



**AGS VICTORIA 2017 SYMPOSIUM**  
**Reactive clays and light structures**

Wednesday, 25 October 2017, 8:15am – 7:00pm

Rydges Hotel, 186 Exhibition Street, Melbourne



**AUSTRALIAN GEOMECHANICS SOCIETY**  
**VICTORIA CHAPTER**



# PREFACE

The Victorian chapter of the Australian Geomechanics Society invited academics and practitioners in the field of geotechnical and ground engineering to attend the 2017 Australian Geomechanics Society Victorian Symposium on 'Reactive clays and light structures' held on 25 October 2017.

The reactive soils of the Melbourne region form a large portion of its complex and variable geology. In particular, the basaltic volcanics situated to the north and west of Melbourne, which cover some 40% of the Melbourne region present numerous geotechnical challenges, particularly for lightly loaded structures. The geotechnical design and behaviour of lightly loaded structures on reactive soils is one aspect of geotechnical engineering where the public tend to have greater awareness, which is often not the case for the variety of soil and rock mechanics problems geotechnical engineers deal with. This is often borne out through their experience with their own residence, and rightly or wrongly, this contributes greatly to the public's perception of the geotechnical profession.

The 2017 Australian Geomechanics Society Victorian Symposium covered a variety of geotechnical challenges associated with reactive soils including residential slabs and footings, roads, pavements and other sensitive infrastructure that interact with reactive soils. The Symposium brought together practitioners from consulting, construction and academia to share and discuss their experiences on the topic of reactive soils and their related geotechnical applications.

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## Keynote Address

# Guidelines for the applications of effective stress principle to shear strength and volume change determination of unsaturated soils

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### ABSTRACT

The application of the effective stress principle to shear strength and volume change in unsaturated soils is presented. Step by step guidelines are provided with relevant procedures for material parameter determination. A relationship is derived between instability index used in AS2870 and the basic soil properties.

*Keywords:* unsaturated soils, reactive soils, shear strength, volume change

### 1 INTRODUCTION

Unsaturated soils account for almost 40 percent of earth's land surface. They are routinely encountered in civil engineering as compacted soils in roads, dams, embankments, highways and runways. Other examples of unsaturated soils include swelling clays, collapsing soils and residual soils. Some 80 percent of Australia's surface soils consist of unsaturated soils. Despite this the application of unsaturated soil mechanics in the practice of geotechnical engineering has been very limited. Possible reasons for this are: 1) inherent complexities associated with the behaviour of unsaturated soils, and 2) intractability and at times confusing nature of some of the constitutive models proposed for the behaviour of unsaturated soils.

The early models of unsaturated soils were primarily cast based on the notion of "two independent stress state variables", in which the net stress (i.e. total stress in excess of pore air pressure) and the matric suction (pore air pressure in excess of pore water pressure) were considered independent (e.g. Fredlund and Morgenstern, 1977; Alonso et al, 1990; Wheeler and Sivakumar, 1995; Cui and Delage, 1996). These models capture some fundamental aspects of the behaviour in unsaturated soils such as collapse, however fail to reproduce some other characteristic features such as plastic swelling at low confining pressures and the volumetric response of normally consolidated unsaturated soils subjected to drying (Alonso et al, 1995; Loret and Khalili, 2000; Wheeler et al, 2003; Gallipoli et al, 2003; Khalili et al, 2008). In addition, they introduce two sets of material parameters for the behaviour of unsaturated soils: one for the net stress and one for the matric suction, which may not be independent, leading to intractable stress-strain relationships. Furthermore, characterisation of two-stress state models requires considerable amount of testing in an unsaturated state, which is very time consuming, cost prohibitive, and in many cases impractical. They also introduce two different theoretical frameworks for saturated and unsaturated soils, rendering transfer of knowledge and experiences from saturated soils to unsaturated soils difficult if not impossible.

In recent years, there has been a major shift towards the effective stress based constitutive modelling of unsaturated soils (e.g. see Khalili and Khabbaz, 1998;

Lewis and Schrefler, 1998; Loret and Khalili, 2000; Khalili and Loret, 2001; Loret and Khalili, 2002; Sheng et al, 2003; Laloui et al, 2003; Gallipoli et al, 2003; Wheeler et al, 2003; Khalili et al, 2008; Masin and Khalili, 2012; Shahbodagh-khan et al, 2015). The advantage of using an effective stress based approach is that a complete characterisation of the shear strength and elastic deformation of the soil to changes in pore air pressure, pore water pressure and total stress are related to a single "effective" stress rather than two or three independent stress variables. This significantly simplifies the deformation model, and reduces the constitutive parameters. Adopting the effective stress approach, quantitative predictions of behaviour of unsaturated soils can be made based on exactly the same parameters used in saturated soils, except for two parameters (i.e. the air entry value and the suction hardening function) which can be determined in any soil physics laboratory.

Over the past two decades, major contributions have been made to the mechanics of unsaturated soils using the effective stress approach. They include: i) coupled flow deformation models (Khalili et al, 2000; Loret and Khalili, 2000; Khalili et al, 2008); ii) effective stress based shear strength determination models (Khalili and Khabbaz, 1998; Khalili et al, 2004); iii) elasto-plasticity (Kohgo et al, 1993; Bolzon et al, 1996; Loret and Khalili, 2000; Gallipoli et al, 2003; Wheeler et al, 2003; Khalili et al, 2008); iv) fully coupled thermo-hydro-mechanical models (Khalili and Loret, 2001; Laloui et al, 2003; Uchaipichat and Khalili, 2009; Masin and Khalili 2012); v) cavity expansion in unsaturated soils (Russell and Khalili, 2006; Pournaghiazar et al, 2013); and vi) cyclic analysis of unsaturated soils (Russell and Khalili, 2006; Khalili et al 2008; Masin and Khalili, 2008; Shahbodagh-khan et al, 2015), and vii) dynamic analysis of unsaturated soil (Wong et al, 2014; Hasan and Wheeler, 2016).

In this work, step by step guidelines are presented for the application of the effective stress principle to shear strength and volume change determination of unsaturated soils for practical purposes. Collapse upon wetting is discussed and equivalencies are established between the basic mechanical soil properties and the instability index used in AS2870 for expansive soils.

## 2 BACKGROUND

### 2.1 Effective stress

The effective stress principle is one of the most fundamental concepts in the science of soil engineering. It converts a multi-phase multi-porous medium to a mechanically equivalent single phase single stress continuum, and allows application of principles of solid mechanics to soil engineering problems (Khalili et al, 2004). It captures the (average) stress of the soil solid skeleton, and serves as a platform for coupling of fluid and deformation fields within the soil (Khalili et al, 2008).

For saturated soils, the effective stress is expressed as

$$\sigma' = \sigma - u_w \quad (1)$$

in which  $\sigma'$  is the effective stress,  $\sigma$  is the total stress and  $u_w$  is the pore water pressure. For unsaturated soils the effective stress is defined as (Bishop, 1959)

$$\sigma' = (\sigma - u_a) + \chi(u_a - u_w) \quad (2)$$

where  $u_a$  is the pore air pressure, and  $\chi$  is the effective stress parameter, ranging from one for saturated soils to zero for dry soils. Equation (2) can also be cast as

$$\sigma' = \sigma_{net} + \chi s \quad (3)$$

where  $\sigma_{net} = \sigma - u_a$  is the net stress, and  $s = u_a - u_w$  is the matric suction.

### 2.2 Effective stress parameter

The effective stress parameter,  $\chi$ , describes the contribution of suction to the effective stress of the soil. It is strongly dependent on the soil structure, and its correct determination is essential for a successful application of the effective stress principle to soil engineering problems. Earlier definitions of the effective stress parameter assumed 1:1 correlation with the degree of saturation,  $S$  (Bishop, 1959; Bishop and Blight, 1963). They provided a geometrical interpretation of the effective stress parameter, however no unique relationship could be found between the degree of saturation and the effective stress parameter (Bishop and Donald, 1961). Based on thermodynamic considerations, Laloui *et al* (2003) stated that  $\chi$  should be expressed in terms of the aerial fractions of the constituents rather than the volumetric fractions. They further argued that  $\chi$  is related to, but not equal to, the degree of saturation and is a function of porosity and the pore air and pore water pressures. Similarly, Hassanizadeh and Gray (1990), Houlsby (1997), Muraleetharan and Wei (1999) showed that ignoring the work of air-water interface the effective stress parameter may be taken as the degree of saturation. In recent years, several authors have advocated the use of the degree of saturation as the effective stress parameter. However, the overwhelming experimental evidence, gathered since 1960's, is against the use of degree of saturation as the effective stress parameter (Figure 1).

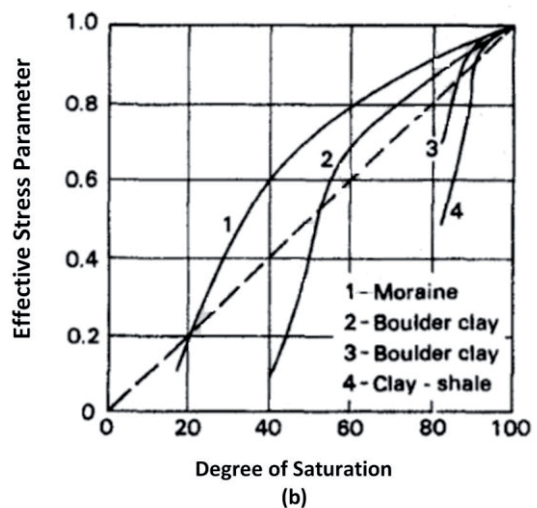
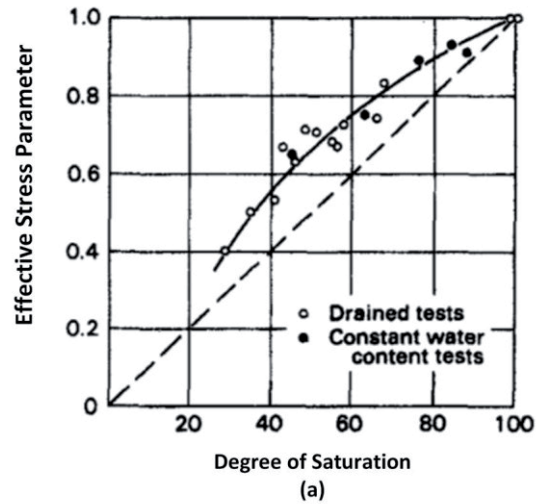


Figure 1. Relationship between the effective stress parameter and the degree of saturation: (a) for a silt; (b) for compacted soils (Bishop and Donald, 1961).

Analysing the shear strength data from a range of soil types, Khalili and Khabbaz (1998) obtained a unique relationship for the effective stress parameter  $\chi$  in terms of the suction ratio,  $s/s_e$  as (Figure 2)

$$\chi = \left(\frac{s}{s_e}\right)^{-0.55} \quad \text{for} \quad s \geq s_e \quad (4a)$$

$$\chi = 1 \quad \text{for} \quad s \leq s_e \quad (4b)$$

in which  $s_e$  is the suction value marking the transition between saturated and unsaturated states. The theoretical basis for (4) can be found in Jiang *et al* (2017) from a thermodynamics point of view, and Khalili and Peric (2018) using the energetics of interfaces.

In equation (4),  $s_e = s_{ex}$  for the main wetting path, and  $s_e = s_{ae}$  for the main drying path, in which  $s_{ex}$  is the air expulsion value and  $s_{ae}$  is the air entry value. Along the scanning curve, the effective stress parameter,  $\chi$ , can be determined using the relationship proposed by Khalili and Zargarbashi (2010). For practical purposes, and as an approximation, the value of the effective stress parameter along the scanning curve may be taken as constant, and equal to its value at the point of suction reversal from drying to wetting and vice versa.

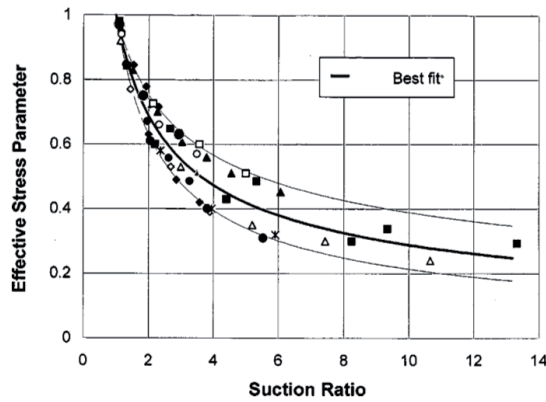


Figure 2. Effective stress parameter versus suction ratio (Khalili and Khabbaz, 1998)

Now, following Brooks and Corey (1964), the relationship between the degree of saturation and suction may be expressed as

$$S_e = \left(\frac{s}{s_e}\right)^{-\lambda_p} \quad (5)$$

in which  $S_e = \frac{s-s_r}{1-s_r}$  is the effective degree of saturation,  $s_r$  is the residual degree of saturation and  $\lambda_p$  is the pore size distribution index. Substituting (5) into (4) we have

$$\chi = S_e^\alpha \quad \text{with} \quad \alpha = \frac{0.55}{\lambda_p} \quad (6)$$

Equations (4) and (6) are equivalent and can be used inter-changeably depending on the availability of relevant model parameters. Expressions similar to (6) have also been proposed by Vanapalli and Fredlund (2000) and Alonso et al (2010). Vanapalli and Fredlund related the value of  $\alpha$  to plasticity index and Alonso et al determined  $\alpha$  based on micro mechanical considerations of pore size distribution in the soil. Both approaches require  $\alpha \geq 1$  which may not be applicable to granular material. (For sands and gravels  $\alpha$  is frequently less than 0.5.) In contrast,  $\alpha = \frac{0.55}{\lambda_p}$  captures the full range of admissible values of  $\alpha$  experienced in real soils through incorporation of  $\lambda_p$  (Khalili and Peric, 2018).

### 3 SHEAR STRENGTH

Based on the effective stress principle the shear strength of unsaturated soil can be determined using exactly the same expression that is used for saturated soils

$$\tau = c' + \sigma' \tan \phi' \quad (7)$$

in which  $c'$  is the effective cohesion intercept, and  $\phi'$  is the effective friction angle.

Accordingly, the following steps may be adopted for calculating the shear strength of unsaturated soils.

1) Determine the effective stress shear strength parameters of the soil,  $c'$  and  $\phi'$ . This can be achieved using standard testing techniques perfected in saturated soil mechanics such as the conventional triaxial consolidated drained compression test or the

consolidated undrained compression test with pore pressure measurements. No testing of the soils in an unsaturated state will be necessary. In the effective stress approach, the critical state friction angle of the soil,  $\phi'_{cv}$ , is unique for both saturated and unsaturated soils (Khalili et al, 2004). The peak friction angle,  $\phi'_{peak}$ , however, may show some dependency to suction. Nevertheless, the experimental evidence suggests that  $\phi'_{peak}$  increases only slightly with increasing suction (less than 6 percent) (Vanapalli et al., 1996), and that for all practical purposes, the value  $\phi'_{peak}$  may be taken as that for saturated soils.

2) Determine the effective normal stress acting on the plane of failure. This can be obtained from equation (2). The normal net stress,  $\sigma_{net}$ , acting on the plane of failure is quantified in the same manner as in saturated soil mechanics. Determination of suction at the point of failure will depend on the prevailing drainage condition during failure. For slow modes of failure, the initial suction of the soil may be assumed to also persist at failure. This is similar to the drained condition in saturated soils, in which the initial ambient pore pressure of the soil is assumed to be applicable to failure. For rapid modes of failure (i.e. constant water content condition), suction at failure will be smaller or larger than the initial suction of the soil depending on the contractive or dilative nature of soil response during shearing, respectively. However, the change in the initial suction towards failure will be relatively minor if the degree of saturation of the soil is less than 70%, due to the internal drainage provided by unsaturation for the dissipation of excess pore pressure. In such cases, the suction at failure of the soil can be taken as equal to the initial suction of the soil. For soils, with a degree of saturation greater than 70%, a fully coupled hydro-mechanical analysis of the soil (Khalili et al, 2008) must be performed in order to estimate suction at failure for constant water content modes of failure.

*Remark: In saturated soils, the undrained shear strength is unique for a given void ratio, as the volume of the sample remains constant during shearing. As such, undrained shear strength of saturated soils is almost independent of the stress path. Unsaturated soils however experience volume change during undrained shearing, and hence their constant water content shear strength is a function of the stress path. Therefore, a unique undrained/constant water content shear strength applicable to all stress paths cannot be established for unsaturated soil.*

#### Initial Suction

The initial suction of the soil can be determined using a range of tensiometer based instrumentation techniques in the field, or filter paper/axis translation techniques in the laboratory. The possible range of initial suctions experienced by the soil may also be established from the soil water retention curve (SWRC) and the in situ water content or degree of saturation of the soil.

#### Effective Stress Parameter

The effective stress parameter,  $\chi$ , can be determined using either of equations (4) or (6). For soils in which the air entry value is determined accurately preference may be given to the use of equation (4). In this case, a complete characterisation of the soil water retention curve (SWRC) may not be necessary, which can lead to savings in the scope of water retention testing. For

soils in which the air entry is small (e.g. sands and gravels) a minor error in the determination of  $s_e$  can lead to a large error in the calculated value of the effective stress parameter. For these soils, the use of equation (6) may be preferred. Both  $s_e$  and  $\lambda_p$  can be obtained from the SWRC determined at the void ratio of interest.

In cases where SWRC is not available, degree of saturation may be used as the effective stress parameter, as a crude approximation. For sands and gravels the degree of saturation will underestimate the value of the effective stress, whereas it will overestimate the effective stress of the clayey soils. The overestimation for clays soils can be quite significant at lower degrees of saturation (Figure 1b).

**Air Entry/ Air Expulsion Value,  $s_e$**

For over-consolidated soils in which the  $s_e$  is less than the pre-consolidation pressure, the suction associated with the first break point in the SWRC can be taken as the  $s_e$  (Pasha et al., 2016). The break point is determined as the intersection between the two straight line portions of the SWRC (Figure 3).

Remark:  $s_e$  is often determined erroneously in the literature as the point of intersect between the horizontal line drawn from the initial state of the soil and the straight line representing the unsaturated portion of the SWRC. This can result in marked underestimation of the  $s_e$  given the log scale of the graph (Figure 3).

For normally consolidated or slightly over-consolidated soils, in order to determine  $s_e$ , particular attention must be given to the volume change of the sample during the SWRC test. If the volume change of the soil is measured during the test, then the water content data from the test must be converted to the degree of saturation and the soil water retention curve be presented in terms of degree of saturation (SWRC-S). In this case, the first break point in the SWRC-S will correspond to the point of air entry and the same procedure described for over-consolidated soils can also be applied to SWRC-S of normally consolidated and slightly over-consolidated soil to obtain  $s_e$ .

If the volume change data is not available or that the test data is only presented in terms of water content, the SWRC should be plotted both in the log-log as well as semi-log scales in the same graph.  $s_e$  can then be determined as the suction value where the water

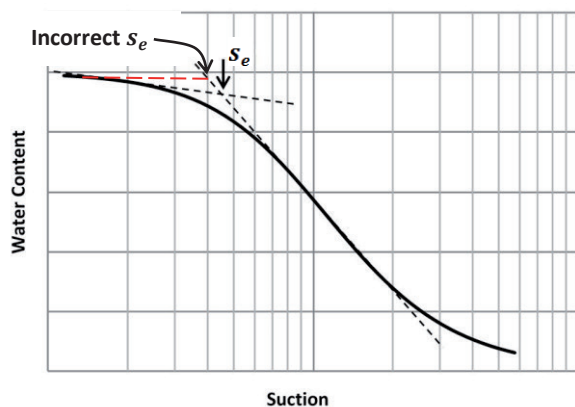


Figure 3. Correct determination of  $s_e$

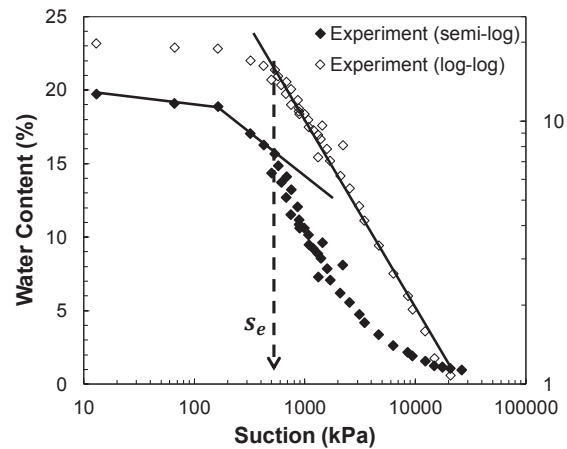


Figure 4. Determination of  $s_e$  for a slightly over-consolidated soil (Pasha et al, 2016)

retention response of the soil deviates from linear behaviour in both plots as shown in Figure 4 (Pasha et al., 2016).

**Void Ratio Dependency of  $s_e$**

For the value of  $s_e$  to be of relevance, it must be determined at the void ratio of interest in the field. If  $s_e$  is determined from a SWRC obtained at a void ratio different from that expected in the field, it must be corrected for the difference in the two void ratios,  $de$ . Following Pasha et al (2017), relationship (8) may be used to calculate the change in  $s_e$  for a change in void ratio

$$ds_e = -\frac{0.55s_e}{e\lambda_p} de \tag{8}$$

in which  $e$  is the current void ratio. Typical variation of air entry value with void ratio for a silty soil is given in Figure 5.

**Pore Size Distribution Index,  $\lambda_p$**

The value of  $\lambda_p$  used in equation (6) can be obtained from the slope of the unsaturated portion of SWRC in the log-log scale.

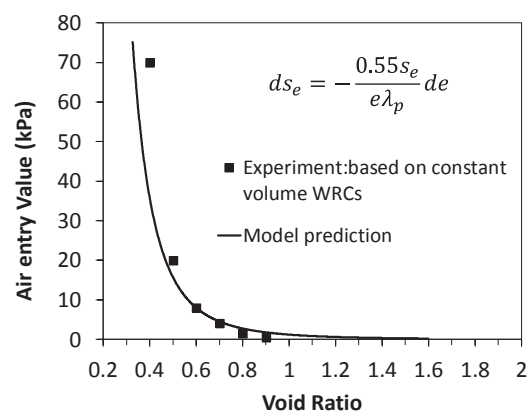


Figure 5. Variation of air entry value with void ratio

#### 4 VOLUME CHANGE

To apply the effective stress principle to volume change analysis, consideration must be given to the following aspects (Khalili et al 2004)

1) Suction has a dual effect on the behaviour of unsaturated soils. It increases the effective stress, and shifts the pre-consolidation pressure of the soil. The greater the suction, the greater the pre-consolidation pressure. Many of the complications in the behaviour of unsaturated soil are due to this increase in the pre-consolidation pressure and stiffening of the soil response with suction. Collapse upon wetting, for example, is a direct consequence of a reduction in the pre-consolidation pressure due to wetting induced reduction in suction.

2) Suction increases (slightly) the slope of the normal compressor line,  $c_c$ , but has no influence on the slope of unloading-reloading line,  $c_r$  (Figure 6).

3) In collapsible unsaturated soils, the increase in the pre-consolidation pressure is greater than the increase in the effective stress due to an increase in suction. A direct consequence of this is that upon desaturation, the collapsible soil response enters the elastic region and that the elastic coefficient of compressibility must be used in predicting volume change in such soils subject to drying at constant net stress. This fundamental aspect in the behaviour of unsaturated soils has been neglected in the previous analyses of volume change in unsaturated soils, leading to erroneous conclusions as to the applicability and/or uniqueness of the effective stress parameter in unsaturated soils.

4) In expansive unsaturated soils, the increase in the pre-consolidation pressure with suction is either absent or is smaller than the increase in the effective stress. Expansive soils do not experience collapse and the volume change during wetting is entirely controlled by the slope of the unloading-reloading curve,  $c_r$ .

Within this context, to determine the volume change of unsaturated soils the following steps may be adopted.

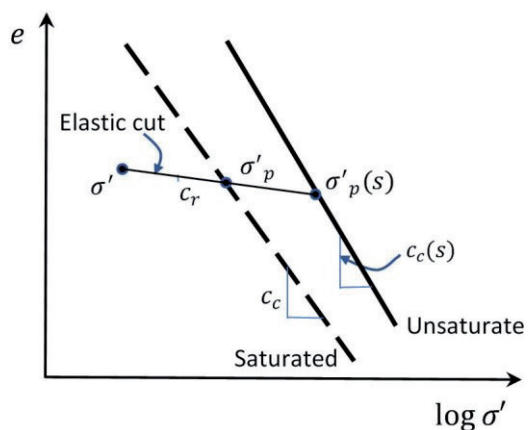


Figure 6. Normal compression lines for saturated and unsaturated states and determination of pre-consolidation pressure through elastic cut

1) Determine the compression indices  $c_c$  and  $c_r$  for the soil by conducting constant suction oedometer test on a representative sample of soil at the target suction,  $s$ .

2) Determine the initial effective stress,  $\sigma'_1$ , and the final effective stress,  $\sigma'_2$ , of the soil. The change in the effective stress may be due to a change in suction, externally applied load or a combination of both. Similar to shear strength calculations, the effective stress is calculated using equation (2); and the effective stress parameter is determined using equations (4) and (6), as appropriate.

3) Determine the pre-consolidation pressure,  $\sigma'_p(s)$ , at target suction,  $s$ .  $\sigma'_p(s)$  is defined as the effective stress corresponding to the point of intersect between an elastic cut passing through  $\sigma'_1$  and the normal compression line determined at suction,  $s$  (Figure 6).

4) Depending on the interplay between  $\sigma'_1$ ,  $\sigma'_2$  and  $\sigma'_p(s)$ , calculate the surface movement,  $y_s$ , according to the following scenarios. The thickness of the compressible soil layer is taken  $H$ .

##### Loading $\sigma'_1 < \sigma'_2$

$$y_s = \frac{H}{1+e} c_r \log \frac{\sigma'_2}{\sigma'_1} \quad \text{for} \quad \sigma'_1, \sigma'_2 \leq \sigma'_p(s) \quad (9a)$$

$$y_s = \frac{H}{1+e} [c_r \log \frac{\sigma'_p(s)}{\sigma'_1} + c_c \log \frac{\sigma'_2}{\sigma'_p(s)}] \quad \text{for} \quad \sigma'_1 < \sigma'_p(s) < \sigma'_2 \quad (9b)$$

$$y_s = \frac{H}{1+e} c_c \log \frac{\sigma'_2}{\sigma'_1} \quad \text{for} \quad \sigma'_1 = \sigma'_p(s) < \sigma'_2 \quad (9c)$$

##### Unloading $\sigma'_1 > \sigma'_2$

$$y_s = \frac{H}{1+e} c_r \log \frac{\sigma'_2}{\sigma'_1} \quad \text{for} \quad \sigma'_2 \leq \sigma'_p(s) \quad (9d)$$

$$y_s = \frac{H}{1+e} [c_r \log \frac{\sigma'_p(s)}{\sigma'_1} + c_c \log \frac{\sigma'_2}{\sigma'_p(s)}] \quad \text{for} \quad \sigma'_p(s) < \sigma'_2 \quad (9e)$$

Equations (9a-9d) are exactly the same as those used in saturated soil mechanics except for (9e) which is only applicable to unsaturated soils. This equation corresponds to wetting induced (unloading) collapse in unsaturated soils. To explain this phenomenon, consider the schematic shown in Figure 7. Initially, the soil is located at point A on the normal compression line ( $A'-A''$ ) corresponding to the initial suction  $s_0$  and effective stress  $\sigma'_1$ . Then, the soil is subjected to wetting by reducing suction from  $s_0$  to  $s$ . The stress state moves from point A to point B, and the effective stress reduces from  $\sigma'_1$  to  $\sigma'_2$ . At the same time, the normal consolidation line of the soil retreats from  $A'-A''$  to  $C'-C''$ , rendering the stress state at point B outside the state boundary and hence unattainable. To reach a new admissible stress state, the soil must therefore undergo volumetric contraction (i.e. at constant effective stress), and collapse onto the new state boundary at point C. The extent of collapse can be calculated using equation (9e) by first taking the stress state from  $\sigma'_1$  to  $\sigma'_p(s)$  along  $c_r$  line, and then from  $\sigma'_p(s)$  to  $\sigma'_2$  along  $c_c(s)$  line (Figure 7).

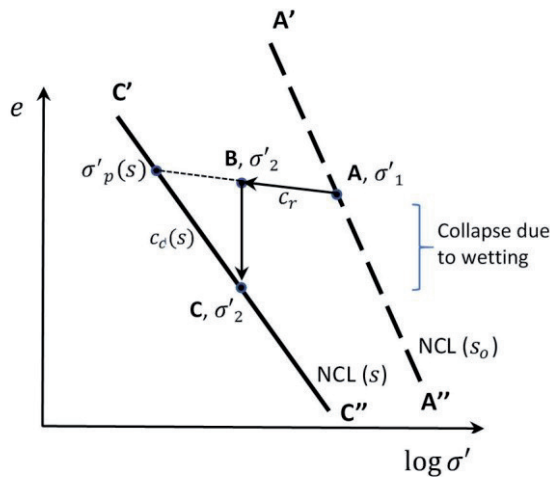


Figure 7. Schematic representation of collapse upon wetting in unsaturated soils

## 5 VOLUME CHANGE – REACTIVE SOILS

The volume change in reactive soils follows the exactly the same principles as those in non-reactive soils, except that reactive soils: 1) do not experience increase in the reconsolidation pressure with suction, and 2) can experience volumetric change due to a change in their pore water chemistry or osmotic suction. Assuming that the change in the osmotic suction is negligible, which is applicable to most practical problems subjected to moderate variations in soil moisture - an implicit assumption in AS2870 - the volume change of reactive soils due to wetting and drying can be calculated using equations (9a) to (9d), respectively.

In AS2870, the site surface movement,  $y_s$ , is determined based on an assumed change in the soil suction,  $\Delta u$ , measured in  $pF = \log s (kPa) + 1$ , an assumed depth of active zone,  $H_s$ , (depending on the location of site) and an empirically established instability index,  $I_{pt}$

$$y_s = \int_0^{H_s} I_{pt} \Delta u dh \quad (10)$$

On the other hand, according to the effective stress principle the surface movement of a site due to unloading can be calculated as

$$y_s = \int_0^{H_s} \frac{dh}{1+e} c_r \log \frac{\sigma'_2}{\sigma'_1} = \int_0^{H_s} \frac{dh}{1+e} c_r \log \frac{\sigma_{net} + \chi s_2}{\sigma_{net} + \chi s_1} \quad (11)$$

Now for shallow reactive soils  $\sigma_{net} \ll \chi s$ . Substituting for  $\chi$  from (4), equation (10) can be rearranged as

$$y_s = \int_0^{H_s} 0.45 \frac{dh}{1+e} c_r \log \frac{s_2}{s_1} = \int_0^{H_s} \frac{0.45 c_r}{1+e} \Delta u dh \quad (12)$$

Comparing (12) with (10), yields

$$I_{pt} = \frac{0.45 c_r}{1+e} \quad \text{for} \quad s_1, s_2 \geq s_e \quad (13)$$

which establishes a relationship between the instability index,  $I_{pt}$ , and unloading-reloading compression index,  $c_r$ . The relationship (13) is only valid if the suction change occurs in the unsaturated region. If the suction changes are below the air entry value (for drying) and air expulsion value (for wetting),  $s_1, s_2 < s_e$ , then all the

moisture variations occur in the saturated region, in which  $\chi = 1$ . In this case, the relationship (13) becomes

$$I_{pt} = \frac{c_r}{1+e} \quad \text{for} \quad s_1, s_2 < s_e \quad (14)$$

Finally, the equivalencies in (13) and (14) are for constrained deformation only. If the movement is unconstrained; e.g. in cracked zones, the instability index must be corrected according to

$$I_{pt}(\text{unconst}) = \frac{I_{pt}(\text{const})}{\alpha} \quad (15)$$

where  $\alpha = 1.7 \sim 2.1$  depending on the Poisson's ratio of the soil.

## 6 CONCLUSIONS

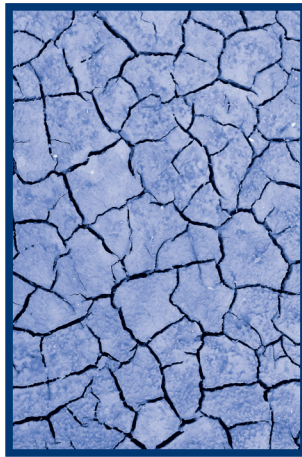
Guidelines are provided for the application of the effective stress principle to shear strength and volume change in unsaturated soils. Detailed discussions are provided on material parameter determination, and typical mistakes made in the literature. In particular, a relationship is proposed for the instability index used in AS2870 in terms of the basic soil properties. The ramification of this is that a mechanistic determination of site movement due to moisture change in reactive soils may be possible using the soil parameters that are used in saturated soils mechanics.

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