbances during sample collection, transportation and preparation (Nelson and Miller 1992). At the seating pressure of 1.5 kPa, the volume increase was governed by changes in soil suction. The high initial degree of saturation (82%) led to a low water intake by the investigated sample. Likewise, vertical fissuring (hair-line discontinuities) in the top 1.0 m depth of the soil resulted in consuming part of the swelling movement in the lateral direction. Consequently, the *SP* measured in the vertical direction recorded a lower value.

4.3 Swell-Shrink Path

Figure 2 shows the results from the swell-shrink test in the form of void ratio versus water content. Theoretical lines representing various degrees of saturation values were obtained from basic phase relationship and using Gs = 2.75. Several field samples were first wetted to achieve complete saturation from an initially unsaturated state. Thereafter, applying different suction values in a pressure plate extractor desaturated the specimens. The void ratio and water content of each sample was determined as described earlier. The data depicted herein indicate an *S* - shaped curve representing the progressive drying of the undisturbed soil. The curve comprises of an initial low structural shrinkage followed by a sharp decline during normal shrinkage and then by a low decrease during residual shrinkage (Haines 1923). During structural shrinkage, some of the larger and relatively stable voids are emptied such that the decrease in soil volume is less than the volume of water lost. Volume decrease in soil is equal to the volume of water lost during normal shrinkage thereby leading to a 45^{o} straight line parallel to the 100% saturation line. During residual shrinkage, air enters the pores close to the shrinkage limit and pulls the particles together due to suction. This leads to a further decrease in soil volume albeit lower than the volume of water lost.

Figure 2 further indicates that the transition between the various shrinkage stages correlate well with consistency limits as follows: (i) structural shrinkage from the *S* = 100% line to the plastic limit; (ii) normal shrinkage from the plastic limit to the shrinkage limit; and (iii) residual shrinkage from the shrinkage limit to complete desiccation. As described later in this paper, the central linear portion of the swell-shrink curve representing only 20% change in saturation (from *S* = 80% to *S* = 60%) is associated with bulk of the volume changes in the expansive Regina clay.

The swell-shrink path is reversible once the sample reaches an equilibrium condition. Tripathy et al. (2002) reported that this stage is usually attained after about four cycles in compacted soils. For natural Regina clay samples having undergone numerous swell-shrink cycles, an equilibrium condition may be considered to have reached. Further, the perpendicular distance between the normal shrinkage line and the saturation line decreases with increasing surcharge pressure. Given the low seating pressure (1.5 kPa) in the free swelling test and the no applied load in the swell-shrink test, results from the two tests can be used in conjunction. The swelling potential and swelling pressure can be determined for any initial degree of saturation using a graphical procedure.

5 ANALYSIS AND DISCUSSION

Figure 3 describes the estimation of swelling parameters for various degrees of saturation. Data from the swell-shrink test were superimposed on the swell-consolidation test. The void ratio and the degree of saturation values at critical stages in soil behaviour were obtained from Figure 2. These stages included wet, field, plastic limit, shrinkage limit, and desiccated conditions. Using the initial void ratio (from Figure 2) and the final void ratio (from Figure 1), the swelling potential for each of the critical condition was estimated by Equation [1]. Next, horizontal lines were drawn for each of the above-mentioned condition and the swelling pressure was graphically determined at the intersection with the compression curve.

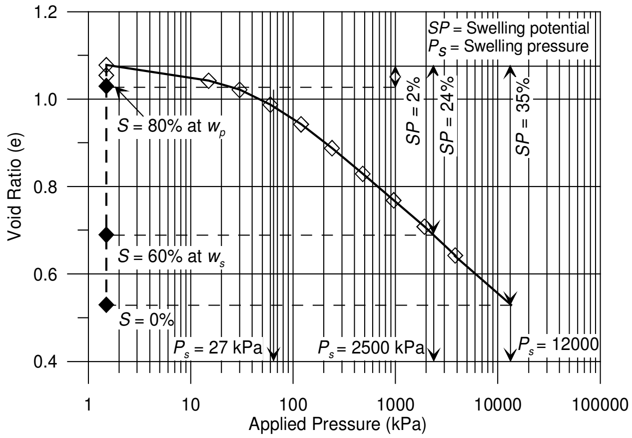

Figure 3: Estimation of swelling parameters for various degrees of saturation

Table 2 summarizes the variation in the estimated swelling parameters with changes in the degree of saturation. The data indicate that the variation in both *SP* and *Ps* is marginal up to the plastic limit (*S* = 80%). The swelling potential increased 12 times (from 2% to 24%) and the swelling pressure increased by two orders of magnitude (from 27 kPa to 2500 kPa) when the soil was dried from the plastic limit (*S* = 80%) to the shrinkage limit (*S* = 60%). This wide range in swelling parameters between the two consistency limits corresponds well with seasonal variations in the field water content profile and explains the engineering problems in Regina clay. The maximum *SP* and *Ps* values are obtained under completely dry conditions.

Table 2: Variation in swelling parameters with saturation

Condition	*S* (%)	*SP* (%)	P_s (kPa)
Wet	100	0	0
Field	82	1.5	3.5
Plastic limit	80	2	27
Shrinkage limit	60	24	2500
Desiccated	0	35	12000

6 SUMMARY AND CONCLUSIONS

The effect of saturation and desaturation on the volume change properties of Regina clay was investigated. Undisturbed field samples were used to determine the swell-consolidation and swell-shrink behaviour of the highly expansive native soil. The *in situ* conditions characterized the deposit to be over-consolidated with a pre-consolidation pressure equal to 140 kPa and an overburden pressure of about 10 kPa. Likewise, the compression index measured (*Cc*) was found to be 0.21 that fell within the range typical of inorganic silty clays. The swelling potential and the swelling pressure were found to be 1.5% and 3.5 kPa, respectively. The low measured values of the swelling parameters are attributed to a high saturation (82%), fissuring that consumed part of the swelling movement in the lateral direction, and disturbances during sample collection, transportation and preparation. The swell-shrink path during progressive drying of undisturbed samples of Regina clay followed an *S* - shaped curve. The curve comprised of an initial low structural shrinkage followed by a sharp decline during normal shrinkage and then by a low decrease during residual shrinkage. Using the results of the swell-consolidation test and the swell-shrink test, it was found that the variation in both *SP* and *Ps* is a maximum during the linear portion of the latter test. The swelling potential increased 12 times (from 2% to 24%) and the swelling pressure increased by two orders of magnitude (from 27 kPa to 2500 kPa) when the soil was dried from the plastic limit (*S* = 80%) to the shrinkage limit (*S* = 60%). This wide range in swelling parameters between the two consistency limits corresponds well with seasonal variations in the field water content profile and explains the engineering problems in Regina clay.

ACKNOWLEDGEMENTS

The authors acknowledge the financial support provided by the Natural Science and Engineering Research Council of Canada and the Mathematics of Information Technology and Complex Systems Inc., Canada. Sincere thanks to the Engineering Standards Branch of the Saskatchewan Ministry of Highways and Infrastructure for providing material support and to the University of Regina for providing laboratory space and computing facilities.

REFERENCES

Azam, S. 2007. Study on the swelling behavior of blended clay-sand soils. *Geotechnical and Geological Engineering.* 25:369-381

Azam, S. and Wilson, G.W. 2006. Volume change behavior of a fissured expansive clay containing anhydrous calcium sulfate. *Proceedings, 4th International Conference on Unsaturated Soils,* Carefree, Arizona, USA, 147. 1:906-915.

Christiansen, E.A. and Saure, E.K. 2002. Stratigraphy and structure of Pleistocene collapse in the Regina Low, Saskatchewan, Canada. *Canadian Journal of Earth Sciences*. 39:1411-1423.

Fredlund, D.G. and Rahardjo, H. 1993. *Soil Mechanics for Unsaturated Soils*. 1st ed., John Willy and Sons Inc., New York, USA.

Fredlund, D.G., Krahn, J., and Hasan, J.U. 1980. Variability of an expansive clay deposit. *Proceedings, 4th International Conference on Expansive Soils*. Denver, USA 1:322-338.

Haines, W.B. 1923. The volume changes associated with variations of water content in soil. *Journal of Agricultural Science.* 13:296-310.

Hu, Y. and Hubble, D.W. 2005. Failure conditions of asbestos cement water mains. *Canadian Geotechnical Journal*. 34:608-621.

Mitchell, J. K. and Soga, K. 2005. *Fundamentals of Soil Behavior.* 3rd ed., John Willy and Sons Inc., New York,

USA.
Nelson, J.D. and Miller, D.J. 1992. *Expansive Soils*. John Willy and Sons Inc., New York, USA.
Prakash, K., Shidharan, A., Baba, J.A. and Thejas, H.K. 2008. Determination of shrinkage limit of fine-grained soils by wax method. *Geotechnical Testing Journal*. 32:86-89.
Tripathy, S., Subba Rao, K.S. and Fredlund, D.G. 2002. Water content–void ratio swell-shrink paths of compacted expansive soils, *Canadian Geotechnical Journal.* 39: 938-959.
Wilson, G.W., Fredlund D.G. and Barbour, S.L. 1997. The effect of soil suction on evaporative fluxes from soil surfaces. *Canadian Geotechnical Journal*. 34:145-155.