Thirty-ninth Canadian Geotechnical Colloquium:
Unsaturated Soil Mechanics: Bridging the Gap Between
Research and Practice

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THIRTY-NINTH CANADIAN GEOTECHNICAL
COLLOQUIUM: UNSATURATED SOIL MECHANICS:
BRIDGING THE GAP BETWEEN RESEARCH
AND PRACTICE

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ABSTRACT
The majority of geoengineering applications occur in the unsaturated zone, which is in the near-surface
region forming the connection between meteorological phenomena above and saturated ground below.
The key characteristic of the unsaturated zone is that water is in tension or, put another way, pore water
pressure is negative. Moisture content, as well as most material properties, vary spatially and temporally
in the unsaturated zone and coupled processes are common. In geoengineering applications in the vadose
zone, unsaturated soils may be present during part or all of its design life. The question is how or when
to consider the unsaturated soils principles in an analysis or design. Although most geoengineering
applications have an unsaturated component, use of unsaturated soil mechanics in practice lingers
behind the prolific number of publications due uncertain benefit of accounting for unsaturated effects,
complexity, and conservativeness among other reasons. The focus of this colloquium is to continue
bridging the gap by illustrating unsaturated soils principles using application-driven examples in the
areas of capillarity as well as flow, strength, and deformation phenomena. As principles of unsaturated
soils become more understood and demand increases for incorporating climate change effects in design,
use of unsaturated soils principles in practice will continue to increase.

Keywords:
unsaturated soils, capillarity, seepage, strength, expansive soils
INTRODUCTION

The majority of geoengineering applications occur in the unsaturated, or vadose zone, which can be up to hundreds of meters in depth depending on the ground profile and climate. Unsaturated soils applications include foundations, excavations, and buried infrastructure (Figure 1a, Costa et al. 2003, Machmer 2012, Jung et al. 2016), infiltration and landslide triggering (Figure 1b, Iverson 2000; Blatz et al. 2004; Tohari et al. 2007; Cascini et al. 2010; Rahimi et al. 2010; Zhang et al. 2010; Robinson et al. 2016), compacted materials (Figure 1c, Sickmeier 2007), cover systems (Figure 1d, Wilson et al. 1994, O’Kane et al. 1998; Aubertin et al. 2009; Dobchuk et al. 2013; Huang et al. 2015; Knidiri et al. 2016), evaporation-aided mine waste consolidation (Bussiere 2007; Qi et al. 2017; Simms 2017), sampling and load tests (Figure 1e, Konrad 1990; Vanapalli and Mohamed 2007; Costa et al. 2003), as well as cryogenic suction processes and contaminant migration (Figure 1f, Konrad and Morgenstern 1980; Lenhard and Parker 1987; Azmatch et al. 2012). The unsaturated component of these examples can be broadly grouped into flow, strength, and deformation phenomena. The key characteristic of the unsaturated zone is that water is in tension or, alternatively, pore water pressure is negative. Soil moisture content, in turn, varies with negative pore pressure or suction. A consequence of variable moisture content is that soil properties, which are relatively constant in the saturated zone, vary spatially and temporally in the unsaturated zone. Unsaturated processes are often coupled, meaning they span across one or more of the flow-strength-deformation groups. For example, infiltration into a slope (Figure 1b) is a flow process, which is coupled with the unsaturated strength of the soil and can lead to landslide triggering. Unsaturated soils, found in the near-surface region, are continually interacting with the surrounding environment as they form the connection between weather systems above and saturated ground below. Thus, in each of the applications in Figure 1, the unsaturated component may be present during part or all of its design life.

The question is how or when to consider the unsaturated component in an analysis or design. Unsaturated soils researchers continue to move the state-of-the-art ahead with numerous unsaturated soils conferences every year. Beginning with the first issue (Hamilton 1963) and continuing with several Canadian Geotechnical Colloquium papers (Fredlund 1979; Barbour, 1998; Bussiere 2007; Simms 2017), the Canadian Geotechnical Journal is recognized as one of the key journals for disseminating unsaturated soils research. In practice, use of unsaturated soils principles develops incrementally with local expertise or in applications that have unsaturated effects at their core (e.g. cover systems or deep geological repositories for spent nuclear fuel). Thus, despite numerous geoengineering applications that include an unsaturated component, a gap exists between research and practice. The gap is due to a number of reasons that include the perceived complicated nature of unsaturated soil mechanics. Another perception that, in all cases, ignoring unsaturated effects provides additional safety in the performance of a geo-structure. In some cases, if the unsaturated components are ignored in design, additional safety factor may be available to the designer, which in my opinion is a reasonable approach. For example, ignoring the unsaturated strength component for a foundation above the water table will underestimate the strength and stiffness of the founding soil. However, in other cases, ignoring the unsaturated effects can lead to inefficiencies that affect the expected performance or lead to a false conclusion of a failure mechanism. For example, ignoring unsaturated soils effects in modelling transient triggering of a rainfall-induced landslide. In this paper, unsaturated soils principles are presented to illustrate
opportunities to broaden use of unsaturated soil mechanics. Unsaturated soils principles are illustrated through application-driven examples in the areas of capillarity as well as flow, strength, and deformation phenomena.

**CAPILLARITY**

Capillarity is a surface tension phenomenon whereby a pressure change occurs across an air-water interface (Lu and Likos 2004, Fredlund et al. 2012). The shape of the air-water interface and the corresponding change in pressure across the interface is described by the classic Young-Laplace Equation (Maxwell and Strutt 1911). Figure 2 includes three examples of capillarity: in zero gravity, in capillary tubes, and on flat surfaces. Canadian astronaut Chris Hadfield is shown in Figure 2a on-board the International Space Station with his hand coated in water. Water preferentially coats his hand as surface tension forces dominate in zero-gravity conditions. Water also forms menisci between his fingers and the direction of the surface tension forces are indicated on the figure. The angle at which surface tension forces act with respect to his fingers is determined by the contact angle shown in Figure 2c, which displays a drop of water on horizontal and sloped surfaces. On a horizontal surface, the interaction between air-water and the surface is apparent by the contact angle, $\alpha$. In this case, the contact angle is less than 90°, which indicates water is the wetting fluid and air is the non-wetting fluid. The solid surface prefers to be in contact with water rather than air. Contact angle is also variable, as illustrated in Figure 2c for the sloped surface. In the sloped case, there is a clear leading contact angle, $\alpha_1$, and trailing contact angle, $\alpha_2$. Contact angle is a property that is unique to each fluid-fluid-solid combination. In some cases, fluids can be wetting while in other cases fluids are wetting. For example, mercury forms beads on a surface indicate air is the wetting fluid for an air-mercury-solid combination.

A classic example of capillarity is given in Figure 2b, which is capillary tubes of different sizes placed into a container of water. Capillary forces act to draw water up into the capillary tubes to different heights. The height that water is drawn up the capillary tube is a function of the radius of the tube as well as the surface tension, contact angle, and density of water. Performing vertical force equilibrium by resolving the weight of the water and surface tension forces, and solving for the capillary rise height, $h$, gives the equation on Figure 2b. The smaller the radius of the tube the greater the height of capillary rise. An important implication of the curved meniscus at the top of each water column is the pressure difference between the air and water in the tube. The pressure difference between the air and water phases also leads to the definition of matric suction (air pressure-water pressure or $u_a-u_w$). The force arrows pointing upward and the concave shape of the meniscus indicate that air pressure is greater than the water pressure. This is a physical representation of the Young-Laplace Equation (Maxwell and Strutt 1911), which describes the pressure difference across curved surfaces.

In unsaturated ground, water is under the same surface tension forces as in the capillary tubes with surface tension forces pulling soil grains together. For a visual example, Bozkurt et al. (2017) recently measured surface tension forces between two glass spheres connected by a water bridge. Figure 3 is a hydrostatic ground profile showing pore pressure and degree of saturation versus elevation. The profile is at hydrostatic equilibrium and no flow is occurring. At the groundwater table, pore pressure is zero and varies hydrostatically above and below. Below the groundwater table, the ground is saturated and degree of saturation is 100%. Moving up from the phreatic surface, three distinct zones are the capillary zone, transition zone, and residual zone. In the capillary zone, degree of saturation remains at,
or close to, 100%. The soil-air-water representation shows occluded bubbles, which is reflective of a continuous water phase and discontinuous gas phase. From the capillary tube analogy, the height of the capillary zone is reflective of the radius of the largest pores in the soil. Moving upward into the transition zone, degree of saturation decreases with elevation. In this zone, the air phase becomes continuous across the element. Moving upward in the transition zone, degree of saturation continues to decrease until the residual zone is reached. The vertical distance from the groundwater table to the residual zone (indicated on Figure 3) is soil-type dependent and ranges from a few millimeters for gravels to hundreds of meters for clays. From the capillary tube analogy (Figure 2) this vertical distance is related to a continuous set of pores that have a radius less than that associated with the vertical distance above the groundwater table (equation on Figure 2b). Within the residual zone, Sr=0% is approached asymptotically. The air phase is continuous while the water phase has become discontinuous and water only coats the soil grains (since water is the wetting fluid and air is the non-wetting fluid).

**Storage Function**

The storage function is the fundamental relationship for unsaturated soils (Brooks and Corey 1964, van Genuchten 1980, Fredlund and Xing, 1994, Barbour 1998, Lu and Likos, 2004, Fredlund et al. 2012). Also termed the soil water characteristic curve or water retention curve amongst other names, the storage function is the relationship between water content and suction as shown schematically in Figure 4a. The storage function provides valuable and important information of the soil. Most often associated with soils, storage functions have also been reported for geosynthetics (Bouazza et al. 2006, Bathurst et al. 2007, 2009; Siemens and Bathurst 2010, Beddoe et al., 2011, Siemens et al. 2012) and are used to develop frozen soil relationships (Azmatch et al. 2012). The storage function is the first, and often only, function that is experimentally measured in unsaturated soils. From the storage function (van Genuchten 1980, Fredlund and Xing, 1994) many other unsaturated soil relationships can be estimated including unsaturated hydraulic conductivity (Mualem 1976, Fredlund et al. 1994, Burdine 1953, Leong and Rahardjo 1997), and unsaturated strength (Vanapalli et al. 1996).

The storage function in Figure 4a is essentially equivalent to the vertical profile of degree of saturation for the hydrostatic pore pressure profile in Figure 3. Analogous zones and soil-air-water representations appear on Figure 4a to reflect this similarity. Suction, or matric suction \( (u_a-u_w) \) to be more precise, is expressed as the difference between the air pressure \( (u_a) \) and water pressure \( (u_w) \). This is consistent with the pressure difference across the air-water interface explained by the Young-Laplace Equation and expressed physically as the curved surface within the capillary tubes in Figure 2c. For the typical case of atmospheric air pressure, suction is equivalent to the absolute value of the negative pore water pressure. Similar to the vertical profile (Figure 3), the magnitude of suction at which the residual water content is reached varies orders of magnitude. Because of this difference, the x-axis is adjusted to ensure the entire storage function can be viewed on a single graph. Coarse-grained soils have suction plotted on an arithmetic axis while fine-grained soils plotted on logarithmic axis. Similar to grain-size curves, logarithmic axes are used for convenience in order to be able to visualize the entire relationship.

An important aspect of the storage function is interpretation of material properties including the air entry value and unsaturated hydraulic conductivity function from experimental data. This is especially true for deformable soils, which will experience significant volumetric strains during drying and cause changes to the soil’s pore-size distribution (Romero and Simms 2008). Historically, storage
functions are plotted using a range of moisture content variables including degree of saturation, volumetric water content, or gravimetric water content. These are mostly due to unsaturated soil mechanics taking on aspects of hydrology or agriculture rather than a geo-perspective. AEV is defined as the suction at which air enters the soil. Soils that experience swelling-shrinkage with changes in suction are particularly susceptible to potential errors in AEV interpretation. Using gravimetric or volumetric water content can cause significant errors in determining AEV or unsaturated conductivity curve as changes in moisture content could reflect loss of water rather than air entering the soil. To properly interpret AEV and the unsaturated hydraulic conductivity curve, moisture content must be plotted in terms of degree of saturation (Fredlund et al. 2012). In deformable soils (Fredlund et al. 2011, Wijaya et al. 2015, Saleh-Mbemba et al. 2016), measurement of the shrinkage curve, in addition to the storage function is recommended to ensure proper interpretation of material parameters including AEV and the unsaturated hydraulic conductivity curve.

FLOW PHENOMENA

Unsaturated flow phenomena commonly occur in the near surface zone with interactions between above-ground weather systems. Examples include infiltration (Figure 1b), cover systems (Figure 1d), water drawn towards a freezing front due to cryogenic suction (Figure 1f), and multi-phase flow applications such as contaminant migration (Figure 1f). In each of these applications, the soil serves as a conveyant for flow while also undergoing changes in storage during transient events. The ability of soil to convey flow applies to movement of water in the near-surface area, water balance calculations, and ground-climate interactions.

The fundamental principle that controls flow phenomena in unsaturated ground is that hydraulic conductivity varies with suction as shown in Figure 4b. Saturated hydraulic conductivity alone is one of the most variable material properties fluctuating more than ten orders of magnitude. In unsaturated ground, the hydraulic conductivity of an individual soil also differs orders of magnitude due to changes in moisture content. The shape of the unsaturated conductivity function (Figure 4b) resembles the storage function (Figure 4a) with three distinct zones. In the capillary zone at suctions less than the air entry value, the saturated conductivity is essentially retained. In the transition zone, hydraulic conductivity decreases as pores desaturate and the water phase becomes more disconnected while the air phase becomes continuous. In the residual zone, the water phase becomes discontinuous and hydraulic conductivity decreases to essentially nil. The range of difference from saturated hydraulic conductivity to the minimum is often greater than 4 orders of magnitude and can be greater depending on the soil type.

That hydraulic conductivity varies orders of magnitude brings into question the validity of the flow laws for saturated ground. Researchers found that D’arcy’s law (D’Arcy 1856) and Bernoulli’s law apply, however, the process for solving the flow equation becomes more complicated. In saturated ground, the differential equation for flow reduces to the Laplace differential equation, which can be solved graphically. For unsaturated flow, the assumptions of isotropic and homogeneous are invalidated. Therefore, a numerical solution is often used, which may take the form of finite element or finite difference methods. With the advent of numerous groundwater flow software, steady-state unsaturated flow problems can be solved relatively quickly using numerical solutions.
Unsaturated flow phenomena impact all the applications in Figure 1 making its understanding vital for application. Ever more powerful computers and software make solving steady-state unsaturated flow problems possible with relative ease, however, care must be taken to appreciate the underlying assumptions and inputs. Unsaturated flow phenomena will be presented in this section in order to illustrate where appropriate assumptions can be made to obtain a suitable answer and where detailed soil information is necessary. The three applications are steady-state vertical flow, steady-state seepage in a dam, and infiltration in homogeneous and layered ground. The applications will illustrate the consequences of making poor assumptions in an analysis and the implications of those consequences.

Steady-State Flow Phenomena

Illustrative example #1: Determination of vertical flow direction in unsaturated ground

The first example illustrates the driving gradient for flow in unsaturated ground. Figure 5a plots two gravimetric moisture content profiles and the question for both profiles is whether flow is upward or downward. In both profiles (Figure 5a), gravimetric water content increases nonlinearly with depth and a constant value is obtained at 5m depth. An initial temptation is to interpret the direction of flow as following the moisture content gradient or that the both profiles show downward flow due to gravity. However, D’arcy’s law and Bernoulli’s law still hold in unsaturated ground and, therefore, water flows due an energy (i.e. total head) gradient. The equation for total head (Figure 5c) includes both elevation head and pressure head. Thus the determination of direction of flow requires knowledge of the storage function. Figure 5b plots the storage function in terms of gravimetric water content versus both matric suction and matric suction head.

In order to properly determine the direction of flow, Bernoulli’s equation (Figure 5c) is employed. A datum is set at 5m depth in order to assess the elevation head of each moisture content measurement. Then the storage function (Figure 5b), is used to calculate the pressure head for each gravimetric water content measurement. The moisture content measurements are plotted on the storage function (ignoring hysteresis) and the matric suction head for each measurement is indicated on Figure 5b. Finally, the total head profile can be calculated and plotted in Figure 5c. Despite seemingly comparable moisture content profiles, opposing flow gradients (one up and one down) are indicated by the total head gradients. This example illustrates the principle that unsaturated flow still follows Bernoulli’s equation, with flow from high total head to low total head.

Illustrative example #2: Unconfined flow through an earth dam

The second example illustrates where assuming a storage function is sufficient to obtain a realistic solution to a flow problem. Terzaghi (1943) used seepage through a dam as an example of where unsaturated principles can be applied. A similar example, given in Figure 6, is an earthen dam with 2H:1V slopes and 2m of freeboard to retain a 10m high water reservoir. A 9m wide toe drain is located on the downstream side of the dam. The solutions provided on Figure 6 include a graphical solution, a finite element method solution which makes an incorrect assumption, and a finite element method solution that employs a sensible unsaturated hydraulic conductivity assumption to obtain an appropriate solution.
The graphical solution is given in Figure 6a and a summary of the flow calculations is given on the figure. In order to draw the flownet, an assumption about the water table is made. The water table is assumed to be a no-flow boundary and the unsaturated component of flow is zero. This is indicated on Figure 6a as K=0 in the unsaturated zone and K=K_{sat} in the saturated zone. Flow of 144 L/d/m of dam was calculated using the flownet solution and a K_{sat}=10^{-6} m/s.

A second solution is shown in Figure 6b, which illustrates an incorrect, or ‘black box’, approach to finite element modelling. With some effort, flow calculations can be performed using software, however, proper care is required even for straightforward problems. In the case displayed in Figure 6b the saturated hydraulic conductivity has been applied to the entire domain, which is indicated on the figure as K=K_{sat} above and below the water table. This assumption could be made to be ‘conservative’ or by mistake. Comparing the flow calculations for Figure 6a and 6b indicate they are significantly different. Assuming that the entire domain has saturated conductivity increases the total flow to 227 L/d/m with 86 L/d/m being the unsaturated component. The flow below the water table (141 L/d/m) is comparable to the flow net (144 L/d/m). Therefore, the issue is with the unsaturated component, which is too high.

A third solution, which sensibly accounts for the unsaturated component of flow, is shown in Figure 6c. In this case, the unsaturated hydraulic conductivity curve was assumed using ‘stock’ storage function from within the software. The only input required is a saturated hydraulic conductivity and the soil type. The solution, shown in Figure 6c, shows the total head contours are similar to Figure 6b. However, the flow calculations are significantly different. Flow below the water table is principally equivalent (141 versus 142 L/d/m) while the unsaturated component is just 13 L/d/m compared with 86 L/d/m in the ‘black box’ solution.

The most representative solution for estimating unconfined flow within a homogeneous earthen dam can be made using a finite element model solution with an assumed unsaturated conductivity function (Figure 6c). The flownet solution ignored the unsaturated component of flow and underestimated flow by 10%. The ‘black box’ solution which applied saturated hydraulic conductivity to the entire domain overestimated flow by 40% and the unsaturated component of flow by 500%. For cases in which the saturated conductivity is known, assuming the unsaturated conductivity function is normally acceptable for steady-state problems. In these applications, the impact of the unsaturated conductivity function serves to decrease in conductivity above the water table. With some practice, steady-state problems can be solved relatively quickly. Sensitivity analyses can also be performed to get a sense of the impact of uncertainties in the geometry, material properties, and boundary conditions.

**Transient Flow Phenomena**

Transient unsaturated flow phenomena are inherently more complicated compared with steady-state and require a higher level of material property measurements. For example, in cover systems (Figure 1d, Wilson et al. 1994, O’Kane et al. 1998; Aubertin et al. 2009; Dobchuk et al. 2013; Huang et al. 2015; Knidiri et al. 2016) transient analysis allows for consideration of soil-weather interactions, which are coupled problems that include heat transfer, vapour flow, and liquid flow, to calculate evaporation from the soil surface (Bitelli et al. 2008; Lehman et al. 2008; Or et al. 2013). In this section, transient flow phenomena are illustrated using unsaturated transparent soil. An unsaturated transparent soil experiment is shown in Figure 7a, which is a series of digital photos of infiltration in a layered...
profile. Transparent soil is formed by matching the refractive indices of a soil and pore fluid and several combinations of transparent soil are found in the literature (Iskander et al. 2015). Saturated transparent soil allows for direct observation from within the soil mass of deformations, strains and soil-structure interaction rather than at the boundary of an experiment. Unsaturated transparent soil takes this concept in another direction (Peters et al. 2011). Observations in unsaturated experiments are often limited by the number of measurement devices that can be located within an experimental apparatus. A nominal number of discrete measurement points are normally included to measure pore pressure, suction, and/or moisture content. Unsaturated transparent soil, combined with digital image analysis, allows for measurement of degree of saturation to the millimeter resolution, which is orders of magnitude more data than can be obtained from discrete measurements.

To illustrate transient moisture migration in a heterogeneous profile, a transparent infiltration experiment is shown in Figure 7a with a fine layer between two coarse layers. A 10cm high pond was placed at the surface and the wetting front descends through the profile. Digital images were collected every 5s in this experiment and a select number are shown in the figure. The wetting front is clearly visible and is indicated by the black arrow. As the wetting front reaches the fine layer, ponding occurs, the soil above the fine layer changes to a darker colour associated with an increase in saturation. In the final photo, ponding is complete and the wetting front continues to descend through the fine layer. These images visually illustrate how moisture migration is affected by a relatively thin finer layer. The finer layer serves as a bottleneck to the system and causes saturation above as well as development of positive pore pressure.

Transparent soil allows for clear identification of the wetting front and measurement of the degree of saturation within the profile at high spatial and temporal resolution. Each digital image is processed by normalizing the image intensity between an image of a dry profile (white) and a saturated profile (black) to calculate the normalized intensity, \( I_N \) at the pixel scale. The calibration curve plotted in Figure 7b is used to convert normalized intensity to degree of saturation. On the plot are calibration points as well as a predicted curve. At \( I_N = 0 \), the soil is dry and \( I_N = 1 \), degree of saturation is 100%. At 90%, discrete air bubbles are visible while at lower saturations the soil visually lightens continually until its dry. Details on the digital image process can be found in Peters et al. (2011) and Sills et al. (2016).

Transparent soil used at RMC is formed from fused quartz and comes in two types, termed the coarse and fine gradations (Figure 7c). Both gradations are uniform sands with \( D_{10} \) equal to 0.75mm and 0.13mm respectively. The saturated conductivity of the two gradations differ by less than one order of magnitude.

**Impact of heterogeneities on infiltration**

To quantitatively illustrate the impact of minor heterogeneities on unsaturated processes, detailed results from two infiltration experiments are plotted in Figure 8 including degree of saturation profiles during the two experiments, wetting front location versus time, and pore pressure profiles from the beginning and end of the experiments. The only difference between the two experiments is the 120mm fine layer placed at 510mm depth below the surface. The \( D_{10} \) of the finer layer is just 0.62mm less than the coarse layers. The water table is located a 180mm elevation and a 10cm pond is placed on the surface at the beginning of the experiment. The uniform profile experiment shows the dry degree of saturation profile to start and the wetting front starting to descend at time=0s. As the wetting front...
descends the soil increases Sr behind the wetting front and the experiment is completed at 380s. The wetting front versus time (Figure 8c) shows the wetting front descending linearly from the surface down to the water table. The initial and final pore pressure results show the initial vertical suction profile which then becomes a linear decrease in pore pressure from the 10cm pond at the surface to 0kPa at the water table.

Infiltration in a layered profile shows the impact of a fine layer with $D_{10}$ differing just 0.62mm between the two gradations. Prior to the wetting front reaching the fine layer, the experimental results are identical in terms of the change in saturation and the wetting front descent rate. As the wetting front reaches the fine layer, the wetting front slows down significantly (Figure 8c). The wetting front continues to descend at a slower constant rate below the top of the fine layer until the water table is reached at the end of the test. The wetting front encountering the fine layer causes a ponding above the top of the fine layer and a hydrostatic pore pressure profile occurs (Figure 8d). The pore pressure dissipates across the fine layer and is essentially 0 kPa in the lower coarse layer.

Small heterogeneities affect the mobility of the wetting front, infiltration rate, and pore pressure response during infiltration. In the field, small differences in grain-size leading to a layered profile often occur naturally or during construction. Comparing the degree of saturation profiles at the end of infiltration (last profiles on Figure 8a and 8b) shows they are identical above the fine layer in the uniform and layered experiments. The wetting front also descends at the same rate until the sharp change when the fine layer is reached. Below the fine layer, the wetting front moves 4-times slower compared with the uniform profile. From a flow perspective, the fine layer serves as a bottleneck which reduces the transmissive capacity of the system. The pore pressure response at the end of the test is also significantly different. Pore pressure in the uniform coarse profile decreases from the 10cm pond at the surface to 0kPa at the water table (Figure 8d). In the layered experiment, a hydrostatic profile occurs above the fine layer with the initial suction dissipated below the fine layer. These experiments illustrate the impact of a thin and minor heterogeneity on infiltration. The fine gradation layer is just, 120mm thick and the $D_{10}$ is only 0.62mm different. However, the fine layer caused important differences to the pore pressure response and water balance calculations.

In the steady-state examples given above, estimations of the unsaturated functions were adequate to model the results. However, in transient applications, the unsaturated functions have a more significant role in the results. The two experiments plotted in Figure 8 were modelled using the finite element method (Siemens et al. 2014) and the results are plotted on Figure 8c and 8d. The storage functions and unsaturated conductivity functions were measured for coarse and fine gradations. The results match the model values throughout the experiments, however, the both the experiments and numerical simulations require experience to obtain reliable results. If infiltration rate is the most important consideration, another option is to use Green-Ampt model (Green and Ampt 1911), which gives the infiltration rate and wetting front location (Siemens et al. 2013). With proper calibration of the Green-Ampt parameters and knowledge of the conductivity of the transmissive zone these models are options depending on the requirements of the application.

Flow Phenomena Summary

Flow phenomena impact most geotechnical applications that include an unsaturated component. Applications in unsaturated ground (Figure 1) are inherently connected to the weather systems above
ground and therefore will interact hydraulically. Flow mechanisms in unsaturated ground are impacted by the hydraulic conductivity function which varies with suction. This section illustrated that the driving energy for flow follows Bernoulli’s law, showed how reasonable assumptions can be made for steady-state applications, and demonstrated the influence that small heterogeneities have on transient unsaturated flow.

**STRENGTH PHENOMENA**

Geotechnical applications that include an unsaturated strength component include foundations (Figure 1a), buried infrastructure (Figure 1a), landslides (Figure 1b), compacted construction materials (Figure 1c), and pile and plate load tests (Figure 1e). For these type of applications located in the vadose zone, a component of their performance is dependent on unsaturated soil principles. Within the unsaturated zone, strength is a function of suction. Commonly found evidence of unsaturated strength principles are sand castles constructed from a coarse-grained (also termed cohesionless) soils as depicted on Figure 9. If a coarse-grained soil is dry (Sr=0) or saturated (Sr=1), the maximum slope the soil is stable at is the angle of repose or angle of internal friction. However, if the soil is unsaturated it is stable at steeper slopes, which allows for creation of impressive structures such as the one shown in Figure 9. In an unsaturated state, surface tension forces act at the granular level to increase the normal stress between particles. In this framework, the friction angle of the soil remains constant while the apparent cohesion intercept is affected by suction. Figure 9 shows a Mohr-Coulomb strength envelope for saturated and unsaturated soils. For saturated and dry soils (Sr=1 and Sr=0), apparent cohesion is nil and the strength envelope is dependent only on the friction angle of the soil. For unsaturated soils, an apparent cohesion term elevates the strength envelope vertically. The amount of apparent cohesion is related to the storage function. The apparent cohesion intercept increases through the capillary zone and transition zone and decreases in the residual zone (Vanapalli et al. 1996, Lu and Likos 2004, Fredlund et al. 2012).

All soils in the vadose zone have an unsaturated component of strength associated with apparent cohesion. In practice, the unsaturated component is infrequently counted on in design due to its inherent transient nature. In this section, the focus is placed on examples in which ignoring unsaturated strength would have led to the wrong answer/development of an application. An interesting historical example of unsaturated soil strength as well as the effect of suction on plate load experiments are examined.

**Historical Example**

An historical application of unsaturated soil mechanics is illustrated in Figure 10a, which is a wall painting from a tomb in southeastern Egypt (Newberry 1895). The wall painting shows a statue on a sled being dragged over the sandy desert. The application of unsaturated soil mechanics is shown by the person adding water to the ground directly in front of the sled. Once thought to be a ceremonial act, he is actually using unsaturated principles to minimize interface friction between the sand and the bottom of the sled.

In order to test this hypothesis, Fall et al. (2014) performed a set of experiments to examine the influence of moisture content on interface friction. The experiment (plotted in Figure 10b as Load versus Displacement) consisted of placing a mass on top of a 11x7.5cm sled and pulling it across a sandy material prepared at specified water contents. The results (Figure 10b) show that an initial stiff response...
transitions to one where load remains constant with displacement. The volumetric moisture content of each experiment is noted on Figure 10b. The authors observed that when the sand was dry or at high moisture content the sled sank into the ground and plowed sand out of the way. Between these two extreme moisture contents, the sled sank less into the soil and less plowing occurred. Figure 10b indicates that when the soil is dry or saturated the effort to pull the sled across the sand is high compared with a minimal load for VWC=5%. At VWC=5%, for this particular sandy material, the strength of the material is maximized and the load to pull the sled is minimized.

The unsaturated soil principle illustrated in Figure 9 is displayed with this historical application. Figure 9 shows that apparent cohesion is a function of saturation. At Sr=100% or Sr=0%, strength reduces to the saturated value, which from an interface-friction perspective, led to plowing of the sled through the sand and an increase in pulling load. At VWC=5%, the soil strength was maximized, the sled slid on top of the sand and the pulling load was minimized. The Egyptians wanted to maximize speed and mobility for transport of statues and construction materials. Therefore, minimizing the interface friction between the sled and the ground allowed for more efficient transport. In a dry desert environment, the near-surface sand would be at a moisture content less than this optimal value. Thus the person on the front of the sled wetting up the surficial sand would temporarily increase the sand strength and minimize pulling effort. From Figure 10b a 30% reduction in pulling load was found for the optimized moisture content compared with both dry and wetter sand. Thus an 30% efficiency was gained by taking advantage of unsaturated principles.

Plate Load Tests

The effect of suction on unsaturated soil strength can be important in back analysis of failures as well as interpretation of insitu tests. Understanding the impact of suction on the unsaturated strength envelope can significantly impact interpretation of the failure mechanism. To understand the significance of suction on surface loads, Vanapalli and Mohamed (2007) performed a series of laboratory experiments where they applied a vertical pressure to the surface of a soil formed at specified moisture contents. The experiments were performed in a 90x90x75cm (width x length x height) reinforced box and the loading surface was 10x10cm. The soil is characterized as a sandy-gravel with the storage function plotted in Figure 11a. The soil has an AEV of 3 kPa (~30 cm) and its residual saturation is obtained at a suction of just 8 kPa (~80 cm). The friction angle of the soil is $\phi'=39^\circ$.

The question is, what impact could 6kPa suction have on the failure load. The four plate load experiments were performed at suctions of 0, 2, 4, and 6kPa with the results plotted in Figure 11b. The results show that the applied stress-settlement curves differ due to the suction. In each test a peak stress is obtained followed by some decrease for the lower suction tests. A stiffer response to the load is notable for the unsaturated experiments. A less stiff response and a lower peak stress is apparent for the test performed at 0 kPa suction (i.e. saturated). The peak value increases with increasing suction over the range of tests performed. At just 6kPa suction, the peak load capacity increases 7-fold compared with the saturated test.

Practical implications can be drawn from these tests. The only difference in the plate load experiments was the moisture content of the soil. Reducing the moisture content increases suction and, in turn, increases the apparent cohesion intercept of the failure envelope (Figure 9). The implication for interpreting plate load tests in the field is the need to measure moisture content in the zone of influence.
Any variation in moisture content will have a corresponding effect on the strength of the soil. In the experiments presented in Figure 11, the degree of saturation varied from 58-100% and had a 7-fold effect on the peak capacity of the load test.

**Strength Phenomena Summary**

Unsaturated soils experience an increase in apparent cohesion associated with capillary forces acting at the granular scale. However, owing to the inherent transient nature moisture migration within the unsaturated zone, apparent cohesion is also a transient. In analysis of failures, and insitu tests knowledge of unsaturated strength principles is recommended in order to correctly identify the governing mechanisms.

**DEFORMATION PHENOMENA**

Deformation phenomena associated with unsaturated soils often coincide with a change in the environmental conditions that alter the moisture regime. The key characteristic with respect to deformation phenomena is that unsaturated soils can experience volumetric strains due to fluctuations in both stress and moisture content (i.e. suction). Volumetric deformations associated with stress are perhaps more intuitive compared with deformations associated with moisture fluctuations. Classic examples of volumetric deformations associated with moisture changes are expansive soils and collapsible soils. Expansive soils respond to wetting and drying cycles with significant volumetric swelling and shrinkage as displayed on Figure 12 due to their clay mineralogy. Collapsible soils are recently deposited or altered soils, which have a high void ratio, high sensitivity, and low inter-particle bonding. In contrast to expansive soils, collapsible soils respond to wetting by subsiding or hydroconsolidating with significant settlement (Clemence and Finbarr, 1981; Basma and Erdil 1992; Rogers et al. 1994; Houston et al. 2001). Problems associated with collapsible soils include identifying and characterizing them during a site investigation, predicting the magnitude of wetting, quantitative predictions of collapse strains, and selecting design or mitigation alternatives (Houston et al. 2001). In this section, the main principles of unsaturated soil deformation phenomena associated with wetting-drying of expansive clay soils are illustrated and a conceptual framework for swelling potential is presented.

The most well-known soils that encounter deformations associated with moisture fluctuations are expansive soils. The most susceptible structures to expansive soil effects are lightly loaded infrastructure such as shallow foundations (Figure 12a, Domaschuk 1986). The American Society of Civil Engineers report that expansive soils cause owners more financial loss than the combined effect of hurricanes, earthquakes, and tornadoes. For reference Hurricane Sandy caused approximately $65.6 billion of damage (Wikipedia). In the basement schematic (Figure 12a), a combination of events induces both swelling and shrinkage, which leads to damaging differential displacements. Prior to construction often a vegetative layer is removed. The vegetative layer provides an elevated suction environment through the root systems, which uptake moisture from the ground. Once the vegetative layer is removed and the area covered, the soil beneath the house experiences a suction decrease as the house cuts off direct access to the ground surface. The soil swells and the house heaves in response to the decrease in suction. Adjacent to the house, a shrinkage scenario ensues as a large tree is planted and the root systems descend deep below the ground in the area next to the foundation. A growing tree continually requires more moisture
from the ground, which comes from its roots and leads to an increase in suction around the perimeter foundation. Owing to the increasing suction, the soil shrinks in response. The overall effect of the swelling under the center of the house and the shrinkage on the periphery is differential settlements and cracking of the basement. The damaging cost to owners is worsened as they often occur to personal dwellings and few options may be apparent, which leads to extreme actions. The photograph in Figure 12b shows an example of extreme actions with basement walls being removed from a house, which were then replaced along with the foundation (Siemens 2007).

To illustrate the magnitude of volumetric strains that can occur in expansive soils, digital images from a shrinkage experiment and a swelling experiment are given in Figure 12d and 12e. In the shrinkage experiment (Figure 12d), the test begins with an initially saturated sample. The specimen is initially located within a test ring to provide confinement, which is later removed once separation between the ring and the specimen occurs. The specimen dries in the low humidity laboratory environment and shrinks. After 8 days, the specimen shrinks 75% of its initial volume. The suction-volumetric strain path for the shrinkage experiment is illustrated in Figure 12c, which is a constitutive model for volumetric strains in unsaturated soil for both matric suction and net normal stress. Initially saturated, the specimen is at 0 kPa matric suction. As evaporation of the pore water occurs, matric suction increases and the soil shrinks in response. In the swelling experiment (Figure 12e), an unsaturated expansive soil is given access to water through filter paper strips in contact with a water reservoir (Lim and Siemens 2013). The initially unsaturated sample takes on water and swells in response. After 10 days, the soil swells to -59.4% volumetric strain. Figure 12c plots the volumetric strain-matric suction path, which is followed during the swelling experiment. The specimen is initially unsaturated and at elevated suction. As the specimen is given access to water, matric suction decreases and the sample swells in response.

Unifying Concept of Swelling Potential

Owing to the cost of damage to infrastructure constructed in expansive soils as well as their importance for use as environmental barriers (i.e. landfill liners), much research has sought to characterize the swelling potential of expansive soils. Swelling potential is measured in the laboratory (ASTM D4546) and also estimated from index properties (Komine and Ogata 2003; Prakash and Sridharan 2004; Cui et al. 2012; Ito and Azam 2013). Concepts for analysis and prediction of infrastructure constructed in expansive soil are based on moisture content changes, suction, or empirically based (Fredlund 1983; Briaud et al. 2003; Houston et al. 2011; Vanapalli and Lu 2012; Puppala et al. 2014). Owing to the number of continuing resources and publications associated with expansive soils, the research and practice communities are still in need of a practical methodology to assess the effect of swelling potential to infrastructure.

Swell Equilibrium Limit Concept

A unifying concept for swelling potential is illustrated in Figure 13a using the application of a retaining wall. The retaining wall is constructed on swelling ground and expansive soil is also used as backfill material. An extended infiltration event occurs to induce wetting conditions to the system. During wetting, swelling potential can be satisfied by a combination of volumetric expansion and swelling-induced stresses, which depend on the boundary conditions during wetting. Three cases are
illustrated in Figure 13a and their stress-volume paths plotted in Figure 13b in terms of specific volume 
\(V = e + 1\) versus mean stress \(p = (\sigma_1 + \sigma_2 + \sigma_3)/3\). The three cases are unconfined swelling, swelling under 
the foundation and swelling adjacent to the retaining wall. For the first case, in front of the retaining wall 
swelling occurs under unconfined conditions like the laboratory experiment in Figure 12e. In Figure 
13b, the stress-volume path begins at zero stress and moves vertically upward until the swelling 
potential is satisfied. The second case for swelling is under the foundation. In this case the expansive soil 
is subjected to an elevated stress level prior to the infiltration event, which serves to attenuate the 
magnitude of swelling compared with the unconfined case. In Figure 13b, the stress-volume path begins 
at an elevated stress level and moves vertically upward until the swelling potential is satisfied. The third 
case for swelling is adjacent to the retaining wall. In this case, volumetric confinement is provided by 
the retaining wall, which restricts expansion. Thus swelling potential is satisfied by swelling-induced 
stresses during wetting. From a stress-volume perspective, the soil state begins at a location associated 
with the stress level equal to the overburden stress and a volume equivalent to the end of construction. 
During wetting, the stress-volume path is sloped upward to the right as expansion is limited by the 
existence of the stiff retaining wall. At equilibrium, all three soil elements’ swelling potential have been 
satisfied and their end-points are on the Swelling Equilibrium Limit (SEL, Siemens and Blatz 2009).

**Measurement of Swelling Equilibrium Limit**

The SEL is unique for each soil and is measured experimentally with a series of swelling 
experiments such as those presented in Figure 13b. The experimental methodology (Siemens and Blatz 
2007, Lim and Siemens 2013) consists of bringing identically prepared unsaturated specimens to defined 
initial conditions and then wetting under idealized boundary conditions (Figure 13b). Results from three 
triaxial swelling experiments to define a SEL are plotted on Figure 13b. In each test, the specimen is 
brought to an initial stress condition and then given access to water. During the swelling phase, 
controlled boundary conditions define the stress-volume path followed during the test. In Figure 13b 
each test was brought to an initial stress state of 250 kPa and then the swelling phase was initiated. The 
three boundary conditions applied during the tests were constant mean stress (CMS, vertical path), 
constant stiffness (CS, upward sloping path), and constant volume (CV, horizontal path). The constant 
mean stress-volume path is equivalent to the soil swelling under the foundation in Figure 13a as the 
initial total stress is maintained during the swelling phase. When subjected to a constant mean stress, 
swelling potential is satisfied by volumetric expansion. The constant stiffness stress-volume path is an 
idealized representation of swelling adjacent to the retaining wall (Figure 13b). In the experiment, the 
soil swells against a linear-elastic spring boundary condition. The third swelling test is constant volume, 
in which swelling potential is satisfied entirely with swelling-induced stresses. The initial volume is 
maintained constant during swelling and a horizontal path is followed during swelling. At the end of 
each test, swelling potential is satisfied and the soil state lies on the SEL.

The unifying concept of swelling potential is illustrated for three soils in Figure 13c-e, which 
include their SELs and unsaturated isotropic compression lines. The SEL is defined by fitting a 
logarithmic curve to the end of swelling test points as shown in Figure 13c-e for bentonite-sand buffer, 
Bearpaw clay, and Lake Agassiz clay. The triaxial swelling experiments shown in Figure 13b have their 
end of swelling test states plotted in Figure 13c for reference. Also on SEL plots are shown the isotropic
compression line on which each of the swelling tests was initiated. The area between the SEL and the isotropic compression line is the swelling potential for each soil. The swelling potential is at a maximum at zero stress and decreases with increasing stress. The isotropic compression curve and SEL converge at higher stresses where the confining stress overcomes the swelling potential of the expansive soil.

Use of Swell Equilibrium Limit

Practical use of SEL concept relies on knowing the initial stress and volume states of a soil and the boundary conditions during swelling. For a house foundation or a retaining wall these are defined for constant mean stress areas and strategies are being developed for more complex cases (Lim 2014). Finite element modelling of foundations (Siemens and Blatz 2008) have shown the potential for analysis tools. The question arises whether SELs can only be defined in the laboratory or whether they can be estimated from readily measured material properties as has been used in other analyses (Komine and Ogata 2003; Prakash and Sridharan 2004; Cui et al. 2012; Ito and Azam 2013). Lim and Siemens (2016) showed that SELs can be estimated from liquid limit, free swell potential, plasticity index, and initial specific volume. The process involved fitting SEL equation parameters ($V_{SEL} = A + B \ln(p)$) to the material properties for the soils as plotted in Figure 14a-b for liquid limit. In the plots SEL parameter ‘A’ increases with increasing liquid limit while SEL parameter ‘B’ decreases. Parameter ‘A’ is the y-intercept of the SEL equation at zero mean stress while ‘B’ is related to the curvature. In both plots Figure 14a and Figure 14b linear fits are provide although, in the future, non-linear fits may be justified when the database size is increased. Using the predicted values from Figure 14a-b the SEL for Regina clay was estimated based on its liquid limit of 76% (Fredlund 1975). The resulting SEL is plotted alongside experimental data in Figure 14c and the results are comparable.

Deformation Phenomena Summary

Expansive soils display most distinctly that volumetric deformations in unsaturated soils are induced by changes in suction. Due to this susceptibility, expansive soils annually cause similar financial loss as the combined effect of hurricanes, earthquakes, and tornadoes (ASCE). The SEL provides a conceptual framework to analyze and predict swelling behaviour and the ability to predict SELs from index properties gives more encouragement to its use. Future research is directed at development of a practical analysis tool.

SUMMARY

Unsaturated soils are found throughout nature in the vadose zone and form the connection between above ground meteorological systems and saturated ground below. Geoengineering applications in the vadose zone include an unsaturated component during some or all of their design lives (Figure 1). Use of unsaturated principles is increasing in practice owing to knowledge accessibility, appreciation of unsaturated effects on soil behavior, and increase in computing capabilities to incorporate unsaturated relationships. This paper serves to make unsaturated soil mechanics more accessible and broaden their use in practice. More and more designs are asked to incorporate climate change effects to predict performance decades into the future. Incorporating climate change effects into design, will only serve to increase the use of unsaturated soil mechanics in practice.
ACKNOWLEDGEMENTS

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<td>605 ° degrees</td>
<td>629 p</td>
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<td>606 A, B SEL equation parameters</td>
<td>630 q</td>
<td>applied stress</td>
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<tr>
<td>607 AEV air entry value</td>
<td>631 r</td>
<td>capillary tube radius</td>
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<td>608 CMS constant mean stress</td>
<td>632 S</td>
<td>settlement</td>
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<tr>
<td>609 CS constant stiffness</td>
<td>633 SEL Swelling Equilibrium Limit</td>
<td></td>
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<tr>
<td>610 CV constant volume</td>
<td>634 Sr degree of saturation</td>
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<tr>
<td>611 c^&quot; apparent cohesion</td>
<td>635 T_s surface tension</td>
<td></td>
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<td>612 d day</td>
<td>636 u_a air pressure</td>
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<td>613 D_{10} 10% passing</td>
<td>637 u_a-u_w matric suction</td>
<td></td>
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<tr>
<td>614 e void ratio</td>
<td>638 u_w water pressure</td>
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<tr>
<td>615 FEM finite element model</td>
<td>639 V specific volume, e+1</td>
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<td>616 h height</td>
<td>640 W width</td>
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<td>617 h capillary rise</td>
<td>641 W gravimetric water content</td>
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<tr>
<td>618 h_e elevation head</td>
<td>642 (\alpha) contact angle</td>
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<td>619 h_p pressure head</td>
<td>643 (\gamma_w) unit weight of water</td>
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<tr>
<td>620 h_t total head</td>
<td>644 (\varepsilon_v) volumetric strain</td>
<td></td>
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<tr>
<td>621 I_N normalized pixel intensity</td>
<td>645 (\Delta) displacement</td>
<td></td>
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<tr>
<td>622 K hydraulic conductivity</td>
<td>646 \rho_w density of water</td>
<td></td>
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<td>623 K_{sat} saturated hydraulic conductivity</td>
<td>647 \phi^' friction angle</td>
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<td>624 L liter</td>
<td>648 \sigma_1, \sigma_2, \sigma_3 principal stresses</td>
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REFERENCES


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Figure 1. Geotechnical applications that include an unsaturated component: a) Foundations, tunnels and excavations, b) Infiltration and landslide triggering, c) Compacted materials for construction, d) Cover systems, e) Sampling, pile and plate load tests, and f) Cryogenic suction and contaminant migration.
Figure 2. Capillarity: a) Capillarity in zero gravity (image captured from Canada Space Agency video https://www.youtube.com/watch?v=o8TssbmY-GM accessed 29 November 2016), b) Capillary rise in tubes, and c) Contact angle on flat and sloped surfaces.
Figure 3. Hydrostatic ground profile highlighting soil type effect on capillary rise.
Figure 4. Typical a) Storage function and b) unsaturated hydraulic conductivity function.
Figure 5. Vertical flow illustrative example: a) Two gravimetric water content profiles, b) Storage function with gravimetric water content values added to interpret suction, and c) Total head profiles with flow direction indicated.
Figure 6. Steady-state flow comparison for: a) traditional flownet, b) FEM assuming saturated conductivity in unsaturated zone, and c) FEM with assumed unsaturated conductivity function.
Figure 7. Transparent soil for unsaturated applications showing: a) Series of photos from an infiltration test in a layered system (Siemens et al. 2014), b) Calibration curve for image analysis (after Sills et al. 2016, and c) Grain-size distributions for coarse and fine transparent soils (after Siemens et al. 2014).
Figure 8. Impact of heterogeneities on infiltration; a) Saturation profiles from open infiltration in coarse transparent soil, b) Saturation profiles from open infiltration in layered soil profile, c) Elevation of wetting front versus time, and d) Equilibrium pore pressure profile (after Siemens et al. 2014).
Figure 9. Unsaturated strength phenomena illustrating variation in apparent cohesion with degree of saturation.
Figure 10. a) Wall painting from tomb of Djehutihotep in southeastern Egypt (after Newberry 1895), and b) Interface friction experiments plotted as load versus displacement.
Figure 11. Effect of suction on plate load experiments in sand (after Vanapalli and Mohamed 2007) showing a) Storage function and b) Applied stress versus settlement.
Figure 12. Unsaturated deformation phenomena including a) Combined effects of heave and shrinkage on a residential structure (after Domaschuk 1986), b) Removal of basement walls from a residential structure affected by expansive soils, c) Constitutive model for volumetric strains due to changes in suction and net normal stress, d) Shrinkage experiment, and e) Free swell experiment (after Lim and Siemens 2013).
Figure 13. Swelling Equilibrium Limit results: a) Retaining wall constructed in expansive soil illustrating swelling conditions, b) Swelling paths and experiments for bentonite-sand-buffer, and SEL summary plots for c) Bentonite-sand-buffer, d) Bearpaw clay, and e) Lake Agassiz clay (after Lim and Siemens 2016).
Figure 14. Swelling Equilibrium Limit estimation for Regina clay: a) SEL parameter 'A' and b) SEL parameter 'B' versus liquid limit and c) SEL estimation comparison with laboratory data (after Lim and Siemens 2016).