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State of the Art Report. Advances in Unsaturated Soil Mechanics: Constitutive modelling, experimental investigation, and field instrumentation

Nasser Khalili
School of Civil and Environmental Engineering, UNSW Sydney, Australia, n.khalili@unsw.edu.au

Enrique Romero
Department of Civil and Environmental Engineering, UPC, and CIMNE, Spain, enrique.romero-morales@upc.edu

Fernando A. M. Marinho
Department of Sedimentary and Environmental Geology - IGc/USP, Brazil, fmarinho@usp.br

ABSTRACT: This state-of-the-art covers advances in the constitutive modelling, experimental investigation and field measurement of suction for unsaturated soils. It consists of three distinct parts. Part 1 is devoted to the constitutive modelling and covers major developments in the field including elasto-plasticity of soil skeleton, encompassing the critical role of unsaturation, and treatment of cyclic loading, expansive soils, aggregation/cementation/structure, rate dependency of response, and anisotropy. An attempt has also been made to address the choice of stress variables for constitutive modelling of unsaturated soil including the role of the effective stress concept principle. Part 2 is devoted to the experimental advances in unsaturated soils, focusing on the hydro-mechanical phenomena as well as the microstructural effects and the experimental approaches for their examination. Part 3 is dedicated to the measurement of suction in the field, various probes that can be used for this purpose, their advantages and disadvantages, and the correct approach for their positioning and installation.

RÉSUMÉ : Cet état de l'art couvre les avancées en matière de modélisation constitutive, d'investigation expérimentale et de mesure sur le terrain de la succion des sols non saturés. Il se compose de trois parties distinctes. La première partie est consacrée à la modélisation constitutive et couvre les principaux développements dans le domaine, y compris l'élasto-plasticité du squelette du sol, englobant le rôle critique de l'insaturation, et le traitement de la charge cyclique, les sols expansifs, l'agrégation/la cémentation/la structure, la dépendance de la réponse à la vitesse, et l'anisotropie. Une tentative a également été faite pour aborder le choix des variables de contrainte pour la modélisation constitutive du sol non saturé, y compris le rôle du principe du concept de contrainte effective. La partie 2 est dédiée aux avancées expérimentales dans les sols non saturés, en se concentrant sur les phénomènes hydromécaniques ainsi que sur les effets microstructuraux et les approches pour les examiner. La troisième partie est consacrée à la mesure de la succion sur le terrain, aux différentes sondes qui peuvent être utilisées à cette fin, à leurs avantages et inconvénients, et à l'approche correcte de leur positionnement et de leur installation.

KEYWORDS: Unsaturated soils, constitutive modelling, experimental investigation, field instrumentation

1 INTRODUCTION

Since the pioneering work of Bishop and his co-workers in the 1950's, tremendous advances have been made in the constitutive modelling, experimental investigation and field monitoring of unsaturated soils. The early work on the mechanics of unsaturated soils followed the classical approach in soil mechanics where the soil resistance to shearing and the soil deformation due to loading were treated separately as two independent phenomena. The first fully integrated strength-deformation model for the behaviour of unsaturated soils, within a general framework of elasto-plasticity, was introduced by Alonso and his co-workers in the late 1980's and early 1990's. This was followed by a rapid succession of seminal contributions which have since underpinned perhaps all the advanced developments in the constitutive modelling of unsaturated soils. Nevertheless, the theoretical developments in the mechanics of unsaturated has not been without a controversy. There have been robust discussions and disagreements on the choice of stress variables and their conjugates for the purposes of constitutive modelling, the role of the stress state variables as distinct from stresses that are used for the constitutive modelling, the role of suction in unsaturated soil mechanics, the applicability of the effective stress to unsaturated soils and its true meaning, as well as the most appropriate way to tackle the intricate aspects of air and water interaction within the soil's pore space and their cross-coupling with the non-linear deformation of the soil matrix. One the key aims of this state-of-the-art is to address these as well as other points of disagreement and pave the way for the future developments in the field, as well as highlighting the tremendous advances that have accomplished to date. Topics covered will include: stress state variables for multi-phase media, derivation of the effective stress for fluid(s) saturated porous media, hydro-mechanical coupling of water retention response, elasto-plasticity in unsaturated soils, and the treatment of cyclic loading, expansive soils, aggregated and structured soils, rate dependency and anisotropy.

In parallel with the constitutive modelling of unsaturated soils, significant advances have also been accomplished in the experimental and field investigations of unsaturated soils. These have clarified the fundamentals of unsaturated soil mechanics and have underpinned the advances in the theoretical developments. Significant progress has been made in the use of multi-physics and multi-scale techniques and their application to unsaturated soils. New devices and experimental procedures have been developed allowing phase separation, investigation of coupled processes, and bridging scales for the purpose of constitutive modelling. These have permitted unparalleled access to small scale microstructural soil features at the pore/grain level with a profound impact on understanding the larger-scale phenomenological processes. The advances in the experimental techniques have also permitted investigation of new unsaturated geomaterials (such as rockfill materials, artificially prepared mixtures, bentonite-based materials) and new multi-physics processes in support of emerging fields in geotechnical engineering. Of particular interest has been the development of technologies to improve resiliency of geo-infrastructure, adapt to climate change, and mitigate the associated risk.

The section on the experimental and field investigations of this state-of-the-art will describe laboratory techniques and the advances made over the last three decades. Particular focus will be on studying the coupled hydro-mechanical and multi-scale phenomena, and the experimental techniques that are advanced for the cross-disciplinary applications of unsaturated soil mechanics. In particular, the experimental techniques for controlling and measuring suction, hydro-mechanical cells, microstructural studies, 1-g and centrifuge scaled tests will be discussed. In addition, advances in ground engineering, environmental and energy geotechnics, bioinspired technologies, tailings and industrial processes will be covered. The basic concepts of pore pressure measurement in both the laboratory and the field will be presented. A brief historical account of the development of suction measurements in the field is presented. The specifics of the most common suction probes are discussed and their limitations and advantages are highlighted. The correct positioning of these probes in the field for a reliable measurement of suction in the field is also covered.

2 BASIC CONCEPTS

2.1 *Kinematics and Definitions*

Unsaturated soils are three phase porous media consisting of solid particles and a pore space filled with water and air. One fluid, water, is wetting, and the other air, is not. Phases (solid, s, water, w, and air, a) represent the constituents when viewed as a part of the mixture. The solid constituent is assumed to be slightly compressible, or incompressible if simplifications are desired. Each constituent has a mass M_α and a volume V_α, $\alpha = s, w, a$, which make up the total mass $M = M_s + M_w + M_a$ and the total volume $V = V_s + V_w + V_a$. Intrinsic quantities are defined using subscripts and apparent quantities using superscripts. For example intrinsic mass density of α phase is denoted $\rho_\alpha = M_\alpha/V_\alpha$, whereas the apparent mass density is written as $\rho^\alpha = M_\alpha/V$; hence $\rho^\alpha = n^\alpha\rho_\alpha$, where $n^\alpha = V_\alpha/V$ is the apparent volume fraction of constituent α. The apparent volume fractions satisfy the constraint $n^s + n^w + n^a = 1$. The sign convention of soil mechanics is adopted throughout. Tensor quantities are identified by boldface letters.

2.2 *Stress State Variables*

The smallest number of stress variables controlling the state of a system are called the *stress state variables* of that system. For a multi-phase system of β constituents, the number of stress state variables are equal to $\beta - \xi$, where ξ is the number constraints within the system. For example, the stress state for saturated soils (a two-constituent material), when no constraints are imposed, is captured via two stress state entities; i.e. $\boldsymbol{\sigma}$, the total stress tensor and, p_w, the pore water pressure. Imposing the solid constituent incompressibility constraint, as a special case, the stress state variables reduce by one, taking the form $(\boldsymbol{\sigma} - p_w\boldsymbol{I})$, in which $\boldsymbol{I}$ is the identity tensor.

For unsaturated soils, the stress state variables are $\boldsymbol{\sigma}$, p_w and p_a, where p_a is the pore air pressure. Again, introducing the incompressibility of the solid grains, the stress state variables reduce to being one of any pair of the stresses: $(\boldsymbol{\sigma} - p_a\boldsymbol{I})$, $(\boldsymbol{\sigma} - p_w\boldsymbol{I})$ and $(p_a - p_w)$ (Fredlund and Morgenstern 1977). The most commonly adopted pair is $\boldsymbol{\sigma}^{net} = (\boldsymbol{\sigma} - p_a\boldsymbol{I})$, the net stress tensor, and $s = (p_a - p_w)$, the matric suction. Although constitutive models using the combination of $(\boldsymbol{\sigma} - p_w\boldsymbol{I})$ and $(p_a - p_w)$ have also been presented (Geiser 1999).

2.3 *Effective Stress*

The effective stress principle plays a critical role in the constitutive modelling of multi-phase media. Often referred to as the axiom of soil mechanics (Atkinson 2007), it enters the elastic as well as elasto-plastic constitutive equations of the solid phase, linking a change in stress to straining or any other relevant quantity of the soil skeleton, such as compression, distortion and a change of shearing resistance (e.g., see Terzaghi 1936, Biot 1941, Rice and Cleary 1976, de Boer and Ehlers 1990, Coussy 1995). Without the advent of the effective stress principle, many of the significant achievements of solid mechanics - such as elasto-plasticity, visco-plasticity, wave propagation, and numerical methods - could not be extended to soil engineering problems without empiricism.

Based on experimental observations, and for saturated soils, Terzaghi (1936) defined the effective stress, $\boldsymbol{\sigma}'$, as "*the total stress in excess of the neutral stress*"

$$\boldsymbol{\sigma}' = \boldsymbol{\sigma} - p_w\boldsymbol{I} \tag{1}$$

in which $\boldsymbol{\sigma}'$ is the effective stress tensor, and p_w is the neutral stress or the pore water pressure. Terzaghi stated that "*all measurable effects of a change of stress, such as compression, distortion and a change of shearing resistance are exclusively due to change in the effective stress*". The first use of this principle is implicit in Terzaghi's work on the theory of one-dimensional consolidation, albeit without referring to the term effective stress (Terzaghi 1923). Bishop and Blight (1963) defined the effective stress as that function of the stress state variables that controls the mechanical effects of a change in stress, such as volume change and a change in shear strength. Lade and De Boer (1997) defined the effective stress as "*the stress that controls the stress-strain, volume change, and strength behaviour of a given porous medium ….. The pore pressure may be zero or negative, or it may be positive and very large, but the effective stress must be expressed such that it produces the same material response for any pore pressure. It is to be expected that some properties of the given porous medium will be part of a more comprehensive effective stress expression, unlike the simple expression in equation (1), which does not involve any material properties.*".

In essence, the effective stress principle converts a multi-phase, multi-stress state porous medium to a mechanically equivalent, single-phase, single-stress state continuum, hence permitting the application of the principles of continuum solid mechanics to fluid-filled deformable porous media (Blight 1967, Khalili et al. 2004, Nuth and Laloui 2008). Specifically, the effective stress enables capturing "*the stress-strain, volume change, and strength behaviour of a porous medium, independent of the magnitude of the pore pressure*", and solely based on the underlying drained (effective) properties of the soil skeleton (Lade and De Boer 1997).

Within this context, the effective stress may be considered as the *constitutive stress* of the solid skeleton (e.g. Skempton 1960, Broja and Koliji 2009, Alonso et al. 2010), with "*its seat* *exclusively in the solid skeleton of the soil*" (Terzaghi 1936). This distinction is important as it clarifies that effective stress only pertains to the solid skeleton of the porous medium, and thus only controls the stress state of the solid skeleton rather than the overall state or equilibrium of the multi-phase system (e.g. see Coussy 1995, Lade and De Boer 1997, Cheng 2016 and more recently Guerriero and Mazzoli 2021).

Another important, yet less recognised, role of the effective stress principle is that it furnishes a platform for coupling the deformation of the solid skeleton to the volume change of the fluid constituents. This is critical for establishing consistent hydro-mechanical models for multi-phase porous media (Loret and Khalili 2000, Khalili et al 2008, Vaunat and Casini 2017).

In its most general form, the effective stress expression for a multi-phase porous medium, consisting of ς fluids, may be expressed as (Khalili 2008)

$$\boldsymbol{\sigma}' = \boldsymbol{\sigma} - \textstyle\sum_{\varsigma} \eta_{\varsigma}\, p_{\varsigma} \boldsymbol{I} \tag{2}$$

in which $\boldsymbol{\sigma}'$ is the effective stress tensor, $\boldsymbol{\sigma}$ is the total stress tensor, and p_{ς} fluid pressure. Specialisation of (2) for saturated soils results in

$$\boldsymbol{\sigma}' = \boldsymbol{\sigma} - \eta p_w \boldsymbol{I} \tag{3}$$

in which η is the effective stress parameter for saturated porous media.

For unsaturated soils, (2) becomes

$$\boldsymbol{\sigma}' = \boldsymbol{\sigma} - \eta_w p_w \boldsymbol{I} - \eta_a p_a \boldsymbol{I} \tag{4}$$

in which η_w and η_a are the effective stress parameters of the pore-water pressure and pore-air pressure, respectively. Within the practical range of stresses, and for soils with incompressible solids, it can be shown that that the effective stress parameters η_w and η_a satisfy the constraints $\eta_w + \eta_a = 1$ and that (4) reduces to (Bishop 1959)

$$\boldsymbol{\sigma}' = \boldsymbol{\sigma} - p_a \boldsymbol{I} + \chi (p_a - p_w) \boldsymbol{I} \tag{5a}$$

or

$$\boldsymbol{\sigma}' = \boldsymbol{\sigma}_{net} + \chi s \boldsymbol{I} \tag{5b}$$

in which $\boldsymbol{\sigma}_{net} \equiv \boldsymbol{\sigma} - p_a \boldsymbol{I}$ is the net stress, $s \equiv p_a - p_w$ is the matric or matrix suction and $\chi = \eta_w$ is the effective stress parameter quantifying the contribution of suction to the effective stress of the solid skeleton.

2.3.1 *Effective stress parameter for saturated soils*

The early developments on the role of the effective stress principle in the constitutive modelling of saturated porous media were marred with remarkable, yet largely forgotten, controversy, arguments and personal tragedy. Of note were the heated exchanges between Karl von Terzaghi, widely known as the father of modern soil mechanics, and Paul Fillunger, the father of mixture theory. Paul Fillunger suicided around the time of these arguments. A comprehensive account of the exchanges, and events leading to this tragic outcome, is detailed in the comprehensive work of de Boer (2000, 2005).

Numerous investigations have been reported in the literature on the determination of the form of the effective stress for saturated porous media. Several candidates have been proposed for the effective stress parameter, η.

Fillunger (1930, 1936), on theoretical grounds, and satisfying the equilibrium equations of the entire soil mixture; i.e. using the volume averaged stresses of the solid and water, expressed the effective stress as

$$\boldsymbol{\sigma}' = \boldsymbol{\sigma} - n p_w \boldsymbol{I} \tag{6}$$

in which n is the porosity. Fillunger essentially defined the effective stress as the *partial/apparent stress* of the solid phase, expressed as $\boldsymbol{\sigma}^s = (1-n)\boldsymbol{\sigma}_s$, in which $\boldsymbol{\sigma}_s$ is the intrinsic stress of the solids (Bowen 1976). While this expression was appropriate for the equilibrium and the balance of momentum of the solid-water mixture, it was invalid as a constitutive stress for the mechanical response of the solid skeleton and relating straining of the soil skeleton to a change in stress. Similar, expressions were also made by Hoffman (1928), Lubinski (1954), Biot (1955), Schiffman (1970). Terzaghi (1923, 1945) also initially proposed $\eta = n$ for saturated soils and rocks, but found experimentally that $\eta = 1$.

Skempton and Bishop (1954) speculated that the effective stress is equal to the *average intergranular stress* acting between particles and/or clasts. Accordingly, they derived the expression

$$\boldsymbol{\sigma}' = \boldsymbol{\sigma} - (1 - a_c) p_w \boldsymbol{I} \tag{7}$$

in which a_c is the contact area between particles per unit area of the porous medium. It was also argued that a_c is negligible for soils, with (7) reducing to (1) for most geotechnical engineering applications.

Equation (7) was based on satisfying equations of equilibrium for the porous medium. This is frequently presented in soil mechanics textbooks as the proof of Tezaghi's expression for the effective stress of saturated soils. However, as noted by Alonso et al (1987) the form of the equations of equilibrium "*is not necessarily a proof of an effective stress statement for constitutive modelling*" of porous media. The fundamentals of such an approach were also questioned by Bloch (1978). Later, based on experimental data using oedometer tests on lead shot, Skempton (1960) showed that in fact equation (7) was invalid and that the contact area between particles played no role in the formulation of the effective stress.

The effective stress expression that was supported by many investigators (e.g. Biot 1941, Skempton 1960, Nur and Byerlee, 1971, Lade and De Boer 1997, Coussy 2010) involved the ratio of the bulk modulus of the solid grains, K_s, and the drained bulk modulus of the solid skeleton, K, as

$$\boldsymbol{\sigma}' = \boldsymbol{\sigma} - \left(1 - \frac{K}{K_s}\right) p_w \boldsymbol{I} \tag{8}$$

It was noted that for most soils K_s was significantly larger than K, leading to η approaching unity and recovering Terzaghi's effective stress expression. However, for rocks and concrete η typically attained values less than 1 (e.g. see Brace 1965), rendering application of Terzaghi's effective to such media invalid. Proofs of (8) have been presented by Biot (1941) through theory of poro-elasticity and invoking existence of elastic potential, and Nur and Byerlee (1971) through establishing a strain equivalency between the saturated porous medium and an equivalent continuum with the same underlying mechanical properties. Similar proofs using the theory of poro-elasticity have also been present by Geertsma (1957, 1966), Lade and De Boer (1997), Coussy (2010) amongst many others. The main premise in all these developments has been the fundamental requirement that the effective stress of the solid skeleton is related uniquely, and in a one-to-one manner, to the elastic strain of the solid skeleton through

$$\boldsymbol{\sigma}' \equiv \boldsymbol{D}^e \boldsymbol{\varepsilon}^e \tag{9}$$

in which $\boldsymbol{D}^e$ is the underlying drained elastic property tensor of the solid skeleton, and $\boldsymbol{\varepsilon}^e$ is the elastic strain tensor. Within this context, any stress satisfying (9) is the effective stress of the solid skeleton. Once the effective stress expression is established in the elastic region, it can then be applied to the elasto-plastic analysis by invoking an appropriate elastic-plastic model (de Boer and Ehlers 1990, Loret and Khalili 2000, 2002, Coussy 2010). As elaborated by de Boer and Ehlers (1990), the treatment of elastic-plastic or viscous skeletons would not change the basic statement about the effective stress.

Suklje (1969) also used the theory of poro-elasticity but derived the relationship

$$\boldsymbol{\sigma}' = \boldsymbol{\sigma} - \left(1 - (1-n)\frac{K}{K_s}\right) p_w \boldsymbol{I} \tag{10}$$

which is different from the relationship presented in (8) by the term $(1-n)$. In this derivation, Suklje assumed that the macroscopic volume change of the solid skeleton due to an all around pore pressure is equal to the strain of the solid fraction of the soil skeleton. This is clearly not correct as it ignored the structure of the solid skeleton and change in the porosity as the solid grains strained.

The following points, from the literature, are key to understanding the role of the effective stress:

- The effective stress is a constitutive stress with the sole role of capturing the mechanical response of the solid skeleton due to a change in stress.
- The effective stress is distinct from *stress state variables of the soil*, the *partial stress* of the solids and the *intergranular stress*.
- The effective stress pertains only to the solid skeleton of the soil, and as such it may only be considered as the stress state variable of the solid phase. It is not, and it cannot be expected to be, a stress state variable of the entire soil-water mixture. Only for the special case of saturated soils, with the constraint of incompressible solid grains, does the effective stress fulfil the role of both the constitutive stress and the stress state parameter.
- The effective stress is exclusively associated with, and only with, the elastic component of straining of the solid skeleton (equation 9). This is irrespective of whether the stress-strain response of the soil skeleton is in the elastic, elasto-plastic or elasto-visco-plastic region.
- The form of the effective stress may be obtained, as is the convention, by establishing a mechanical equivalency between a mutli-phase system and a continuum, with the continuum having the same underlying drained mechanical properties of the multi-phase system.
- The mechanical equivalency may be established based on a strain equivalency (e.g. Nur and Byerlee 1971), strength equivalency (Vanapalli and Fredlund 1996, Khalili and Khabbaz 1998) or an energy equivalency analysis (de Boer and Ehlers 1990, Gray and Schrefler 2001, Borja (2006, Einav and Liu 2018). Irrespective of the way the effective stress is established, it must be applicable to all mechanical responses of the soil; e.g. deformation and strength.

An observation in the rock mechanics literature, which is contrary to the last point listed above, is that the deformation is controlled by Skempton's effective stress (8), whereas the shear strength is correlated with Terzaghi's effective stress (1). This is referred to as a paradox of rock mechanics. However, as elaborated on by Lade and de Boer (1997), both phenomena are controlled by the same form of the effective stress as presented in (8). They note that "*during the process of shearing concrete and/or rock, micro fissures develop and open up, and at the time of peak failure, sufficient deterioration of the solid material has occurred such that the bulk modulus of skeleton forming the shear zone has decreased dramatically. Therefore, the expression in equation (8), which is most often employed for the stress-strain calculations, approaches unity near failure, even at high confining pressures. This means that the expression in equation (8) captures the effective stress for both stress-strain and strength behaviour.*"

2.3.2 *Effective stress parameter for unsaturated soils*

Similar to saturated soils, numerous contributions have been made to the determination of effective stress parameter, χ, for unsaturated soils. They range from rigorous analyses based on thermodynamic laws to simple volume averaging procedures, or expressions that are based entirely on intuition.

Earlier expressions of the effective stress parameter for unsaturated soils assumed a direct equivalency with the degree of saturation (Bishop 1959, Bishop and Donald 1961, Bishop and Blight 1963). They provided a geometrical interpretation of the effective stress parameter, however no unique relationship could be found between χ and the degree of saturation. Examples of the use of degree of saturation as the effective stress parameter, $\chi = S_r$, in constitutive modelling of unsaturated soils include (Schrefler 1984, Öberg and Sällfors 1995, Bolzon et al. 1996, Jommi 2000, Gallipoli et al. 2002, Gallipoli et al., 2003, Wheeler et al. 2003, Laloui et al 2003, Sheng et al. 2004, Tamagnini 2004, Sun et al. 2007a,b,c, Nuth and Laloui 2008, Romero and Jommi 2008). While convenient from a constitutive modelling view point, there is overwhelming experimental evidence, gathered since 1960's, against the appropriateness and the use of the degree of saturation as the effective stress parameter (e.g. see Zerhouni, 1991, Vanapalli and Fredlund 2000, Khalili and Zargarbashi 2010, Alonso et al. 2010, Pereira et al. 2010, Konrad and Lebeau 2015, as typical examples). In general, $\chi = S_r$ tends to over-estimate the effective stress and the shear strength of unsaturated clays (Pereira et al. 2010), whereas the reverse happens for sands (Konrad and Lebeau 2015). Furthermore, $\chi = S_r$ implies that, at large suctions, where the degree of saturation approaches a limiting residual value, the effective stress and thus the shear strength are directly proportional to suction. This is not supported by the experimental evidence (Loret and Khalili 2002, Alonso et al. 2010). In addition, on theoretical grounds, incorporating physics at the microscale (Alonso et al. 2010, Lu et al. 2010), and based on the thermodynamics of multiphase systems (Fuentes and Triantafyllidis 2013, Jian et al. 2017, Einav and Liu 2018), it is shown that $\chi = S_r$ is only recovered when the work of the air-water interface is neglected (Hassanizadeh and Gray 1990, Houlsby 1997, Hutter et al. 1999, Gray and Schrefler 2001, Borja 2006, Gray and Schrefler 2007, Coussy 2010, Zhao et al. 2010, Nikooee et al. 2013 (to name a few)).

Consensus is gradually emerging with the view that it is better to make χ a function of the effective degree of saturation, $S_e = (S_r - S_{re})/(1 - S_{re})$

$$\chi = f(S_e) \tag{11}$$

where S_{re} is the residual degree of saturation.

Karube and Kato (1994), Karube et al. (1998), and more recently Kim et al. (2010), associated the effective stress parameter to the contributions of the "*bulk water*" and "*meniscus water*". Bulk water was defined as the pore water that occupied the pore volume between soil particles, and the meniscus water was taken to exist at the contact points of the soil particles only. Two effective stress parameters were defined: one corresponding to the bulk water, χ_b, and the other to the meniscus water, χ_m

$$\chi_b = \frac{(S_r - S_{rd})}{(1 - S_{rd})}, \quad \chi_m = \frac{(1 - S_r)(S_{rd} - S_{re})}{(1 - S_{re})(1 - S_{rd})} \tag{12}$$

where S_{rd} is the driest degree of saturation obtained from the main wetting path of the soil water retention curve (SWRC) at the suction of interest. The sum of χ_b and χ_m was taken the effective stress parameter which reduced to the effective degree of saturation, S_e, (Kim et al. 2010)

$$\chi = \chi_b + \chi_m = S_e \tag{13}$$

Similarly, Konrad and Lebeau (2015) divided the degree of saturation into two components: absorbed, S_{rb}, and capillary S_{rc}, and related χ exclusively to S_{rc} as

$$\chi = S_{rc} = S_e$$

Lu and Likos (2004, 2006) and Lu et al. (2010) defined the effective stress parameter as

$$\Theta = \frac{(\theta - \theta_{re})}{(\theta_s - \theta_{re})} \cong S_e \tag{14}$$

where Θ is the normalised volumetric water content, θ is the volumetric water content, θ_s is the saturated volumetric water content and θ_{re} is the residual volumetric water content. For all practical purposes Θ may be approximated as S_e.

In parallel with the above developments, and by analysing strength data from 14 different cases involving glacial tills, silts, sandy clays and clays, Khalili and Khabbaz (1996, 1998) proposed, within a good accuracy, a unique relationship between χ and the ratio s/s_e

$$\chi = \begin{cases} 1 & for \quad s/s_e \leq 1 \\ s/s_e{}^{-\gamma} & for \quad 1 \leq s/s_e \leq 14 \end{cases} \tag{15}$$

where s_e is the suction value marking the transition between saturated and unsaturated states. For the main wetting path $s_e = s_{ae}$, and for the wetting drying path $s_e = s_{ex}$, in which s_{ae} is the air entry value and s_{ex} is the air expulsion value. Khalili and Khabbaz (1998) showed that the best-fit value of the exponent assumes $\gamma = 0.55$ for different soil types. Extension of (15) to $s/s_e \geq 14$ was given in Russell and Khalili (2006). The effect of suction reversals was in turn examined by Khalili and Zargarbashi (2010).

Describing the water retention with the Brooks and Corey (1964) model, Mašín (2010, 2013) and Khalili (2018) showed that (15) may alternatively be presented as

$$\chi = S_e{}^{\frac{\gamma}{\lambda_p}} \tag{16}$$

in which λ_p is the slope of soil water retention curve presented in log-log scale. As pointed out by Khalili (2018) equations (15) and (16) are equivalent and may be used inter-changeably for the determination of the effective stress parameter, depending on the availability of the relevant input data.

Xu (2004) and Xu et al (2015) adopted fractal mechanics and showed that, using self-similarity of pores in soils, the fractal representation of the effective stress parameter will be of the form

$$\chi = (s/s_e)^{D-3}$$

in which D is the fractal dimension of the pore size distribution, with a value that usually lies between 2.4 and 2.6 for natural soils.

Vanapalli et al. (1996) and Fredlund and Vanapalli (2000), linked the suction contribution to the shear strength of unsaturated soil to the aerial fraction of the water constituent, and proposed

$$\chi = S_r{}^{\kappa} \tag{17}$$

with κ being a fitting parameter attaining a value of 1 for non-plastic granular materials and 3 for highly plastic clays. An expression identical to (17) was also presented by Alonso et al. (2010) based on micro mechanical considerations of soils with two dominant pore sizes. In both approaches, $\kappa \geq 1$, reduced to $\kappa = 1$ for granular materials. This resulted in $\chi = S_r$ for non-plastic silts and sands which may not be applicable, as discussed earlier. For granular materials κ is typically less than 0.5. In addition, both approaches maintained one of the key drawbacks of correlating χ directly with S_r; that is at large suctions, the degree of saturation, and hence $S_r{}^{\kappa}$, in this case, approached a limiting residual value, making the shear strength linearly proportional to suction. Again, as stated previously, this is not supported by experimental evidence.

Some investigators have also advocated the use of a '*suction stress*' to represent the contribution of suction to the effective stress of the solid skeleton (e.g. Karube and Kato 1994, Karube et al. 1998, Lu and Likos 2004, Lu and Likos 2006, Lu et al. 2010).

Karube and Kato (1994), Karube et al. (1998) and Lu and Likos (2004) defined the suction stress, σ^s, as the multiplication of the effective stress parameter and the matric suction, $\sigma^s = \chi s$. This definition is unproblematic when determining the shear strength of unsaturated soils. For the purpose of constitutive modelling, however, it is more fundamental to quantify χ separately from suction, s, since χ not only captures the contribution of suction to the effective stress, it also acts as the coupling term between flow and deformation fields in unsaturated soils (Khalili et al. 2008). When σ^s is adopted in a fully coupled hydro-mechanical analysis, it is necessary to recover χ from σ^s using $\chi = \sigma^s/s$.

However, more recently, Lu and Likos (2006) and Lu et al. (2010) extended the notion of suction stress to include not only the capillary effects, i.e. χs, but also physico-chemical effects such as van der Waals forces, electrical double-layer forces and cementation in a lumped fashion and relating all these effects to matric suction and volumetric water content of the soil. There are several difficulties with this approach:

- Physico-chemical effects are controlled by the chemistry of the pore water and the mineralogy of the soil. There are no direct correlations between the matric suction and/or the water content with the physico-chemical effects in a soil. To capture physico-chemical effects one must introduce chemical potential or osmotic suction as an independent state variable.
- Physico-chemical effects are intra-aggregate phenomena and affect primarily the mechanical properties of the soil rather than the effective stress. Therefore, their inclusion in the effective stress statement is inappropriate and cause major difficulties in the constitutive modelling of soils. For example, it is accepted that cementation increases the shear strength of soils. However, this is achieved through an increase in the strength parameters of the soil rather than an increase of the effective stress. If one is to introduce the cementation in the effective stress equation, increasing cementation must cause not only an increase in the shear strength, but a volume contraction of the soil, which is not the case.
- Physico-chemical phenomena are also present in saturated soils, and through the use of Terzaghi's effective stress principle, their effects are already reflected in the mechanical properties of the soil. Re-including the physico-chemical effects in the effective stress expression will lead to double counting of such effects.
- The usual approach in the constitutive modelling of multi-phase media is to carefully and systematically identify physical and chemical processes present in the system and capture their behaviour through introducing phenomenon significant variables and parameters. Lumping independent processes into a single constitutive expression can lead to intractability of cause and effect within the system.

2.4 *Soil Water Retention Curve*

Another important aspect in the mechanics of unsaturated soils is the soil water retention curve (SWRC), which provides a fundamental relationship between the amount of water held in the soil pore space and matric suction. It is the basis for many empirical relationships for unsaturated soils such as shear strength, volume change and hydraulic properties (e.g. Biarez et al. 1987, Fredlund et al. 1978; Vanappali et al. 1996, Khalili and Khabbaz 1998, Gallipoli et al. 2003; Zhou et al. 2012a,b), and appears explicitly in the constitutive formulations of unsaturated soils, linking pore-phase volume changes to a change in suction (Dangla et al. 1997, Loret and Khalili 2000, Jommi 2000, Wheeler et al. 2003, Sheng et al. 2004, Khalili et al. 2008, Coussy 2010, Khoshghalb and Khalili 2013, Salimzadeh and Khalili 2014, Song and Borja, 2014, and Ghaffaripour et al. 2019). Indeed, more research effort has been devoted to the quantification and understanding of the water retention behaviour than any other phenomenon relevant to the mechanics of unsaturated porous media. The SWRC is commonly described as a plot of either gravimetric water content, or volumetric water content, or degree of saturation, against the logarithm of matric suction. Due to hydraulic hysteresis, the SWRC typically has two different branches, one corresponding to wetting and the other to drying. Generally, the water content is higher for drying compared to wetting at the same suction value.

Numerous approaches have been proposed for mathematical representation of soil water characteristic curve. Summaries have been presented by Fredlund and Xing (1994), Leong and Rahardjo (1997), Sillers et al (2001) and Lu and Likos (2004). The models proposed by Brooks and Corey (1964), van Genuchten (1980) and Fredlund and Xing (1994), in particular, have been popular in the geotechnical engineering community. The model proposed by Brooks and Corey (1964) is one of the earliest and simplest equations for the soil water characteristic curve. The van Genuchten (1980) model takes into account the pore size distribution of the soil and provides a continuous soil water characteristic curve. As such, it allows for greater flexibility in fitting the degree of saturation over the entire suction range. The approach proposed by Fredlund and Xing (1994) is similar to the van Genuchten (1980) model and is developed by modifying the pore size distribution function given by van Genuchten (1980).

It is generally understood that deformation can markedly affect the water retention response of soils (Vanapalli et al. 1999, Ng and Pang 2000, Mašín 2010, Romero et al. 2011, Gallipoli et al. 2003 and 2012, Salager et al. 2013, Fredlund 2015, Pasha et al. 2016 and 2017, Ng et al. 2018, Gao and Sun 2017, Lu and Yi 2017, Zhou et al. 2019, Liu et al. 2020, Pasha et al. 2019 and 2020). Changes in the soil's pore structure directly affect water movement in the soil by altering the size distribution of the pores as well as the pore connectivity that has direct impact on the SWRC. Therefore, the SWRC obtained at a specific soil volume or compacted state cannot be used at other states. In other words, the SWRC must be determined at the void ratio of interest to be of a fundamental and practical value. Ignoring the impact of void ratio on the SWRC can lead to significant error and misunderstanding in extracting fundamental engineering properties of unsaturated soils based on the SWRC (Pasha et al. 2016, Lajmiri at al. 2020).

Many researchers have examined the coupling between SWRC and soil volumetric strain. Notable contributions include Wheeler et al. (2003), Gallipoli et al. (2003), Nuth and Laloui (2008), Mašín (2010), Gallipoli (2012), and Pasha et al. (2016, 2020). Experimental contributions in this area are attributed to Vanapalli et al. (1999), Romero and Vaunat (2000), Ng and Pang (2000), Lee et al. (2005), Miller et al. (2008), Tarantino (2009), Salager et al. (2013) to name a few. The effect of volume change on soil microporosity and its link with the SWRC can also be found in the experimental results of Simms and Yanful (2002), Koliji et al. (2006), Romero et al. (2011) and Casini et al. (2012).

Despite the important effect of the void ratio and volume change on the SWRC, a review of the literature shows that in the majority of the cases the void ratio dependency of SWRC is ignored. This has led to erroneous use and at times contradictory interpretations of SWRC data, particularly in relation to the evolution of the air entry value and pore size distribution index with void ratio (e.g. see Rassam and Williams 1999, Sun et al. 2007a,b,c, Nuth and Laloui 2008, and Morvan et al. 2011). Some studies indicate an increase in the value of the pore size distribution index with decreasing void ratio, leading to a steeper SWRC in the unsaturated region for an increase of compaction (e.g. Huang et al. 1998, Karube and Kawai 2001, and Zhou et al. 2018). Others suggest an opposite trend with the pore size distribution index decreasing with decreasing void ratio, leading to a flatter SWRC in the unsaturated region (e.g. Gallipoli et al. 2003, Mašín 2010, Pasha et al. 2017 and 2020). There are also a number of studies that assume the slope of the SWRC is unaffected by void ratio (e.g. Nuth and Laloui 2008, Gallipoli 2012, Hu 2013, Russell 2014).

Noting the important role of the void ratio on the water retention characteristics of soils, several SWRC models, with two or more fitting parameters, have been proposed in the literature that explicitly take into account the effect of void ratio. Examples include the work of Gallipoli et al. (2003), Tarantino (2009), and Sheng and Zhou (2011). Several other investigators have attempted to capture the void ratio dependency of the SWRC through the net stress (e.g. see Zhou and Ng 2014). However, such a representation is unlikely to lead to a satisfactory outcome, as two samples of the same soil, under the same net stress, but different loading and unloading histories, may possess entirely different void ratios and hence exhibit markedly different SWRCs. Also, fitting type models often require extensive experimental data for the calibration of the model, and tend to perform poorly outside the domain of the calibration data.

Mašín (2010) was perhaps first to propose a predictive model for the evolution of SWRC with void ratio. Adopting equation (15) and invoking the existence of elastic and plastic potential for unsaturated soils, he was able to derive an expression that captured elegantly the evolution s_e with void ratio as

$$\dot{s}_e = \frac{\gamma s_e}{e\lambda_p}\dot{e} \tag{18}$$

We recall that s_e is the suction value that separates saturated from unsaturated states of a soil. Dependency of λ_p (the slope of SWRC in a log-log state) on void ratio and suction was in turn given by

$$\lambda_p = \frac{\gamma}{ln\chi_o}\ln\left[\left({\chi_o}^{\lambda_{po}/\gamma} - \chi_o\right)\left(\frac{e}{e_o}\right)^{\gamma-1} + \chi_o\right] \tag{19}$$

where $\chi_o = (s/s_{eo})^{-\gamma}$. s_{eo} and λ_{po} are the values of s_e and λ_p at the reference void ratio e_o. γ was taken at 0.55. Using (18) and (19) and the form of SWRC at e_o as the reference response, Mašín (2010) was able to predict evolution of a range of soils with a change in void ratio with remarkable accuracy.

Khoshghalb and Khalili (2013) and Pasha et al. (2017) extended Mašín's work to include hydraulic hysteresis effect, and provided an alternative relationship for the evolution of λ_p with e as

$$\dot{\lambda}_p = -\lambda_p \frac{3(1-\gamma)\left(2^{\frac{(1-\gamma)}{\lambda_p}}-1\right)}{2e}\dot{e} \tag{20}$$

Using the fractal distribution of particle size distribution Khoshghalb et al (2015) in turn showed that the evolution of s_e with e can be represented by

$$\dot{s}_e = \frac{\alpha s_e}{e D_s{}^{-1}} \dot{e} \quad (21)$$

which is identical to equation (18). In equation (21), α is the grain shape factor which assumes a value of one for circular particles and a value greater than one for other shapes. D_s is the fractal dimension of the particle size distribution. A similar approach was presented by Russell (2014).

3 CONSTITUTIVE MODELLLING OF UNSATURATED SOILS

Many constitutive models have been proposed over the past five decades to describe the nonlinear hydro-mechanical behaviour of unsaturated soils. The early work primarily focused on the applicability of the effective stress equation to describe the volume change and shear strength characteristics of unsaturated soils. Bishop and Blight (1963) obtained strong experimental evidence for the validity of the effective stress in unsaturated soils by performing a series of "null tests" and demonstrating that the shear strength and volume change characteristics remain unchanged when the individual components of the effective stress are altered but in such a way that the effective stress remained constant. However, Jennings and Burland (1962) questioned the validity of the effective stress in unsaturated soils arguing that it cannot explain the collapse phenomenon upon wetting. They conducted a series of consolidation tests on several unsaturated soils, and showed that upon flooding, i.e. a reduction in suction, all samples collapsed, rather than expanding as is implied by the effective stress principle. Similar arguments were also put forward by Aitchison (1965), Matyas and Radhakrishna (1968), Brackley (1971), Fredlund and Morgenstern (1977) and Gens et al. (1995), among others, leading to many investigators concluding that the effective stress is not applicable to unsaturated soils, not fully appreciating what the role of the effective stress is (explained in section 2.3.1).

Fredlund and Morgenstern (1977) suggested that the constitutive behaviour of unsaturated soils can be described using two independent stress variables, namely net stress (the total stress in excess of pore air pressure) and matric suction (pore air pressure in excess of pore water pressure), rather than a single effective stress. Several advanced constitutive models were developed within this framework, including the contributions of Alonso et al. (1990), Wheeler and Sivakumar (1995), Cui et al. (1995), Cui and Delage (1996), Vaunat et al. (2000), Rampino et al. (2000), Georgiadis et al. (2005), Thu et al., (2007), Sheng et al. (2008a,b). Indeed, most of the early advanced constitutive models for unsaturated soils were predominantly based on the two-stress state approach.

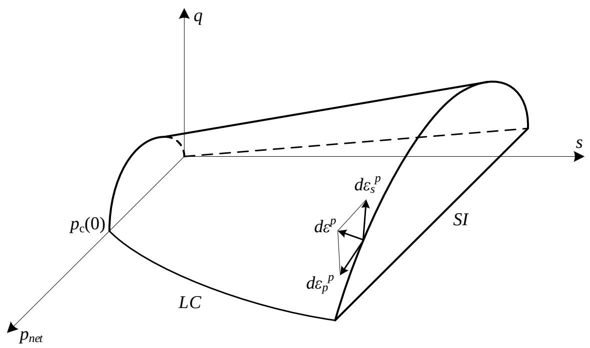


Figure 1. Barcelona Basic Model (BBM).

However, in the past two decades, there has been a shift towards use of the effective stress approach in constitutive models for unsaturated soils. Examples include Kohgo et al. (1993), Modaressi and Abou-Bekr (1994), Bolzon et al. (1996), Hutter et al. (1999), Loret and Khalili (2000), Karube and Kawai (2001), Loret and Khalili (2002), Wheeler et al. (2003), Gallipoli et al. (2003), Laloui et al (2003), Sheng et al. (2003, 2004), Tamagnini (2004), Santagiuliana and Schrefler (2006), Li (2007a,b), Sun et al. (2007a,b), Samat et al. (2008), Khalili et al. (2008), Buscarnera and Nova (2009), Coussy et al. (2010), Sheng and Zhou (2011), Zhang and Ikariya (2011), Kikumoto et al. (2011), Zhou et al. (2012a,b, 2018), Buscarnera and Einav (2012), Dangla and Pereira (2014), Hu et al. (2015), Zhou and Sheng (2015), Bellia et al. (2015), Lei et al. (2016), Mun and McCartney (2017), Lloret-Cabot et al. (2013, 2017), Bui et al. (2017), Zhao et al. (2019), Zhang et al. (2019), Moghaddasi et al. (2021), Phan et al. (2021), Ghorbai et al. (2016, 2021a,b), Komolvilas et al. (2022). It is increasingly recognised that previous arguments against the use of the effective stress were false and were invariably based on a linearly equivalent elastic theoretical framework, which cannot be used to explain non-recoverable volumetric responses such as collapse. As pointed out by Loret and Khalili (2000), even in saturated soils, it would be difficult, if not impossible, to explain irrecoverable volumetric deformations such as dilation and/or collapse (i.e. in metastable structures) in terms of the effective stresses alone without invoking an appropriate plasticity model. Other stimuli for the use of the effective stress have been: i) the mechanical response of soils to changes in total stress, pore water pressure and pore air pressure can be related to a single stress variable, rather than two or three independent stress state variables, ii) both saturated and unsaturated states of the soil can be treated using a single constitutive framework, which is particularly useful when considering the cyclic response of variably saturated porous media such as embankments and slopes, or when the boundary between saturated and unsaturated states is subject to significant fluctuations.

3.1 *Two Stress State Constitutive Models*

The two stress state variables constitutive models are formulated by adopting the stress state variables: net stress, $\boldsymbol{\sigma}_{net} \equiv \boldsymbol{\sigma} - p_a \boldsymbol{I}$, and suction, $s \equiv p_a - p_w$, as stress drivers of the mechanical response. With this approach a fully coupled two-phase hydro-mechanical elasto-plastic model is required to capture the mechanical response of the solid skeleton to variations in $\boldsymbol{\sigma}_{net}$ and s. In addition, the approach requires determination two sets of material parameters: one corresponding to $\boldsymbol{\sigma}_{net}$ and the other relating to s.

Alonso et al (1990) were first to develop a comprehensive elasto-plastic constitutive model for unsaturated soils within the two stress state variables framework. Commonly referred to as the Barcelona Basic Model (BBM), the model assumed isotropic plasticity rules, similar to those used in Cam-clay type models. The effective stress was replaced by net stress and an additional soil skeleton volumetric strain component was expressed in terms of matric suction. The yield surface was expressed in the mean net stress, p_{net}, deviatoric stress, q, and matric suction, s, space as

$$q^2 - M^2(p_{net} + p_s)(p_c - p_{net}) = 0 \quad (22a)$$

with

$$p_s = ks \quad (22b)$$

where $p_c(s)$ is the apparent isotropic preconsolidation pressure at the suction value of interest, p_s is the cohesion intercept, k is a material constant. A graphical representation of the yield surface is presented in Figure 1. At $s = 0$, the cohesion intercept, p_s, reduces to zero, and the modified Cam-clay ellipse is recovered in the $q - p_{net}$ plane. The yield surface in the $s - p_{net}$ plane was defined using two sets of yield curves, referred

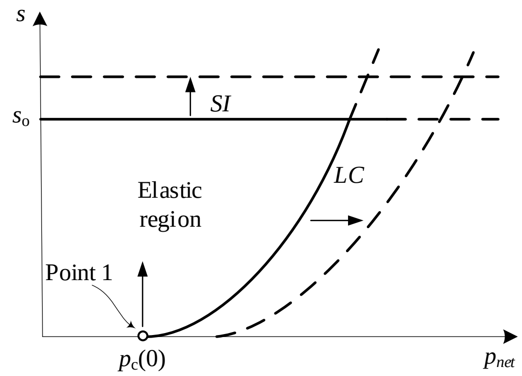


Figure 2. Load-Collapse (LC) and suction increase (SI) yield curves.

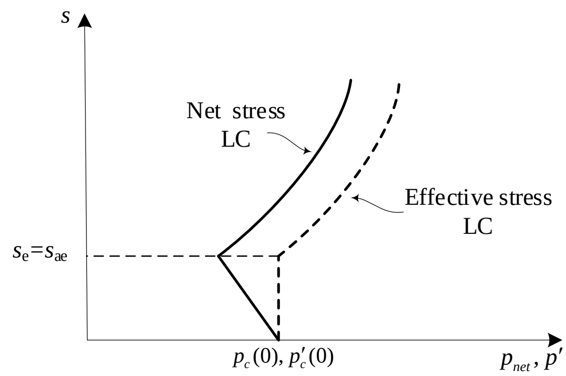


Figure 4. LC curves in effective stress and net stress planes.

to as the load-collapse (LC) curve and the suction increase (SI) yield locus (Figure 2). The contribution of matric suction to plastic behaviour was captured through the LC curve, and a suction-dependent apparent cohesion, represented by the intercept of the yield surface with the deviator stress axis (22b). The LC curve represented the shift in the preconsolidation pressure of the soil with unsaturation or suction and, for the first time, enabled modelling of collapse upon wetting - one of the key characteristics of unsaturated soil behaviour. The LC curve was one of the most fundamental and innovative aspect of BBM. It was mathematically expressed as

$$\left(\frac{p_c(s)}{p_o}\right) = \left(\frac{p_c(0)}{p_o}\right)^{[\lambda(0)-\kappa]/[\lambda(s)-\kappa]} \tag{23}$$

in which $p_c(0)$ is the isotropic preconsolidation at zero suction, and $p_o < p_c(0)$ is the reference stress at which one reaches the saturated normal compression line, starting from an unsaturated compression line; i.e. though a wetting path that only involves elastic swell (see Alonso et al. 1990). $\lambda(s)$ and κ are the slopes of the normal compression line at the suction of interest and reload and unload line in $v - \ln p_{net}$ plane (Figure 3). κ was assumed to be independent of suction for convenience, even though there was experimental evidence to the contrary (Alonso et al. 1990).

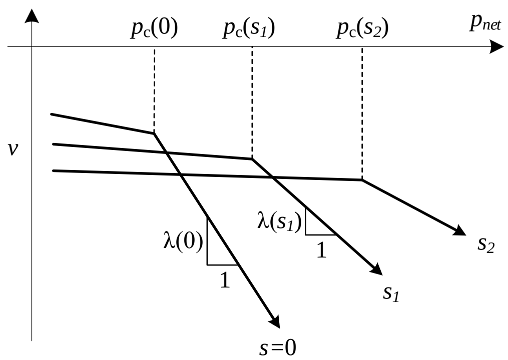


Figure 3. Evolution of normal compression line with suction - BBM.

Undoubtedly, BBM was a major step forward in the constitutive modelling of unsaturated soil, and has formed the basis for virtually all subsequent two stress state constitutive models proposed for the behaviour of unsaturated soils, e.g. Josa et al. (1992), Gens and Alonso (1992), Alonso et al. (1999a), Wheeler and Sivakumar (1995), Wheeler (1996), Cui and Delage (1996), Vaunat et al. (2000), Wheeler et al. (2002), Chiu and Ng (2003), Benatti et al. (2013) among others. Nevertheless, it was formulated based on limited experimental data (Alonso et al. 1987), and inevitably included assumptions/simplifications which were restrictive. Some aspects are discussed below:

- The most obvious restriction in the model is due to the assumption that unsaturation commences from s=0, hence omitting the essential role of s_e in separating saturated from unsaturated states in a soil. While an unsaturated soil is always associated with a pore water that is in suction, not all soils with a suction are unsaturated. A classic example is a saturated heavily over consolidated soil subjected to undrained shearing, which generates negative pore pressure during shearing without being unsaturated.
- The form of the LC curve assumed in BBM, see Figure 2, violates well established principles of saturated soil mechanics. To elaborate, we note that below s_e the soil is saturated, and unsaturation only occurs when $s > s_e$. We also note that for a saturated soil, suction has no influence on the preconsolidation pressure, p'_c. Hence, for the LC curve to be valid, p'_c must remain constant in the range $s = [0, s_e]$, before increasing with suction at $s > s_e$ (Loret and Khalili 2000). This is represented by the dashed line in Figure 4. Now, invoking the validity of effective stress principle for saturated soil, the net stress representation of the LC curve must therefore follow a slope of -1:1 until $s = s_e$, before increasing with $s > s_e$, as depicted b y the solid line in Figure 4. It is noted that this will result in a non-convex yield surface in the BBM.

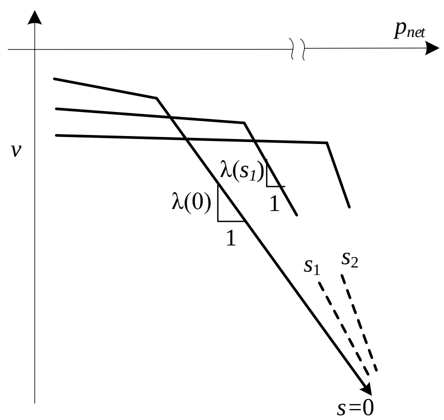


Figure 5. Convergent evolution of normal compression lines with suction.

- Central to the suction hardening model presented in BBM is the assumption that the slope of the isotropic compression line $\lambda(s)$ decreases with increasing suction (Figure 3).

Divergent normal compression lines at different values of suction are physically inadmissible; i.e. at high stresses, the normal compression lines for saturated and unsaturated soils must converge rather than diverge (Figure 5). To explore this further, consider two identical samples of the same soil with the initial void ratio of e_o. One sample is initialised to $p_{net} = 100$ kPa and $s = 0$, and the other to $p_{net} = 100$ kPa and $s = 300$ kPa. Now, both samples are subjected to a $p_{net} = 500$ MPa at constant suction. It is evident that the final void ratio of the samples will be identical. The experimental evidence also suggests that λ(s) increases slightly with suction, with a tendency to converge at high stresses (Figure 6). It is noted from that for $\lambda(s) > \lambda(0)$,

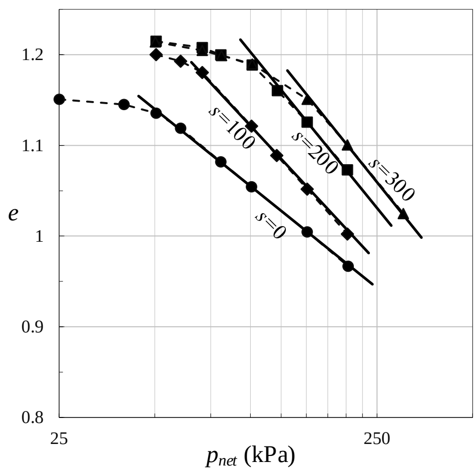


Figure 6. Convergent normal compression lines with suction (data from Wheeler and Sivakumar 1995)

equation (23) predicts softening of soil skeleton with suction rather than hardening. Also, for the case of $\lambda(s) = \lambda(0)$, applicable to most practical problems, the model predicts no suction hardening, i.e. $p(s) = p(0)$. Both outcomes are contrary to the physics of unsaturated soils. It is also recognised that the divergent normal compression lines suggest increasing collapse potential with p_{net}, which again is not supported by experimental evidence (Gens et al. 2006).

- Another novel feature in the BBM is the introduction of the suction increase (SI) locus. This implies that a normally consolidated soil, located at Point 1 in Figure 2, subjected to suction loading at constant p_{net}, will initially undergo elastic straining followed by an elasto-plastic response once the suction exceeds a previously attained maximum, s_o (Figure 7). In fact, experimental evidence (e.g. Fleureau et al. 1993) demonstrates exactly the opposite (Figure 8). As is shown, a normally consolidated soil subject to increasing suction will initially exhibit an elasto-plastic response until the point of air entry, $s = s_{ae}$, beyond which, the soil enters the elastic region, marked by a drastic reduction in the rate of volume change with suction. This is a characteristic feature of all collapsible unsaturated soils. The existing experimental data also suggest that once an unsaturated soil enters the elastic region, increasing suction will only expand the extent of the elastic domain and irreversible strains can no longer develop due to mere application of suction. The existence of SI as a limit of elasticity for the mechanical response of the soil is therefore highly questionable.

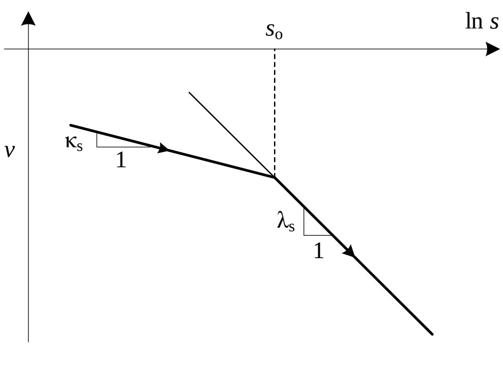


Figure 7. Mechanical response of a normally consolidated soil subjected to increasing values of suction – BBM.

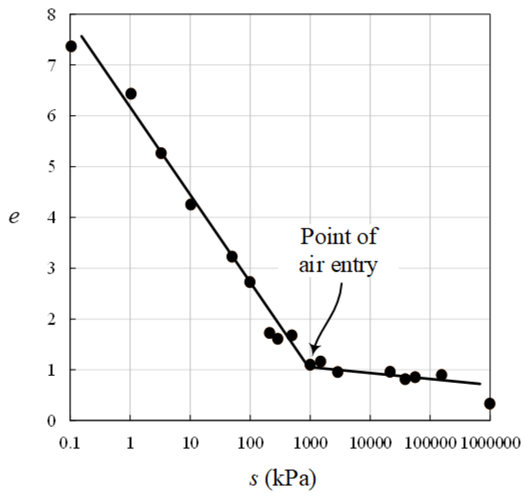


Figure 8. Mechanical response of a normally consolidated soil subjected to increasing values of suction – experimental data (data from Fleureau et al 1993).

- In the BBM, the contribution of suction to the shear strength of the soil at the critical state is exclusively captured through the cohesion intercept, p_s, which is assumed to be a linear function of suction though the material constant k. It is well understood that the shear strength in unsaturated soils is a strongly nonlinear function of suction, and that equation 22b is unlikely to capture the deviatoric response of unsaturated soils, particularly at the critical state. It is also noted that the assumption of k being constant is akin to assuming that the effective stress parameters χ is independent of suction/degree of saturation.
- Finally, for a two stress state model to be complete, it is necessary that strain conjugates are defined for each of the state stresses along with the corresponding elasto-plasticity elements such as the elastic model, the yield surface, the flow rule, the hardening model, etc. BBM is silent with respect to the elasto-plasticity elements associated with the hydraulic response, and its cross coupling with the mechanical model.

Josa et al. (1992) modified the BBM using non-linear relations for the variation of void ratio such that collapse strains diminished with net stress before reducing to zero at high stresses. Wheeler and Sivakumar (1995) adopted a similar approach to Alonso et al. (1990) but assumed an associative flow rule and defined the slope of the CSL in the deviator stress-mean net stress plane as a function of matric suction. Wheeler et al. (2002) attempted to extend the application of (23) to convergent normal compression lines by requiring that the compression lines at different values of suction passed a single point at high stresses. This may not be physically justifiable. This constraint also implied that $p(s)$ could be obtained by an elastic cut through the normal compression lines in the $v - \ln p_{net}$. While such an approach is entirely valid in the $v - \ln p'$ plane, it is not correct in the net stress plane as it ignores the stress path taken by the soil due to the suction increase (for a correct approach, see Alonso et al. 1990). Cui and Delage (1996) extended BBM to include anisotropy by using an inclined ellipse as a yield function and a non-associated flow rule. Wheeler (1996) and Dangla et al. (1997) included the elasto-plasticity associated with the hydraulic model. The coupling of the hydraulic and mechanical models was also attempted by Vaunat et al. (2000), Wheeler et al. (2003) and Sheng et al. (2008a,b,c). Sheng et al. (2008a,bc) adopted the correct form of the LC curve. The resulting non-convexity of the yield surface was addressed through an explicit integration scheme adopting small perturbations of strain and suction (Sheng et al 2008c). Despite these attempts, many of the two stress state models proposed in the literature retain all the original features of BBM without alteration.

3.2 *Effective Stress Constitutive Models*

In the effective stress approach, a single effective stress, $\boldsymbol{\sigma}'$, rather than two or three independent stresses, is introduced as the plasticity driver. The approach is similar to that of saturated soils except that an additional hardening parameter is introduced to allow for the expansion of the elastic region with unsaturation.

As discussed previously, the key advantage of the effective stress approach is that it permits capturing the *mechanical* response of the solid skeleton, both in the elastic as well as elasto-plastic regions, independent of the hydraulic model, and solely based on $\boldsymbol{\sigma}'$. In this approach, there is no need for the introduction of suction, s, as an additional independent stress variable, as the *stress* effect of s with respect to mechanical response is *a priori* accounted for in $\boldsymbol{\sigma}'$. Another important advantage of the effective stress approach is that the hydraulic model is reduced to the water retention response of the soil. These simplify the constitutive relationships of unsaturated soil, and significantly reduce model parameters.

In the effective stress approach, the transition between saturated and unsaturated states is seamless and the model parameters are exactly the same of those used in saturated soils except for to two entities that can be determined in any soil physics laboratories. More importantly, the need for testing in an unsaturated state is significantly diminished. Testing soils in an unsaturated state is time consuming, requires sophisticated laboratory equipment and expertise, and has been one of the key inhibitors of the use of unsaturated soil mechanics in practice. Finally, many of the restrictions of BBM, which are related to the choice of stress variables, do not apply to the effective stress models for unsaturated soils.

Using the effective stress framework, the yield surface for an unsaturated soil is simply defined as $f = f(\boldsymbol{\sigma}', \xi)$ where ξ is the parameter controlling the size of the yield surface, determined from the physics of the problem. The plastic potential is defined as $g = g(\boldsymbol{\sigma}', \zeta)$, where ζ controls the size of the plastic potential, determined by requiring g to pass through the current stress point, $\boldsymbol{\sigma}'$. Similar to saturated soils, the flow rule emanates from the normality rule applied to the stress strain conjugates $(\boldsymbol{\sigma}', \boldsymbol{\varepsilon})$ and plastic potential g.

Adopting the modified Cam-clay model as the plasticity driver, and invoking the associativity of the flow rule, the yield function and the plastic potential for an unsaturated soil in the $q - p'$ plane is expressed as

$$f(\boldsymbol{\sigma}', p'_c) = q^2 - M^2 p'(p'_c - p') \tag{24}$$

This is identical to the model for saturated soils (Figure 9), except that p' is quantified using (5) and the preconsolidation pressure, $p'_c(\varepsilon_p^p, \Lambda)$, is made a function of plastic volumetric strain, ε_p^p, and the hardening parameter, Λ. The hardening parameter, Λ, captures the stiffening effect of suction/unsaturation on the soil's solid skeleton. Inspired by the work of Alonso et al. (1990), the unsaturation hardening parameter is typically taken as $\Lambda = s$

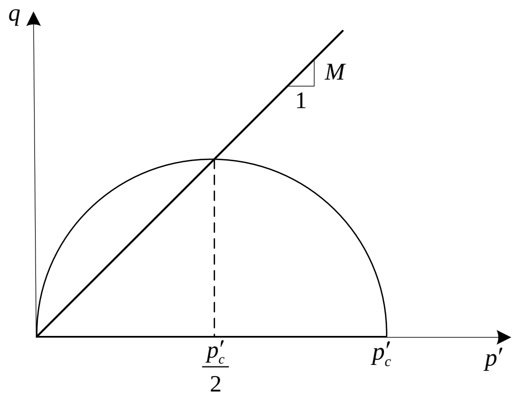


Figure 9. Effective stress model for unsaturated soil - Modified Cam-Clay model.

(e.g., Kohgo et al. 1993, Modaressi and AbouBekr 1994, Loret and Khalili 2000, Khalili and Loret 2001, Khalili et al. 2004, Sheng et al. 2003, Eberhardsteiner et al. 2003, Borja 2004, Tamagnini and Pastor 2004, Sheng et al. 2004, Ehlers et al. 2004, Russell and Khalili 2006, Mašín and Khalili 2008). However, more recently, it is shown that $\Lambda = S_r$ may be a more suitable hardening parameter, since it naturally captures the effect of hydraulic state and the hysteresis on the stiffening of the soil skeleton (e.g. Gallipoli et al. 2003, Tamagnini 2004, Zhang et al. 2007a,b, Gallipoli et al. 2008, Zhou et al. 2012a,b, Komolvilas and Kikumoto 2017, Song and Khalili 2019, Moghaddasi et al. 2021).

It is instructive to reiterate that even when Λ is taken as s, it enters the plasticity model has a hardening parameter rather than a stress variable. The role of s, in this regard, is formally identical to that of temperature and cementation in constitutive modelling of soils, even though the physical effect of temperature is opposite to that of suction.

Kohgo et al. (1993) and Modaressi and AbouBekr (1994) were amongst first to present an elasto-plastic model for unsaturated based on the effective stress, accounting for the dual effect of suction in increasing the effective stress and the preconsolidation pressure. Bolzon et al. (1996) used Bishop's effective stress for casting the elasto-plastic equations and introduced the suction hardening effect directly into the plastic modulus. As a results they were unable to predict collapse upon wetting. This was rectified by Santagiuliana and Schrefler (2006). Loret and Khalili (2000) proposed a comprehensive framework for constitutive modelling of unsaturated soils. This framework was formulated considering a three-phase porous medium and the effective stress principle. The work presented a fully coupled flow-deformation model taking into account the elasto-plasticity of the solid skeleton as well as the air and water phases within the system. The key features of this model were: the continuity of behaviour at the transition between saturated and unsaturated states; incorporation of air entry suction directly into the formulation as the demarcation point between saturated and unsaturated states; incorporation of suction in the yield surface as the hardening parameter; the use of the soil water characteristic curve in determination of coupling between the water and air phases, and capturing the effect of plasticity on the coupling of flow and deformation fields. Qualitative predictions of the model were shown to produce characteristic features of unsaturated soils such as collapse upon wetting, and plastic followed by elastic response of normally consolidated soils during drying. Loret and Khalili (2002) developed an elaborate elasto-plastic constitutive model for unsaturated soils using as an extension of the Cam-clay plasticity model. In this work, both the elastic behaviour and the failure surface at critical state were defined in terms of the effective stress, and the stiffening effect of suction observed in many experiments (Alonso et al. 1990; Wheeler and Sivakumar 1995) was captured using a simple hardening model. Suction was incorporated into the effective stress using the relationship established by Khalili and Khabbaz (1998). The model adopted an associated flow rule with a split elliptical function for the yield surface and plastic potential. The slope of the critical state line in the deviatoric stress-mean effective stress plane was assumed to be a material constant. The model required a minimal number of material parameters. Loret and Khalili (2002) used this model to provide good predictions of the Wheeler and Sivakumar (1995) data. Adopting the same framework Russell and Khalili (2006) proposed a unified bounding surface model for sand and clays for monotonic loading of unsaturated soils. Khalili et al. (2008) extended the theoretical developments of Loret and Khalili (2000) to include mechanical as well as hydraulic hysteresis effects within the context of the bounding surface plasticity.

Other effective stress based models presented in the literature are due to Lewis and Schrefler (1998), Jommi (2000), Sheng et al. (2003), Wheeler et al. (2003), Gallipoli et al. (2003), Laloui et al. (2003), Tamagnini (2004), Sun et al. (2003, 2007a, b), Mašín and Khalili (2008), Morvan et al. (2010), Wong et al. (2010), Manzanal et al. (2011), Lloret-Cabot et al (2013, 2014, 2017) , Bellia et al. (2015), Lai et al. (2016), Ghorbani et al. (2016), Zhang et al. (2019), Ghaffaripour et al. (2019), Bruno and Gallipoli (2019), Fabbri et al. (2019), Sitarenios and Kavvadas (2020), Moghaddasi et al. (2021).

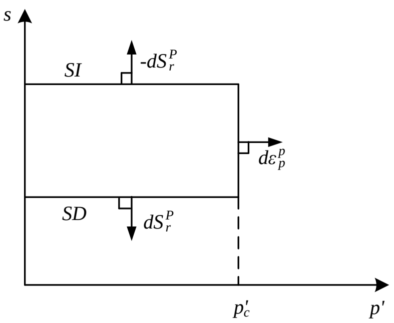


Figure 10. Elasto-plastic hydro-mechanical model (after Wheeler et al. 2003).

As shown by several researchers, irrecoverable deformations such as collapse can be readily captured within an effective stress framework by introducing suction or degree of saturation as a hardening parameter in the defining the yield surface (Kohgo et al., 1993; Modaressi and AbouBekr, 1994; Loret and Khalili, 2000; Khalili and Loret, 2001; Khalili et al., 2004; Russell and Khalili, 2006; Mašín and Khalili, 2008; Moghaddasi et al., 2021).

Wheeler et al. (2003) presented an elasto-plastic model based on three yield surfaces, a yield surface representing mechanical behaviour (LC curve) and the other two (SI and SD) representing water retention behaviour of soil (Figure 10). These three surfaces were coupled through dependency of the LC curve on suction and shift in the SWRC with volume change or void ratio (see section 2.4). The model was originally developed for 1D isotropic loading, and later extended to the general stress space by Lloret-Cabot et al. (2013, 2017). Sheng et al. (2004) extended Wheeler's model to deviatoric loading and proposed a more general model of hydraulic hysteresis, cast into a classical theory of elasto-plasticity. In contrast to the previous models, the SI, SD and LC yield surfaces were allowed to move independently of each other and non-associative flow rules were defined for SI and SD surfaces. Variations to Sheng's model were proposed by Sun and his colleagues (i.e. Sun et al., 2007a,b, 2008). Tamagnini (2004) extended the Cam-clay model for unsaturated soils including the hydraulic hysteresis effects. The model was based on the effective stress concept using $\chi = S_r$. The constitutive relationships of Romero and Vaunat (2000) were adopted for the definition of hysteretic soil water retention curves (SWRCs). In the model, the evolution of the yield surface was governed by plastic volumetric strain as well as degree of saturation.

Common to the above models is that the hydraulic behaviour of soils is captured through an elastic-plastic formulation involving definition of yield surfaces (SI and SD), evolution law and flow rule. Such a formalism is rarely warranted for one-dimensional phenomena such as the water return behaviour. The models proposed essentially use the SWRC to obtain the elastoplastic parameters of the hydraulic model in order to reproduce the same SWRC, albeit in an approximate way. A more prudent and usual approach will be to introduce SWRC as a constitutive input into the three-phase model. Complete examples can be found in Loret and Khalili (2000), Khalili et al. (2008), Komolvilas and Kikumoto (2017) and Moghaddasi et al. (2021). In such an approach, the void ratio dependency of SWRC is treated as a material nonlinearity, without invoking an elaborate elasto-plasticity model.

Sołowski and Sloan (2012) proposed a different approach for constitutive modelling of unsaturated soils through introducing the concept of equivalent stresses. A normalised yield locus was introduced that did not depend on suction; i.e. the change in suction did not lead to a change in the yield locus size. They stated that the equivalent stress concept may be used to quickly extend models for saturated soils into unsaturated regime. As mentioned by Sołowski1 and Sloan (2015), the applicability of this approach to describe the complexities associated with the behaviour of unsaturated soils during wetting and drying cycles, as well as during loading and unloading is not clear and needs further investigation.

3.3 *Cyclic Plasticity Models for Unsaturated Soils*

Conventional elasto-plastic constitutive models fail to describe soil behaviour during load reversals. This has, over the years, prompted the need for enhanced constitutive models capable of correctly describing the soil response subjected to cyclic loading. Among the various cyclic plasticity models available in the literature, kinematic hardening and bounding surface plasticity models have been shown to provide a convenient framework to model a number of observed aspects of cyclic behaviour of soils such as hysteretic response, memory of past stress history, smooth degradation of stiffness during loading, small strain stiffness and early onset of plastic strains (Gajo and Muir Wood, 2001). Bounding surface plasticity has attracted a great deal of attention due to its simplicity and efficiency in modelling cyclic behaviour. The advantage of the bounding surface formulation is that it is geometric in nature with little appeal to the physical reasoning of the problem. Yet, it has been shown to accurately capture the cyclic response of soils and provide a smooth transition from elastic to elasto-plastic behaviour. The method is computationally simple, uses fewer model parameters and results of the simulation fit experimental data with reasonable accuracy.

Khalili et al. (2008) proposed a fully coupled flow deformation model for the cyclic analysis of unsaturated soils including hydraulic and mechanical hystereses. The model was developed within the context of the bounding surface plasticity using the critical state theory and the effective stress approach. An important feature of model was that at any stress point two directions of plastic stain increment vector are defined (Khalili et al. 2005). This was shown to be essential to capture volumetric behaviour of the soil during unloading and re-loading on the dry side of the critical state line. A void ratio-dependent water retention model was adopted, taking the effect of hydraulic hysteresis into account.

Liu and Muraleetharan (2012a,b) proposed a coupled hydro-mechanical constitutive model in the general stress space for unsaturated sands and silts under both monotonic and cyclic loading conditions. The model was the extension of the isotropic elastoplastic model of Muraleetharan et al. (2009). They considered the hysteretic properties of the SWRC within a bounding surface plasticity framework for the mechanical behaviour. The stress driver of plasticity was assumed to be the intergranular stress defined as $\boldsymbol{\sigma}' = \boldsymbol{\sigma} - p_a\boldsymbol{I} + n^w(p_a - p_w)\boldsymbol{I}$, where n^w is the water volume fraction. They allowed for the effect of suction on hardening of the bounding surface through the irrecoverable water content calculated from the SWRC model. Pedroso and Farias (2011) as well as Cui and Zhao (2017) extended the BBM for the cyclic analysis of unsaturated soils. Two yield surfaces were defined in order to provide a smooth transition from elastic to plastic behaviour. Oka and Kimoto (2012, 2022), Oka et al. (2019), and Shahbodagh (2011) proposed a cyclic elasto-viscoplastic constitutive model for the dynamic analysis of unsaturated soils. The model was developed within the context of the overstress theory and the effective stress concept. The nonlinear kinematic hardening rule, softening due to the structural degradation of soil particles, and suction hardening were taken into account in the model. The suction hardening equation proposed by Cui and Delage (1996) for unsaturated silts was adopted in the model. Shahbodagh et al. (2015) proposed a numerical model based on the theory of multiphase mixtures for nonlinear large deformation dynamic analysis of unsaturated porous media including hydraulic hysteresis. In the model, the coupling between solid and fluid phases was enforced according to the effective stress principle allowing for the suction dependency of the effective stress parameter. The effect of hydraulic hysteresis on the effective stress parameter and soil water characteristic curve were taken into account in the model. They showed that the effect of hydraulic hysteresis could markedly alter the response of an unsaturated soil during dynamic loading that will invariably involve complex cycles of strain-induced wetting and drying. Zhou et al (2015) developed a bounding surface model for unsaturated soils under cyclic loading conditions. The approach proposed by Gallipoli et al. (2003) was adopted to capture the evolution of bounding surface with suction. Three bounding surfaces are defined in the model: one describing elastoplastic behaviour during compression; one describing elastoplastic behaviour during shearing, and one expressing elastoplastic behaviour as suction changes. Komolvilas and Kikumoto (2017) developed a cyclic elastoplastic model for the analysis of liquefaction in unsaturated soils. The model was the extension of the modified Cam clay model. It was based on the subloading surface concept and the effective stress approach. The state boundary surface was defined as a function of the degree of saturation. The soil water characteristic curve adopted considered the effects of specific volume and hydraulic hysteresis. Gholizadeh and Latifi (2018) developed an effective stress-based hydro-mechanical model for unsaturated soils under cyclic loadings. Two separate mechanisms were adopted for the mechanical behaviour of the material, i.e. the conventional plasticity for isotropic loading and a multi-yield surface plasticity framework for deviatoric loading. Xiong et al. (2019) proposed an elastoplastic constitutive model for unsaturated soil under monotonic and cyclic loading. The model was developed based on the work of Zhang et al. (2007a,b) and Zhang and Ikariya (2011). The concept of stress-induced anisotropy proposed by Zhang et al. (2007a,b) was adopted in the model to capture the cyclic response of soil. Bishop's effective stress and degree of saturation were used as state variables in the framework. The superloading and subloading concepts were introduced in the model to consider the influence of overconsolidation and structure on deformation and strength of soils. Cao et al. (2021) proposed a constitutive model for unsaturated soils subject to high-cycle traffic loading. The model was developed based on the BBM and the shakedown concept. Cai et al. (2022) proposed a fractional-order bounding surface model for unsaturated soils under cyclic loading with constant matric suction. The effective stress and matric suction are used as the constitutive variables. The effective stress parameter proposed by Khalili and Khabbaz (1998) was adopted in the model. The movable mapping centre rule was used to describe the hysteresis characteristics of the dynamic stress–strain curve. Ghasemzadeh and Ghoreishian Amiri (2013) proposed an elastoplastic constitutive model for unsaturated soils under isotropic loading conditions. The model was developed based on the elasto-plastic framework of Muraleetharan et al. (2009), considering the effect of hydraulic hysteresis. Bounding surface and subloading surface plasticity frameworks were employed to describe hydraulic and mechanical behaviour, respectively. The model was later extended by Ghasemzadeh et al. (2017) for general stress states. Kaewsong et al. (2019) proposed a constitutive model for unsaturated soils within the framework of bounding surface plasticity. The model was the extension of the bubble model proposed by Al-Tabbaa and Wood (1989). An elliptical elastic bubble was defined inside a modified Cam-clay bounding surface. The size of the elastic bubble was modelled as a function of suction, degree of saturation, and plastic volumetric strain. The model was shown to be capable of capturing the effects of recent suction history on nonlinear stress–strain relation and shear modulus degradation at small strains. Bruno and Gallipoli (2019) proposed a bounding surface model to describe the behaviour of unsaturated soils under isotropic loading. The model was based on the hydraulic law of Gallipoli et al. (2015) and the mechanical law of Gallipoli and Bruno (2017). The bounding surface theory was used for modelling both hysteretic water retention and mechanical behaviour of soil. Ghorbani et at. (2021a) also adopted a similar bounding surface approach but included the effect of soil anisotropy.

3.4 *Constitutive Models for Unsaturated Expansive Soils*

The mechanical behaviour of expansive soils, also referred to as reactive soils, is influenced by not only the mechanical loading and matric suction, but also the physico-chemistry of the soil or osmotic suction. A range of constitutive models have been proposed in the past three decades for the behaviour of expansive soils mainly based on an extension of the BBM and the experimental observations on compacted bentonite in relation to buffer materials in waste disposal facilities. Of particular interest has been the irrecoverable (plastic) swell observed in expansive soils due to wetting at low confining pressure.

The most notable contribution on this topic was due to Gens and Alonso (1992) who proposed a double structure model for the mechanical behaviour of unsaturated expansive clays, within the framework of BBM (Alonso et al., 1990). The key elements of the model were: i) the mechanical behaviour of the macrostructure followed the BBM, ii) the micro-structure was always saturated and elastic, and iii) micro-structure was unaffected by macro-structure, but the deformation of the microstructure was able to affect the macrostructural level. In particular, it was assumed that elastic microstructural "*swelling will affect the soil skeleton through increasing its macro-structural void ratio. This plastic volume change leads in turn to a movement of the LC to the left (a softening in hardening plasticity terms) in response to the new structural arrangement.*", and iv) the ratio of macro-structure plastic strain to elastic swell of micro-structure was directly proportional to the over-consolidation ratio of the soil, attaining a value of zero for a normally consolidated compacted clay.

This framework was further refined in the Barcelona Expansive Model (BExM) by Alonso et al. (1999a), where the deformations of two structure levels (micro-structure and macrostructure) were elaborated and the behaviour of micro-structure was extended into the unsaturated domain, characterised by using the effective stress principle. This modelling framework was subsequently improved and extended by Sánchez et al. (2005) and Navarro et al. (2014), and integrated into numerical simulation codes to solve coupled boundary value problems (Sánchez et al., 2005, 2008).

There are several fundamental difficulties associated with the conceptual model underpinning the BExM-type constitutive models:

- As shown by Khalili et al. (2010) and Mašín (2013) an elastic, isotropic, self-similar expansion of the micro-structure will not lead to a change in the macro-structural configuration of the soil. The volume change of the soil will simply be in the form of a magnification of the macro-pore structure and the associated micro-structural grains. This cannot lead to a change in the macro-mechanical behaviour and softening as was envisaged in the model,
- All volume changes emanating from an underlying elastic deformation (i.e. elastic expansion of the micro-structure) by definition must be reversible. It is unusual to associate a plastic straining with an elastic phenomenon without invoking other processes,
- The function defining the plastic coupling of the macro-structure to the elastic straining of the micro-structure is based on limited mechanical reasoning and appears to serve only a fitting role,
- The existence of double structure is not specific to compacted bentonite. It is also present in compacted kaolin. However, kaolin does not show an expansive behaviour,
- The expansive behaviour of clays is heavily influenced by their physico-chemical properties of the soil and such effects cannot be captured using net stress and suction alone, and
- The macro-structural model is based on BBM without alteration.

Alonso et al. (2011) and Gens et al. (2011) extended the BExM double structure model to include the hydro-mechanical coupling mechanisms. However, they ignored the dependency of water retention behaviour on the volumetric deformation in their models. This issue was later addressed by Della Vecchia et al. (2013) who developed a fully coupled hydro-mechanical model within the double structure framework. This model was an extension of the model by Romero et al. (2011). Mašín (2013) proposed a hypoplastic framework for double structure hydromechanical modelling of unsaturated expansive clay in an extension of Mašín and Khalili (2008). Fully coupled hydromechanical models were presented for each structural level. The effective stress representation by Khalili and Khabbaz (1998) was applied at both micro- and macro-structural levels. Void ratio dependency of water retention properties at the two structural levels were also considered. Li et al. (2017a,b) presented a constitutive model for unsaturated expansive clays, adopting the conceptual model of Baker and Frydman (2009) to extend the work input approach for the constitutive modelling of double structure unsaturated materials. In this model, the bounding surface concept was employed for modelling the mechanical behaviour.

In addition to the double structure approach, some researchers have adopted a single structure/porosity approach for modelling the behaviour of expansive unsaturated soils. Sun and Sun (2012) presented a model for unsaturated expansive soils, with a focus on the coupled hydro-mechanical behaviour. This model takes into consideration the coupled effects of degree of saturation and void ratio on the mechanical and water-retention behaviour of soil. Li and Yang (2017) proposed a constitutive model for the hydro-mechanical response of unsaturated expansive soils through introducing the concept of a macro-structural neutral loading line. The concept was incorporated into the unsaturated model of SFG (Sheng et al., 2008a) to derive a volume change equation for unsaturated expansive soils. Takayama et al. (2017) extended the elasto-plastic constitutive model of Ohno et al. (2007) to unsaturated expansive soils. Cui et al. (2002) developed a non-linear elastic model to predict the volume change behaviour of swelling clays with dense structure.

Physico-chemical effects in the constitutive modelling of expansive soils have been considered by Liu et al. (2005), Zhang and Zhou (2008), Lei et al. (2016), Guimarães et al. (2013), and Chen et al. (2016) to name a few. Liu et al (2005), within the BBM framework, proposed a chemo-mechanical unsaturated model following the work of Hueckel (1997) for saturated soils. Zhang and Zhou (2008) extended Hueckel's work to unsaturated expansive soils but they adopted an effective stress approach. Lei et al. (2016) used the effective stress approach to extend the chemo-mechanical model of Loret et al. (2002) using the general plasticity framework of Loret and Khalili (2002). In this model, a generalised effective stresses expression was introduced incorporating the effects of pore water chemistry. They related the mechanical properties to the evolution of molar fraction of absorbed water as well as matric suction as softening/hardening parameters. Guimarães et al. (2013) extended BExM to incorporate the chemical effects. They adopted BBM for the behaviour of macrostructure and defined a modified effective stress expression, including the osmotic suction for the microstructure. Chen et al. (2016) utilised theory of mixtures to developed a constitutive model for coupled hydro-chemo-mechanical analysis of unsaturated highly swelling materials within the framework of non-equilibrium thermodynamics. The work included the combined effects of chemical osmosis and hydration swelling, assuming passive air pressure.

In summary, a great deal of work has been presented on the mechanics of unsaturated expansive soils in the past three decades. Even so a number of clear gaps remain in the knowledge. Some of the questions that require attention include: Is the soil structure the only driver of expansive behaviour? Should physico-chemical effects be considered in the constitutive modelling of expansive soils? What are the appropriate state parameters with respect to the physico-chemistry of expansive soils? What are the influences of osmotic suction/chemical potential on the strength of reactive soils and how will the microstructural alterations influence the response? (The current experimental evidence is inconclusive.) What are the combined effects of osmotic and matric suctions on the effective stress of the soil, if any, and how can these be quantified? What is the appropriate scaling parameter for capturing the contribution of osmotic suction for the effective stress of the soil skeleton? How does pore water chemistry alter the soil stiffness and strength and to what extent? What is the impact of repeated cycles of osmotic loading and unloading and physico-chemical hysteresis on the mechanical response and strength of reactive soils? These, as well as many other unresolved, yet important, fundamental questions will require concerted and systematic research effort.

3.5 *Constitutive Models for Unsaturated Aggregated, Cemented, Fissured Soils*

The hydro-mechanical and constitutive behaviour of unsaturated soils can be significantly affected by the presence structure such as aggregation, cementation and fissuring. Several constitutive models have been developed to capture the essential features of the unsaturated soils with structure (Khalili 2008, Yang et al. 2008, Borja and Koliji 2009, Cai et al. 2010, Pereira et al. 2014, Le Pense et al. 2016, Bruno et al. 2020, Moghaddasi et al. 2021).

Khalili (2008) presented a comprehensive hydro-mechanical model for the three-phase analysis flow-deformation analysis of fissured porous media. The formulation consisted of three separate, yet overlapping models: the deformation model, flow of air model and flow of water model. The deformation model was cast in the effective stress space satisfying the equations of equilibrium. The flow model was based on the theory of double porosity (Khalili and Valliappan 1996). The coupling between the air and water flow was established through the water retention curve. The coupling between the fluid flows and deformation was established through the effective stress parameters. Borja and Koliji (2009) presented a double porosity elasto-plastic hydro mechanical model for aggregated unsaturated porous media based on the mixture theory. The work adopted the effective stress principle, which was derived based on the continuum principles of thermodynamics for multi-phase double porous media. Following the classical theory of plasticity, the yield function was defined in terms of the effective stress but adopting plastic internal variables and local suction in the two scales of porosity as the hardening parameters. Yang et al. (2008) proposed a bounding surface constitutive model incorporating the combined effects of unsaturation and the initial structure. The model was suited for monotonic loading of structured soils with a fixed projection centre for the mapping rule. Cai et al. (2010) presented a binary model for cemented unsaturated soils. The cementation behaviour was considered elastic where as that due to unsaturated elasto-plastic. Pereira et al. (2014) presented a constitutive model for volumetric analysis of cemented/ structured unsaturated soils. The model was formulated in the effective stress space, with suction, plastic volumetric strain and degree of structure as drivers of isotropic hardening/softening. It was shown that the model could predict the maximum collapse due to wetting and loss of structure. Le Pense et al. (2016) presented a hydro-mechanical constitutive model for clayey soils accounting for damage-plasticity couplings as well as hardening effects due to suction and plastic volumetric strain. A double effective stress incorporating both the effect of suction and damage was defined based on thermodynamical considerations. Coupling between damage and plasticity was achieved by using strain equivalency and the use of the double effective stress into plasticity equations. Bruno et al. (2020) presented a bounding surface model predicting the combined effects of cementation and partial saturation on the mechanical behaviour of soils subjected to isotropic loading. The loss of cementation caused by loading, wetting or drying of a normally consolidated soil was described through a cementation bonding function which monotonically decreased with increasing stress. Moghaddasi et al. (2021) proposed a fully coupled hydro-mechanical bounding surface elasto-plastic model for describing the behaviour of unsaturated structured soils. The hydraulic characteristics of structured soil were captured through a void ratio-dependent hysteretic water retention model formulated based on the effective stress principle. The effects of structural degradation and the degree of saturation on the compressive and tensile strength of the material were considered through controlling the size of the bounding surface, allowing for a smooth transition of the response from structured to unstructured states. A plastic work hardening approach was adopted to consider the effects of stress magnitude and accumulated plastic strain on the degradation process.

3.6 *Constitutive Models for Rate-Dependent behaviour of Unsaturated Soils*

Rate dependency is an important element in the mechanical behaviour of geomaterials for capturing creep, creep-induced failure and the rate of loading on the material response. Nevertheless, many of the constitutive models for unsaturated soil have been constructed within the framework of the rate independent theory.

Oka et al. (2006) were amongst first to develop elasto-viscoplastic constitutive model for unsaturated soils accounting for the effect of unsaturation hardening. A fully coupled three-phase hydro-mechanical model was presented adopting the advantage of the effective principle within the overstress-type viscoplasticity framework. Oka and Kimoto (2012, 2022), Oka et al. (2019) and Shahbodagh (2011) extended this model to allow elasto-viscoplastic dynamic analysis of unsaturated soils. The nonlinear kinematic hardening rule, softening due to the structural degradation of soil particles, and suction hardening were taken into account. The suction hardening equation proposed by Cui and Delage (1996) for unsaturated silts was adopted in the model. De Gennaro et al. (2009) proposed a rheological model for partially saturated chalks, including strain rate effects by means of an extended strain rate-dependent hardening law, albeit restricted to isotropic loading condition. Zou et al. (2013) developed a rheological creep model for unsaturated soils, where time was included explicitly to describe the viscoplastic strain. A difficulty with this approach was that any application of the model requires the definition of an origin for the time which cannot be defined in an objective way. De Gennaro and Pereira (2013) developed a viscoplastic constitutive model for time-dependent behaviour of unsaturated geomaterials using BBM as the reference elastoplastic framework. The plastic mechanism associated with the SI surface was ignored in their model. They used the isotache approach and defined a strain rate dependent hardening law to extend the BBM. Bui et al. (2017) developed a viscoplastic damage model for partially saturated rocks. They obtained the constitutive relations by decomposing the inelastic strains into viscoplastic strain and strain due to micro-cracking. Separate handling of viscoplastic deformations from damage deformations is, however, questionable since only the combined effects can be measured (Zienkiewicz and Cormeau, 1974).

3.7 *Modelling of Strain Localisation in Unsaturated Soils*

Strain localisation, or shear banding, is the concentration of deformation in a narrow zone due to intense shear straining. It is the precursor to failure and can occur in a range of materials including alloys, metals, plastics, polymers, granular materials and soils. In unsaturated porous media, a breakdown in interfacial effects; e.g. due to shearing or water infiltration, is shown to reduce strength and cause pronounced shear banding (Cui and Delage 1996, Cunningham et al. 2003, Higo et al. 2013).

Using the multiphase mixture theory, several approaches have been proposed for the analysis of strain localisation in unsaturated porous media. Schrefler et al. (1996, 2006) and Zhang et al. (1999, 2007) presented a formulation for dynamic strain localisation in porous media with two fluid phases based on an extension of Biot's formulation assuming passive air pressure. They studied the characteristics of the natural length scale which exists in the multiphase models due to the seepage process inducing a rate-dependent behaviour for the soil mixture. Borja (2004) developed a mathematical framework, based on Cam-clay plasticity theory, to detect the inception of strain localisation in partially saturated porous media under plane strain compression at the constitutive level. Ehlers et al. (2004 , 2011), Shahbodagh (2011), Lazari et al. (2015), Oka et al. (2019) and Oka and Kimoto (2012, 2022) developed computational models for capturing quasi-static and dynamic strain localisation in multiphase elasto-viscoplastic porous media. The material rate-dependency considered was shown to eliminate the numerical instability and introduce a natural length-scale into the problem, avoiding the need to perform a diagnostic analysis for the onset of localisation. Schiava and Etse (2006) investigated the potential of bifurcation in partially saturated soils under uniaxial and plane strain loading conditions at constant suction. Callari et al. (2010) studied, numerically, the response of a perfectly homogeneous soil under plane strain compression and showed that the strain localisation can be triggered by a heterogeneous effective stress state induced by fluid flow coupling from the early stages of testing. Buscarnera and Nova (2011) investigated the instability of unsaturated soils under triaxial compression using the controllability approach proposed by Nova (1994). Perić et al. (2014) used the bounding surface model proposed by Khalili et al. (2008) and derived analytical solutions for the onset of strain localisation in unsaturated soils under undrained, constant water content, and drained loadings. Borja et al. (2013), Song and Borja (2014), and Song et al. (2017) studied the influence of initial heterogeneity in unsaturated porous media, with spatial varying density and degree of saturation, on the inception of localisation. In many of these models, the degree of saturation has been adopted as the effective stress parameter. More recently, Song and Khalili (2019) proposed a peridynamics model, developed within the context of the effective stress concept, for strain localisation analysis of multiphase geomaterials at constant suction.

3.8 *Anisotropic Models for Unsaturated Soils*

Unsaturated soils are commonly encountered in civil engineering practice, formed by compaction, sedimentation or weathering of rock, and invariability exhibit anisotropic behaviour. As pointed out Cui and Delage (1996), the microstructure of the soil changes during compaction in order to provide the greatest possible resistance to the applied stresses. This is similar to the effect of sedimentation on natural soft soils and results in the inclination of the yield curves indicating the anisotropy of the soil.

To capture the anisotropic behaviour of unsaturated soils, Cui and Delage (1996) presented a model for compacted silt and captured the effects of anisotropy by using an inclined ellipse as a yield function and a non-associated flow rule. Matric suction was introduced as an independent variable, contributing to the size of the yield surface. Stropeit et al. (2008) proposed an anisotropic elasto-plastic constitutive model for unsaturated soils by combining the features of BBM for unsaturated soils with anisotropy. This model was further enhanced by D'Onza et al. (2010) and Al-Sharrad and Gallipoli (2016) through linking the material anisotropy to both distortion and aspect ratio of the constant suction yield surface. More recently, Sutharsan et al. (2017) developed a critical state-based constitutive model for predicting the unsaturated response of sands. The model was an extension of the framework proposed by Heath et al. (2004). It used the bounding surface plasticity theory and considered the effect of fabric anisotropy. They introduced an anisotropic fabric parameter enabling the model to capture the effect of sample preparation on sand response. More recently, Sitarenios and Kavvadas (2020) proposed an elasto-plastic constitutive model for unsaturated anisotropic within the effective stress framework. The model was the extension of the modified Cam-clay model and included the soil anisotropy effects via rotation of the yield surface.

4 EXPERIMENTAL INVESTIGATION

4.1 *Preamble*

The difficulty in understanding the behavioural features of a wide range of materials found in geological environments, particularly under partially saturated conditions, has promoted the use of experiments and introduced a significant empirical load that has been extended to theoretical and conceptual developments (Alonso 2005). In this regard, the progress of unsaturated soil mechanics has been based not only on close contact with practical engineering applications but also on the development of conceptual frameworks, theories, and models, which rely on laboratory and in situ experiments.

Nevertheless, experimental soil mechanics should not be solely considered a simple tool for soil model calibration/validation and parameter estimation. Instead, laboratory tests should benefit from crucial advances in equipment and new techniques for conducting experiments and making observations. In this regard, experimental soil mechanics must reinvent itself and embrace other spaces, such as incorporating new experimental techniques and protocols, running multi-scale tests (from microstructural to physical model tests), better controlling multi-physics processes and boundary conditions, and improving the characterisation at lower structural levels (microstructural techniques).

Furthermore, what does experimental soil mechanics bring to the table when dealing with partially saturated soils? It can be argued that this field affords considerable advantages in the following aspects:

- Provides a closely coupled view of hydraulic and mechanical phenomena (multi-physics) where two or more fluids under different pore filling conditions coexist in a porous medium.
- Provides the need to approach phenomena at different scales (multi-scale viewpoint at pore/grain level, phenomenological scale, and upscaling to application).

In recent years, the expansion of geotechnical engineering applications, focusing on resilience to adapt to and mitigate climate change effects and the research interest in multi-physics processes is becoming broader (e.g., Gens 2010). There are two important points to highlight in this context of unsaturated soil mechanics opportunities. On the one hand, the importance of the progressive incorporation of reproducible laboratory techniques and well-posed design criteria in codes of practice. On the other hand, progress in these new areas requires advanced experimental techniques with new technologies. Accordingly, besides more traditional geotechnical and mining engineering applications (e.g., Alonso et al. 1987, Fredlund and Rahardjo 1993, Fredlund et al. 2012, Fredlund 2017, Tarantino and Di Donna 2019, Houston 2019, Fredlund 2019), experimental unsaturated soil mechanics is of value in different cross-disciplinary topics, which will be briefly described in the next section, focusing on the testing techniques.

In this section, we present laboratory techniques and experimental contributions in unsaturated soil mechanics over the last two decades relevant to studying the coupled hydro-mechanical and multi-scale phenomena that the different ground engineering and cross-disciplinary applications demand.

4.2 *An insight Into Experimental Techniques and Applications*

4.2.1 *Ground engineering*

A significant number of the published experimental research concerns compacted soils, which are affected by changes in water content. Since Proctor's pioneering work on these materials (Proctor 1933), used in the construction of transport and hydraulic infrastructures (earth embankments/dams), a constitutive and experimental framework has been required to study the hydro-mechanical behaviour of these partially saturated soils at different compaction states (Gens 1995, Tarantino and De Col 2008, Kodikara 2012, Leroueil and Hight 2013, Tatsuoka 2015, Tatsuoka and Gomes Correia 2018). The traditional way to approach this hydro-mechanical behaviour has been exploring different initial states in the Proctor plane (dry unit weight vs water content) and mapping the different engineering properties to the compaction state, such as permeability and water retention properties, stiffness, shear strength, and collapse/swelling on wetting (e.g., Lawton et al. 1989, Vanapalli et al. 1999, Santucci de Magistris and Tatsuoka 2004, Jotisankasa et al. 2009). Nevertheless, advances in experimental unsaturated soil mechanics have required incorporating new constitutive law formulations and numerical models to characterise the initial 'as compacted' state (but not water content and maximum dry unit weight as in the conventional Proctor plane) (Gens et al. 1995, Sivakumar and Wheeler 2000, Della Vecchia et al. 2013). On the one hand, water contents should be transformed into matric suction (considering void ratio effects) through the adequate branches of water retention curves, which pose specific experimental difficulties and display significant hysteresis during drying and wetting (Tarantino and Tombolato 2005, Romero et al. 2011). On the other hand, the attained dry unit weights by compaction should be related to the maximum compaction/preconsolidation stresses (yield stresses) that vary with suction (Alonso et al. 2013). Indeed, the microstructure set up during compaction is also essential information for the initial state, which shapes engineering properties (Gens et al. 1995, Delage et al. 1996, Thom et al. 2007, Monroy et al. 2010, Romero 2013, Alonso et al. 2013). Microstructural experimental techniques will be discussed below (section 4.3.4).

In addition, compacted/emplaced materials span a much wider range of grain size distributions. In this respect, the compacted soil concepts can also be extended to coarser materials such as rockfill material, which is relevant to dams and embankments (Oldecop and Alonso 2003, Romero et al. 2012a, Oldecop and Alonso 2013, Alonso et al. 2016). The similarity in the collapse response on wetting these materials with contrasting particle sizes and differentiated mechanisms also provides a marked multi-scale character to unsaturated soil mechanics.

Besides compacted soils, many geotechnical aspects of partially saturated soils may be observed in swelling and residual soils in arid regions, which are often unsaturated and structured. Geotechnical applications with significant ground-atmosphere interactions can be found on:

- Plastic clays susceptible to swell/shrink during hydraulic processes affecting foundations/retaining walls, pavements and railway infrastructures and their remediation (Nelson et al. 2015, Vahedifard et al. 2015, Soltani et al. 2019).
- Residual tropical lateritic soils and saprolites (rock weathering), alluvial/colluvial/aeolian deposits and volcanic agglomerates prone to volume change on wetting (Leroueil and Barbosa 2000, Benatti et al. 2011, El Mountassir et al. 2011, Kholghifard et al. 2014, Ng et al. 2020).
- Soils susceptible to crack during environmental desiccation or affected by vegetation, such as flood protection embankments (Atique et al. 2009).
- Soils prone to salt accumulation under arid climate conditions (De Carteret et al. 2014).
- Materials prone to degradation on wetting/drying cycling (Fityus et al. 2005, Cardoso et al. 2012, Romero et al. 2014, Pineda et al. 2014a, Pineda et al. 2014b, Stirling et al. 2021).
- Soils affected by rainfall-induced landslides (Lim et al. 1996, Alonso et al. 1999b, Alonso et al. 2003, Springman et al. 2003, Askarinejad et al. 2012, Sorbino and Nicotera 2013, Song et al. 2016, Leong and Ng 2016, Jeong et al. 2017).
- Partially saturated soils influenced by cryogenic suction changes at different ice contents (frost heave and thaw settlements, effects of cyclic freezing and thawing, soil freezing water retention curve) (Pelaez et al. 2014, Caicedo 2017, Mao et al. 2018, Teng et al. 2019).
- Coupled effects generated by soil-groundwater-biological systems-atmosphere interactions (e.g., Cui et al. 2013, Tang et al. 2018) that will be later introduced.
- The effect of degree of saturation on electrokinetic remediation of unsaturated soils (Gabrieli et al. 2008, Yin et al. 2022).
- Settlements of partially saturated loose soils susceptible to densification caused by earthquake-induced cyclic stresses (EN 1998-5:2018) and the dynamic response of partially saturated soils (Pandya and Sachan 2018).
- Rammed earth materials (unstabilised or stabilised with cement/lime or fibrous materials) (Jaquin et al. 2009, Corbin and Augarde 2014, Beckett et al. 2018).
- Capillary barrier systems to prevent water infiltration (Rahardjo et al. 2012, Rahardjo 2015, Scarfone et al. 2020).Influence of entrapped air bubbles in quasi-saturated soils (influence of the compressibility of the pore fluid, water retention and permeability at nearly saturated conditions) (Sakaguchi et al. 2005, Jommi et al. 2019).

As shown by these applications, the effects of matric (or total) suction, degree of saturation and stress changes are crucial for correctly understanding the response of these partially saturated materials. Nevertheless, the difficulty of controlling and measuring matric (or total) suction has been one of the reasons for the developmental delay in experimental unsaturated soil mechanics compared to saturated soils with positive water pressure (Delage et al. 2008).

The first classification of experimental techniques for controlling the amount of water in soils would be:

- Soaking and air-drying tests.
- Constant water content tests.
- Controlled degree of saturation experiments and
- Controlled-suction tests, either transporting liquid water (control of matric suction) or vapour (control of total suction).

The following will briefly outline these experiments. The soaking or fast inundation tests are suitable for studying the effects of sudden changes observed on rapid flooding, such as collapse/swelling (ASTM D4546-21, EN 1997-2:2007) and shear strength loss on wetting (Nicotera 2000, Alonso et al. 2016). Alternatively, partial soaking can also be carried out to

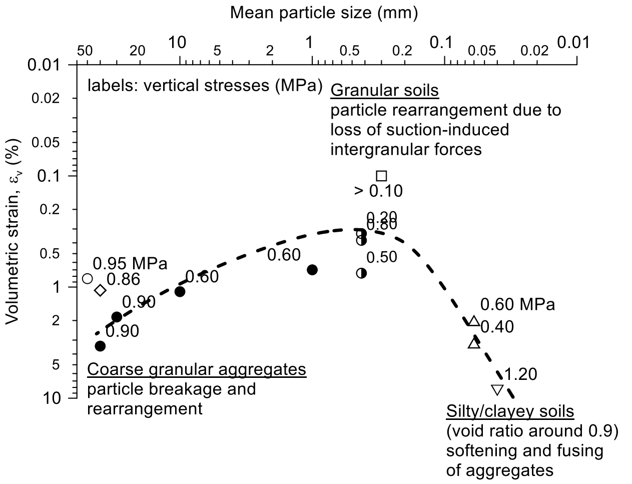


Figure 11. Collapse on soaking for different materials and their different mechanisms.

achieve a degree of saturation of less than one. Other tests have focused on degradation and swell/shrinkage accumulation on cyclic wetting and drying (Pineda et al. 2014a, Alonso et al. 2005). All these tests should be complemented with data of water retention curves on the corresponding branches (wetting or drying) and preferably dependent on porosity. As an example of the use of this rapid flooding procedure, Figure 11 shows the collapse response on soaking at different stresses and starting from hygroscopic conditions (relative humidity around 50%), and encompassing different materials from silty/clayey soils to coarse granular aggregates. These tests are straightforward to study the volume changes on soaking at different mean particle sizes, as well as to discriminate the different phenomena associated with these collapsible phenomena: softening of aggregates and reduction of inter-aggregate pore space in clayey soils, softening of silty/clayey bridges between grains, particle rearrangements due to loss of suction-induced intergranular forces in sands, and particle breakage sensitive to the state (activity) of the fluids and further skeleton rearrangement on coarser granular aggregates (Nara et al. 2012). In addition, another practical application of these fast-soaking tests is the possibility of isolating microstructural effects induced on compaction at a constant suction (the one achieved on saturation), since it can be assumed that the microstructure set on compaction is preserved along this rapid process (Santucci de Magistris and Tatsuoka 2004).

The constant water content tests are appropriate to study the compressibility on loading/unloading and the shear strength properties at specific hydraulic conditions (Tarantino and De Col 2008, Della Vecchia et al. 2011, Wijaya and Leong 2016). These tests are performed on conventional laboratory equipment with systems that prevent vapour loss under air-drained conditions. Nevertheless, the preferred option is improving the cells and measuring suction during these mechanical processes (Tarantino et al. 2011). In case it is not possible to measure the suction locally, the tests should be preferably carried out in the low water content range, in which water retention curves are not sensitive to porosity variations (Romero et al. 2011). Typical (gravimetric) water content ranges are usually below 20% in highly plastic clays, below 14% in clays and 8% in clayey silts to avoid noticeable suction changes during compression and shearing (Romero 2013). In this case, the results should be complemented with water retention curve data on the corresponding branches (wetting or drying) in this high-suction domain. In the light of the development of techniques for suction measurement, which are described in section 4.3.2, these tests should progressively be included in standards for unsaturated soil testing relevant to ground properties and geotechnical design. For example, shear strength envelopes with friction angle and total cohesion depending on suction and degree of saturation - or water content - at failure (Vanapalli et al. 1996).

Another useful alternative, although more challenging to implement, is the controlled degree of saturation test. These experiments are suitable for studying the significant role of saturation, besides suction, on the compression and shear strength response. For example, test results can be helpful to study the evolution of soil compressibility as it approaches asymptotically the saturated compression line, as they allow evaluating the collapse on soaking at different vertical stresses and detecting the maximum collapse zone (collapse strains that increase with confining stress and then decrease). This last issue has been discussed in models in which the plastic compressibility index depends on the degree of saturation (Zhou et al. 2012a,b, Della Vecchia et al. 2013, Alonso et al. 2013). These loading tests are performed by automatically controlling the changes of pore-water volume $\mathrm{d}V_w$ and pore volume space $\mathrm{d}V_v$, while monitoring the pore-water pressure beneath a porous interface (Al-Badran 2011, Burton et al. 2016). These volume changes are related through $\mathrm{d}V_w - S_{r\,ini}\mathrm{d}V_v = 0$, where $S_{r\,ini}$ is the constant degree of saturation. Critical issues are matric suction equalisation at a constant loading rate since water percolates through the low-permeability interface (Burton et al. 2017) and controlling the evaporation of water in the open-air pressure system as described below.

Moreover, finally, controlled-suction tests with relative humidity regulation (to control total suction) or control of air/water pressures using a porous interface permeable to the wetting fluid to set the capillary mechanism of matric suction. These techniques are valuable for studying the gradual effects of progressive wetting and drying stages and representing stress paths for calibrating and validating constitutive models. They should be complemented with water content data (or degree of saturation) throughout the different stages. These controlled-suction techniques will be discussed in section 4.3.1.

4.2.2 *Environmental and energy geotechnics*

Experimental unsaturated soil mechanics is also involved in many geoenvironmental and energy geotechnical applications, encompassing engineered barriers, sealing materials for industrial waste dumps, capillary barriers, landfill seal covers to protect the environment, energy geo-structures, soil pollution and remediation, and resilience to adapt to and mitigate climate change effects in geotechnical infrastructures. The common thread is to understand the coupled thermo-hydro-chemo-mechanical processes, in which experimental unsaturated soil mechanics is essential to broaden the knowledge of these highly coupled phenomena. Topics are linked to transporting fluids and contaminants and water exchanges between the ground and the atmosphere. At the same time, other phenomena are related to heat transfer and chemical effects that present significant interactions with the mechanical response of the materials. For example, the increase in temperature generated by the waste can induce cracking due to desiccation in systems open to the atmosphere affecting their functionality (Zornberg et al. 2003), changes in swelling capabilities on wetting (Romero et al. 2003, 2005a), and an increase in the water permeability (Romero et al. 2001).

Table 1. Selected tests performed on different bentonites (engineered barriers).

MX-80 type:	Swelling
Villar 2005	Compressibility
Tang & Cui 2005	Water retention curve
Karnland et al. 2007	Water permeability
Herbert et al. 2008	Temperature effects
Tripathy et al. 2015	Chemical effects
Molinero-Guerra et al. 2019	Shear stiffness
Pintado et al. 2019	
Mesa-Alcantara et al. 2020	
MX-80 type/sand:	Water retention curve
Wang et al. 2013	Water permeability
Tabiatnejad et el. 2016	Microstructure
	Compressibility
	Chemical effects
FEBEX:	Swelling
Lloret et al. 2003	Compressibility
Villar & Lloret 2004	Water retention curve
Villar 2005	Water permeability
Castellanos et al. 2008	Temperature effects
Romero et al. 2005a	Chemical effects
GMZ:	Swelling
Ye et al. 2013	Water retention curve
Ye et al. 2014	Water permeability
Chen et al. 2015	Thermal effects
Chen et al. 2017	Chemical effects
He et al. 2019	Microstructure
Czech B75:	Water retention curve
Sun et al. 2019	Thermal effects
Sun et al. 2020	Microstructure
FoCa clay:	Swelling
Imbert & Villar 2006	
Kunigel V1 and Kunigel V1/sand:	Swelling
Komine 2004	Compressibility
Romero et al. 2005b	Shear strength
Yamamoto et al. 2019	Gas permeability

Experimental mechanics has been valuable in studying engineered barriers for radioactive waste disposal. These barrier materials are emplaced under partially saturated conditions and subjected to hydration. Bentonite-based materials have been extensively studied in this context, due to their low permeability and their swelling potential upon hydration, which allows closing the engineering gaps. Therefore, studies have concentrated on water retention properties, water and air permeability under partial saturation, compressibility on loading at different suctions, compressibility on suction changes, small-strain shear stiffness and strength, and temperature and chemical effects on hydraulic and mechanical properties. Table 1 summarises selected references for different bentonite types used in engineered barriers.

The already mentioned aspects are also valid when implemented in cover systems for landfills and evapotranspirative ET covers with a reduced desiccation and cracking response. The quantification and measurement of evapotranspiration, i.e. the flow boundary condition at the soil-atmosphere interface, is of interest to these ET cover systems (Cui and Zornberg 2008). Experimental unsaturated soil mechanics is also relevant to geosynthetic clay liners GCLs. However, it encounters some experimental challenges, as explained in Bouazza et al. (2013). GCLs and their particular geometric and structural configuration have required modifying conventional laboratory techniques for measuring/controlling suction. Abuel-Naga and Bouazza (2010) reported a complete review of the existing techniques and their suitability while developing a modified triaxial apparatus that controls the water content and measures the suction. Different authors have also investigated GCL's water retention capacity using different techniques (some references are summarised in Table 2) and the effects of water salinity (Lu et al. 2018).

Table 2. Selected experimental techniques in geosynthetic clay liners.

Reference	Experimental technique
Southen & Rowe 2004	Pressure plate technique
Agus & Schanz 2005	Non-contact filter paper method, psychrometer (dew point chilled mirror) technique, relative humidity sensor
Barroso et al. 2006	Filter paper
Southen & Rowe 2007	Pressure plate (axis translation technique)
Abuel-Naga & Bouazza 2010	Thermocouple psychrometer, capacitive relative humidity sensor
Beddoe et al. 2010	High capacity tensiometers, capacitive relative humidity sensors
Hanson et al. 2013	Pressure plate, filter paper, relative humidity
Bannour et al. 2014	Osmotic technique with polyethylene glycol, vapour equilibrium technique
Rouf et al. 2014	Vapour equilibrium technique
Acikel et al. 2015	Contact filter paper
Rouf et el. 2016	Vapour equilibrium technique
Seiphoori et al. 2016	Dew point chilled mirror psychrometer technique, non-contact filter paper method
Acikel et al. 2018	Dew point chilled mirror psychrometer technique
Acikel et al. 2020	Modified osmotic technique with polyethylene glycol

Energy geo-structures, such as energy piles, diaphragm walls and tunnels, use the ground, sometimes under partially saturated conditions, for heating/cooling structures and heat storage (e.g., McCartney et al. 2016, Vitali et al. 2021). Therefore, the thermal properties and the temperature-induced changes in the hydro-mechanical response are key aspects to consider. In such a context, a challenging issue is the effect of soil saturation on the heat exchange rate in the long-term response of these systems (Laloui et al. 2014). The degree of saturation influences the thermal properties of the ground and, therefore, the heat exchange rate, which affects the efficiency of energy piles (Akrouch et al. 2016) and can result in an average heat exchange rate 40% lower than saturated conditions (Choi et al. 2011, Aljundi et al. 2020). This fact evidences the importance of considering partially saturated conditions in selecting thermal soil parameters for the design (Vieira et al. 2017). Centrifuge tests involving partially saturated states on these thermal applications (foundation's movements during heating and cooling) have been performed by Stewart and McCartney (2014) and Goode and McCartney (2015).

The determination of the water retention behaviour is even more complex in multi-species systems, which is the case of soils contaminated with liquid organic compounds referred to as NAPLs (Non-Aqueous Phase Liquids). LNAPLs (Light NAPLs with lower density than the water) with reduced solubility in the water remain above the water table in the vadose zone. Therefore, experimental techniques on unsaturated soil mechanics can help better understand their flow properties, which are necessary for the proper performance of remediation techniques for polluted soils (Alferi 2011). Furthermore, the infiltration of DNAPL (Dense NAPLs with higher density than water) in saturated soil also concerns capillary interactions between water and DNAPL (Delage and Romero 2008).

A three-fluid system (air, water and oil) should be considered when dealing with contaminated partially saturated soils. It is usually assumed that the more wetting fluid (water) fills the smallest pores, followed by oil and non-wetting air occupying the largest ones (Leverett 1941, Lenhard and Parker 1988). Therefore, cells to apply the axis translation technique should be modified by treating the high air-entry value ceramic discs, depending on whether water flow is allowed (hydrophilic treatment) or water transfer is prevented (hydrophobic treatment with Glassclad®18 used in oil removal at constant water content) (Cui et al. 2003, Manassero et al. 2005, Rabozzi et al. 2006, Alferi 2011). Alferi et al. (2011) used a chilled-mirror psychrometer to determine total suction considering that no significant evaporation of Soltrol®170 Isoparaffin Solvent (LNAFL) occurred (Manassero et al. 2005).

Another multi-species application is the methane hydrate-bearing sediments (MHBSs). Natural MHBSs are highly susceptible to pressure and temperature changes (hydrate dissociation) (Dai and Santamarina 2013, McCartney et al. 2016), and synthetic MHBSs are usually considered to study their thermo-hydro-mechanical behaviour. In addition, the hydrate content alters the pore geometry, changing the capillary pressure-saturation relationship from the hydrate-free condition (Ghezzehei and Kneafsey 2010). Mahabadi et al. (2016) used synthetic tetrahydrofuran THF hydrates, which are stable under atmospheric conditions and miscible in water, to study the significant effects of non-wetting hydrate saturation on the water retention curves (gas entry pressure increase with increasing hydrate saturation). The water retention curves were studied with the axis translation technique, using a treated high air-entry value ceramic with a solution of ethylene glycol to avoid THF hydrate formation.

These different applications have posed significant challenges to using experimental techniques for unsaturated soils. Among these developments, the following aspects can be highlighted.

- The wetting stage of bentonite-based materials involves operating over a very high suction range (sometimes with an initial total suction of more than 200 MPa) towards saturation. This extended range has entailed a combination of vapour transfer techniques up to a total suction of about 4 MPa, followed by liquid transfer using matric suction control (Hoffmann et al. 2005).
- This extended range in bentonites has also required a combination of different sensors to measure suction (relative humidity probes and dew point psychrometers for total suction, and high capacity tensiometers for matric suction) (Cardoso et al. 2007, Toll et al. 2013, Mendes et al. 2019).
- The vapour transport must be efficient in bentonite-based materials, so forced convection techniques have been used by pumping humid air through the material (in case of a high air permeability) or laminating along the boundaries of the sample (Dueck 2004, Pintado et al. 2009a,b).

- In the case of using controlled-suction techniques on materials at high temperatures, care must be taken to avoid losing vapour through any open system (Romero et al. 2003).
- To separate the effects of osmotic and matric suctions in clayey materials, such as increased pore liquid concentration during drying at controlled relative humidity (Mata et al. 2002) and the osmotic suction changes at constant matric suction (Mokni et al. 2014).
- The high air-entry value ceramics used in the axis translation technique (three-fluid systems) should be treated to avoid certain phenomena, such as ice formation of THF or impeding water passage when controlling oil removal. Unfortunately, these treatments induce alterations in the air-entry properties of the ceramics (Alferi 2011).

4.2.3 *Bioinspired technologies*

Nature-based solutions and soil reinforcement techniques with a low carbon footprint are attracting increasing interest (UN 2030 Agenda). Recent trends in research in partially saturated soils involve the use of plants, fungi, and microbial/enzyme/urease induced calcite precipitation to improve the hydro-mechanical properties against erosion, landslides, soil collapse, or to favour soil bearing capacity (Vardon 2014, Ng et al. 2016a, Ng et al. 2019, Tiwari et al. 2021, Assadi-Langroudi et al. 2022). Biological systems (plants, fungi, bacteria) involved in these solutions are constantly evolving in time (root growth and decay, calcium precipitation and eventual dissolution) and space (root development), thus continuously generating changes to soil hydraulic properties (permeability, water retention, Morales et al. 2015a, Jotisankasa and Sirirattanachat 2017, Ni et al. 2019a), to soil structure (soil fissuring, pores and rock fractures clogging, Carminati et al. 2013, Minto et al. 2016, Terzis and Laloui 2018, Koebernick et al. 2017), to mechanical behaviour (Yildiz et al. 2018, Terzis and Laloui 2019, Fraccica et al. 2022a) and even to soil thermal properties (Venuleo et al. 2016, Oorthuis et al. 2021).

Soil-vegetation-atmosphere interactions (root water uptake, evapotranspiration, solar radiation, wind, rainfall, runoff, water infiltration) affect soil suction (Ng et al. 2016b, Ni et al. 2018), leading to different coupled phenomena, as depicted, and briefly explained in Figure 12. Laboratory investigations display a crucial role in these novel applications to provide a comprehensive understanding of the different processes involved and their future implementation in codes and regulations for nature-based solutions (e.g., Bo et al. 2015). However, experimentally studying partially saturated vegetated soils poses particular challenges to address. First, a correct laboratory sample size must be selected to obtain a representative elementary volume REV. In this regard, an adequate soil/root ratio should be ensured in laboratory samples (Fraccica et al. 2022a, 2022b, Yildiz et al. 2018, Switała et al. 2018) and consistent with vegetative species under natural conditions to better assess the soil-root hydro-mechanical behaviour. Fissures generated by biological systems and a sound volume of root architecture should be included (Li et al. 2009, Borges et al. 2018, Li and Shao 2020). Therefore, a large specimen size, which implies a more complex retrieval or preparation, is often required to deal with the issues mentioned above. If the soil sample is retrieved, a large and uncommon soil corer is required with consequences on the alteration of the sample. In situ testing seems an alternative solution to test large-scale vegetated samples (Comino and Druetta 2010, Cammeraat et al. 2005). However, attention has to be paid to minimum displacement rates allowed (usually too high), volume change assessment instrumentation and un-controlled/inhomogeneous hydraulic boundary conditions. Suppose the sample is prepared/compacted in the laboratory (Pallewattha et al. 2019, Fraccica 2019, Yildiz et al. 2018). In that case, attention should be paid to ensure water content/dry density homogeneity, facilitate root growth by leaving an adequate pore-space, and control the hydraulic path prior to testing (plant irrigation and root water extraction). This last aspect is fundamental to ensure repeatability, as an excessive matric suction (above that induced on compaction) generated by roots may induce irreversible shrinkage together with soil fissuring. A second open issue is the appropriate definition of void ratio and degree of saturation in vegetated soils. The different ways to compute void ratios depend on the stress state, either by clogging/stealing void volume at high stresses or opening new fissures at low-stress conditions. They include attributing the roots to the solid phase, to the gas phase or identifying them as a fourth (external) phase (Muir Wood et al. 2016). Therefore, different ways to formulate this variable have been used in literature, sometimes in an empirical way, and according to the specific results observed (Fraccica et al. 2019, Ng et al. 2016c, Ni et al. 2019b). The choice of a given void ratio formulation has direct consequences on the evaluation of other variables such as the degree of saturation and the formalisation of constitutive laws describing soil water retention, compressibility on loading and suction changes, and volume change upon shearing (Ng et al. 2016c, Ni et al. 2019b, Foresta et al. 2020, Karimzadeh et al. 2021). Techniques commonly used in soil mechanics to infer the void ratio, such as paraffin wax tests (ASTM D7263-21), should be complemented by microstructural techniques to explore potential changes generated by biological systems on soil pore-size distribution (Carminati et al. 2013, Scholl et al. 2014, Fraccica et al. 2019, Koebernick et al. 2017). Biological systems (roots and bacteria) are often generating suction and hydraulic gradients within the matrix due to biochemical and physical effects on soil (e.g., root-induced fissures, surfactants released in the matrix, soil water consumption by bacteria: Read et al. 2003, Carminati et al. 2010, Leung et al. 2015, Pham et al. 2016, Liu et al. 2020). These spatially heterogeneous and time-evolving modifications of the hydraulic state should be considered when evaluating soil water

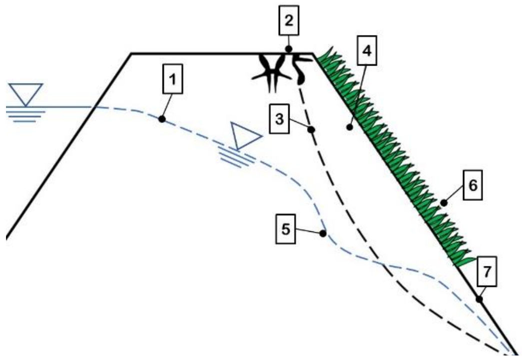


Figure 12. Coupled phenomena in soil-vegetation-atmosphere interactions. 1) Phreatic level affected by soil-atmosphere interactions (drying/wetting cycles). 2) Soil desiccation (shrinkage/cracking and effects on infiltration and landslide initiation). 3) Slope stability (soil cracking and vegetation effects on soil hydro-mechanical behaviour). 4) Vegetation features (root/soil ratios, depth, morpho-physiology) and vegetation growth/decay (water retention curve, permeability, microstructure, volume change and shear strength properties, carbon sequestration, mucilage chemical effect). 5) Vegetation-induced suction ('capillary barrier effect, mechanical strengthening, effect on slope stability). 6) Soil-vegetation-atmosphere interactions (evapotranspiration, leaf coverage). 7) Soil erosion and runoff velocity.

Table 3. Unsaturated hydraulic behaviour in vegetated and MICP improved soils.

Reference	Properties evaluated	Water content	Matric (max. value allowed in MPa) or total suction	Other indirect evaluations of WRC
Vegetated soil				
[2] Fraccica et al. 2019	WRC	Oven-check	[3] Ceramic tip tensiometer (0.20) [4] Dew-point psychrometer	Pore size distribution
[1] Ni et al. 2019a	WRC and Relative hydraulic conductivity	TDR moisture probe	[3] Ceramic tip tensiometer (0.10)	-
[1] Ni et al. 2019b	WRC	TDR moisture probe	[3] Ceramic tip tensiometer (0.10)	WRC model depending on Void ratio and root volume and decay
Jotisankasa & Sirirattanachat 2017 **	WRC and relative hydraulic conductivity	Mass check	[3] Ceramic tip tensiometer (0.10)	-
[1,2] Leung et al. 2015	WRC / suction profile /water balance during simulated rainfall and evapotranspiration	Back-analysis of infiltration columns *	[3] Ceramic tip tensiometer (0.08)	Numerical modelling of water and vapour flows
[2] Ng et al. 2014	Water balance / suction profiles during simulated rainfall	TDR moisture probe	[3] Ceramic tip tensiometer (0.10)	-
MICP improved soil				
Chen et al. 2021	WRC at different calcite content	Mass check	[3] Ceramic tip tensiometer (0.10)	WRC model depending on void ration and calcite content
Liu et al. 2020	WRC after different MICP treatment cycles	Mass check	[4] Dew-point psychrometer	-
Saffari et al. 2020	WRC at different bacterial concentrations	Mass check	[4] Filter paper (10)	-
Saffari et al. 2019	WRC at different bacterial concentrations	Mass check	[3] Matric paper (10)	-
Martinez et al. 2018	WRC	Oven-check	[3] Hanging-water-column (0.004)	-
Morales et al. 2015b	WRC	Mass check	[4] Dew-point psychrometer [3] Axis-translation technique	Pore size distribution

[1] under light conditions, [2] under darkness conditions
[3] matric suction, [4] total suction, WRC: water retention curve
*according to Ng & Leung 2012
** plant leaves trimmed to minimise transpiration

Table 4. Laboratory tests on vegetated and MICP improved soils.

Reference	Laboratory test	Sample size (mm)	[4]hydraulic variable w	s	S_r
Fraccica et al. 2022b	[2]Monotonic triaxial comp.	d: 200, h: 400	KM	M	E
Fraccica et al. 2022a	[2]Uniaxial extension	2×d: 100 h: 50	KM	M	E
Fraccica 2019	[2]Monotonic triaxial comp.	d: 200 h: 400	KM	M	E
Mahannopkul & Jotisankasa 2019	[2]Direct shear	d: 140 h: 150	KM	M	-
Tan et al. 2019	[2]Direct shear	d: 61.8 h: 20.0	KM	-	-
Pallewattha et al. 2019	[2]Direct shear	300×300×200	M	M	E
Yildiz et al. 2018	[2]Inclinable direct shear	500×500×300	M	M	E
Switała et al. 2018	[2]Multi-stage direct shear	250×250×110	M	M	E
Gonzalez-Ollauri & Mickovski 2017	[2]Direct shear	d: 49.0 h: 23.3	M	M	-
Li et al. 2017	[2]Monotonic triaxial comp.	d: 61.8 h: 125.0	M	-	-
Veylon et al. 2015	[2]Direct shear	500×500×300	M	E	E
Zhang et al. 2010	[2]Monotonic triaxial comp.	d: 39.1 h: 80.0	KM	-	-
Comino & Druetta 2010	[1,2]Direct shear	300×300×100	M	-	-
Cammeraat et al. 2005	[1,2]Direct shear	[2]60×60×20 [1]600×600×400	-	-	-
Tiwari et al. 2021	[3]Unconfined comp., split tensile strength	h/d ratio = 2	M	-	*
Mujah et al. 2019	[3]Unconfined comp.	h/d ratio = 2	-	-	-
Vail et al. 2019	[3]Desiccation tests	d: 50 h: 6.4	M	-	-
Mahawish et al. 2018	[3]Unconfined comp.	d: 100 h: 200	-	-	-
Cardoso et al. 2018	[3]Oedometer, Brazilian splitting	d: 70 h: 20	-	-	*
Bahmani et al. 2017	[3]Unconfined comp.	d: 60 h: 120	-	-	E
Cheng et al. 2017	[3]Unconfined comp.	h/d ratio = 1.5÷2.0	M	-	-
Terzis et al. 2016	[3]Triaxial comp.	d: 50 h: 100	M	-	-
Morales et al. 2015a	[3]Direct shear, oedometer, resonant column	d: 60 h: 20	M	M	(soaking during test)
Morales et al. 2015b	[3]Unconfined comp., split tensile strength, bender elements	d: 38 h: 76 d: 50 h: 20	M	M	E
Li 2015	[3]Unconfined comp.	d: 50 h: 100	M	-	*
Al Qabany & Soga 2013	[3]Unconfined comp.	d: 100 h: 250 d: 35.4 h: 100		-	*
Cheng et al. 2013	[3]Unconfined comp., triaxial comp.	h/d ratio = 1.5÷2.0	M	-	E

[1] In-situ mechanical investigations, [2] vegetated soils, [3] MICP improved soils,
[4] w: gravimetric water content, s: suction, S_r: degree of saturation, K: constant water content, M: measurement of water content or suction, E: determined
*controlled temperature and relative humidity, oven-drying or air-drying during curing and before testing

retention properties of bio-remediated soils. Indeed, different arrangements and response times of the measuring tools within the soil could lead to discordant/highly scattered observations for a given hydraulic state. Therefore, it is essential to carry out laboratory experiments at steady-state conditions, ensuring a correct equalisation of suction. Given these considerations, vegetated soil water retention measurements under darkness or sun/artificial light conditions have been proposed in the literature to minimise such biological activities (Table 3 summarises different protocols and measuring tools). Matric suction control by axis translation has been used in microbial induced calcium carbonate precipitation MICP treated soils (Morales et al. 2015b). In contrast, it is still unexplored in vegetated soils, as possible damages caused by high air or water pressures on roots physico-mechanical traits are unknown. Despite providing reliable total suction values above the permanent wilting point of roots (1.5 MPa), vapour equilibrium technique or psychrometers have been so far rarely used (Fraccica et al. 2019, Morales et al. 2015b). Attention must also be paid to dielectric water potential sensors as they are affected by the organic content of soil (Ni et al. 2019b), which inevitably increases during vegetation or bacteria's activity. Although relative permeability of vegetated soil can be evaluated by indirect methods (Ng and Leung 2012), it should be complemented by in-vivo imaging techniques (i.e. X-ray microtomography, neutron radiography and/or infrared images: Anselmucci et al. 2021, Fraccica 2019, Carminati et al. 2010, Parera et al. 2020) to detect preferential paths or suction-induced capillary barriers generated by the roots.

In the literature associated with traditional geotechnical applications with bio-treated soils, mechanical tests at controlled water content or suction, such as unconfined and triaxial compressions and direct shear tests, have been widely used. Nevertheless, climate change effects (mainly droughts) will require focusing on shrinkage and cracking phenomena (Lakshmikantha et al. 2012, Ledesma 2016), in which rooted soils are particularly vulnerable due to their high heterogeneity and interface phenomena with concurrent drying of soil and roots. Within this context, direct tensile tests, such as those reported by Trabelsi et al. 2018, or wetting/drying paths are still poorly investigated in bio-remediated soils. Furthermore, interpreting direct tensile tests of rooted soils in a partially saturated framework is challenging. Therefore, advanced techniques should be able to infer principal directions of deformations (e.g., Particle Image Velocimetry), and tensile strength should be better assessed following a Mohr-Coulomb criterion with information of the total stress, degree of saturation and suction (e.g., Murray et al. 2019, Murray and Tarantino 2019, Fraccica et al. 2022a). Table 4 summarises selected mechanical studies, focusing on controlling or estimating the samples' hydraulic state variables and on the size adequateness of the specimens. Strongly coupled phenomena occur during mechanical tests at different hydraulic states. For example, matric suction not only enhances soil shear strength but also affects soil-roots bonding (Fraccica et al. 2022a) and soil-calcite contacts (Cheng et al. 2013). Therefore, a more accurate measurement/evaluation of suction should be considered in these investigations. In MICP improved soils, important suction values generated by biological activities and curing at given relative humidity are still poorly considered in the interpretation of unconfined compression tests (refer to Table 4).

4.2.4 *Soil chemical improvement*

Several grouting techniques based on the penetration of grout into the soil can be used: compaction, jet/mixing, fracture, and permeation. Compaction and soil mixing have been extensively studied (e.g., Gallagher and Mitchell 2002, Wong et al. 2018), while jet/mixing and fracture are hard to implement in laboratory investigations due to the scale involved and the inhomogeneity generated in soil due to excessive binder pressure (fingering, piston effects). Permeation grouting involves imposing low pressures on the material to be injected, usually a fluid, not to exceed the soil's total stress. The process by which non-wetting solutions or resin-type binders are injected and have to impinge on other wetting/non-wetting fluids (water/air) present in the soil can be studied by unsaturated soil's rheological laws (Wang et al. 2021). Interface phenomena such as surface tension, viscosity, and grout pressures should be considered, as well as the relative permeability and the actual stress-state of the soil matrix when optimising injection processes and testing new grouting products. Injection of partially saturated soils with more aqueous suspensions may decrease suction with potential soil volume changes. Imposing high total suctions on curing at relatively low relative humidity can damage the solidified binder's structure (Axelsson 2006, Spagnoli et al. 2021, Fraccica et al. 2021). Empirical relationships exist between improved grouted soil strength and water content/reactive binder ratio of the injected suspension (Consoli et al. 2016). More recently, direct implications on soil collapsibility have also been observed (Seiphoori and Zamanian 2022). Although many grouting applications are above the groundwater level (Le Kouby et al. 2018), the interpretation of laboratory and in situ tests still relies on the assumption of full saturation of the ground. For this reason, experimental studies usually do not go beyond the estimation of the saturated permeability and the unconfined compressive strength. This last property is anyhow affected by suction (Hossain and Yin 2012), usually not considered, generated by the curing environment or by the chemical process of binder solidification under water depletion conditions. Despite the discussion above, the influence of grouting techniques on soils' final water retention behaviour (clogging of pores, osmotic processes, alteration of water absorption) is still poorly explored. Table 5 summarises selected references in partially saturated grouted soils.

In addition to grouting, there has been work on chemical hydrophobic treatment to soils (e.g., Saulick and Lourenço 2020, Zhou et al. 2021), where there are theoretical and experimental challenges to address at the hydrophobic/hydrophilic interfaces.

Table 5. Mechanical tests in grouted soils with consideration of variables linked to the soil's hydraulic state.

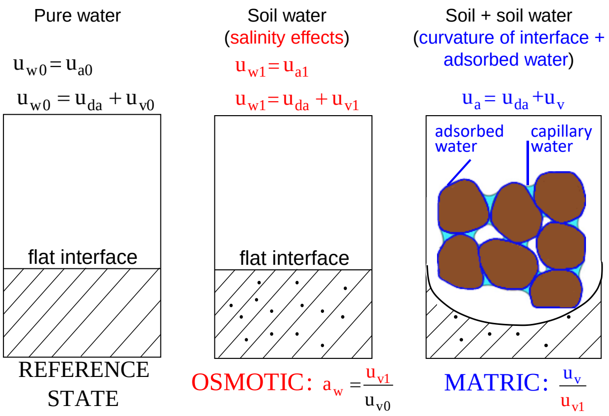
Reference	Laboratory test	[1]variable during curing/testing			
		w	*s*	*Sr*	*RH*
Seiphoori & Zamanian 2022	Oedometer test with inundation	KM	-	E	M
Wang et al. 2021	Grouting, direct shear at different Sr values	KM	E	E	C
Fraccica et al. 2021	Grouting, Unconfined compression	M	-	E	C
Hossain & Yin 2012	Soil-cement interface direct shear at controlled suction	M	C	E	-
Horpibulsuk et al. 2010	Soil compaction and unconfined compressions at constant water contents	KM	-	-	C
Porcino et al. 2011	Unconfined compression	-	-	-	C

[1] *w*: gravimetric water content, *s*: suction, S_r: degree of saturation, *RH*: relative humidity, K: constant water content, M: measurement of water content or suction, E: determined, C: controlled

4.2.5 *Tailings and industrial processes*

Wet slurry tailings are conventionally stored above-ground behind earthen dams and are affected by interactions with the atmosphere. This kind of material can therefore be studied with experimental techniques for unsaturated soils (Fredlund et al. 2003, Oldecop et al. 2011). In low precipitation areas, these materials undergo drying processes, which involve strongly coupled hydro-chemo-mechanical interactions. Shrinkage of the surface layer may also initiate cracking due to movement restrictions or soil's heterogeneity above the tensile strength of the soil that depends on suction and the degree of saturation (Ledesma 2016, Trabelsi et al. 2018, Tollenaar et al. 2018).

In mining materials subject to drying, desiccation and cracking may also introduce significant environmental impacts (Rodríguez et al. 2007). Cracking increases the permeability of the tailings and mining waste ponds, forming preferential paths for pollutant generation and infiltration to deeper layers and yielding oxygen accessibility for oxidation of the metallurgical wastes (Blanco et al. 2013). The long-term stability of tailing dams is also governed by the water content in the shallow unsaturated zone, in which the effects of dissolved salts are also significant. Air-drying experiments on metallurgical tailing sludges at controlled environmental conditions have been recently reported by Garino et al. (2021) using a fully instrumented column experiment. Their results evidenced the formation of a dry tailing crust enclosing a body of mud-consistency material. In addition, the authors discussed the role



$$\psi = -\frac{\rho_w RT}{M_w}\ln\left(\frac{u_v}{u_{v0}}\right) = -\frac{\rho_w RT}{M_w}\ln\left(a_w\frac{u_v}{u_{v1}}\right)$$

$$\pi = -\frac{\rho_w RT}{M_w}\ln(a_w)\,;\; s = -\frac{\rho_w RT}{M_w}\ln\left(\frac{u_v}{u_{v1}}\right)$$

$$\psi = \pi + s$$

Figure 13. Suction components (salinity effects and curvature of the meniscus/adsorbed water).

played by the increasing concentrations of dissolved/precipitated salts in this crust that limit the evaporation rate by increasing osmotic suction. Fine-grained tailings have been studied by Wickland et al. (2006), demonstrating that they can keep saturated conditions for more extended periods thanks to their high air-entry value. Oldecop et al. (2011) also observed high in situ water contents of old abandoned tailings even for low rainfall and high evaporation rates. Despite these high saturations, slope stability problems were not observed even with steep external slopes (40º-45º) attributed to high matric suctions and cementation effects of precipitated salts.

Filtered partially saturated tailings have been studied by Oldecop and Rodari (2021), who present them as efficiently yielding an unsaturated state, which is preferred as it allows avoiding the construction of containing dams, it minimises seepage and prevents liquefaction phenomena. Significant long-term effects in filtered tailings include air-drying phenomena upon tailing discharge and tailing compaction under the weight of subsequent stack lifts.

Experimental unsaturated soil mechanics is also relevant to powder and bulk solids technology and industrial processes (Mitarai and Nori 2006). The rheological properties of clay materials for ceramic processing are affected by their water content and particle size distribution and the presence of salts, sediments, and expandable minerals (Dondi et al. 2008). Moreover, moisture content significantly affects the bulk handling of the fine fraction of mining ore materials, affecting their cohesive/adhesive behaviour and handling properties (Cabrejos 2017, Li et al. 2019). The effect on discharge trajectories in material conveying lines was investigated using a laboratory-scale model of a belt for various materials (coal, gravel, magnetite, riversand) at controlled water contents (Ilic and Wheeler 2017). Moreover, wall adhesion tensile force tests based on standard wall friction tests were used to model the contact mechanisms between unsaturated bulk iron ore materials and selected surfaces employed in material conveying designs (Fang et al. 2020).

Hygroscopicity of granular materials, among other grain-scale effects (Torres-Serra et al. 2021b), affects their flowability and, thus, the performance of conveying processes. Column collapse experiments have been increasingly used to study a wide range of granular materials with industrial applications (Torres-Serra et al. 2020, 2021a). The effect of particle shape and the morphology of capillary water on granular stability was also studied on wet sand (Scheel et al. 2008). Particularly, the shear strength of unsaturated sands in the pendular state (hygroscopic conditions) was characterised by Richefeu et al. (2006), and their increasing tensile strength with water content was studied by Kim and Sture (2008) and Lu et al. (2007).

4.3 *Experimental Techniques in Unsaturated soil Mechanics*

4.3.1 *Controlled-suction techniques*

This section addresses the research effort during the last two decades regarding the development of controlled-suction techniques and the advances in measuring suction (matric, osmotic, and total). Several seminal state-of-the-art papers have described these techniques, such as Ridley and Wray (1996), Delage (2004), Agus and Schanz (2005), Marinho et al. (2008), Bulut and Leong (2008), Vanapalli et al. (2008), Blatz et al. (2008), Delage et al. (2008) to cite some of them.

The classification regarding controlled suction techniques can be based on how water is transferred. For example, water transport can involve slow vapour phase transfer by regulating the relative humidity of the air in contact with soil $RH = u_v/u_{v0}$, where u_v is the vapour pressure interacting with soil and u_{v0} the saturation vapour pressure over a flat surface of pure water at the same temperature. A relationship is therefore required to transform the relative humidity into the different components of suction (matric s, osmotic π, and total ψ). Figure 13 shows a simplified schematic of the suction components. The illustration in the middle indicates the osmotic component associated with the activity of the pore water, $a_w = u_{v1}/u_{v0}$, where u_{v1} is the vapour pressure over a flat surface of soil water that depends on the ion concentration at the far-field. The illustration on the right presents the matric component, u_v/u_{v1}, related to the curvature of the meniscus (fluid pressures) and the adsorbed water (physico-chemical interactions) (Lu and Zhang 2019).

The figure presents the thermodynamic relationship between the different suction components considering the equality of the chemical potential (molar Gibbs function) of water species in the two phases, vapour an ideal gas, water incompressibility and

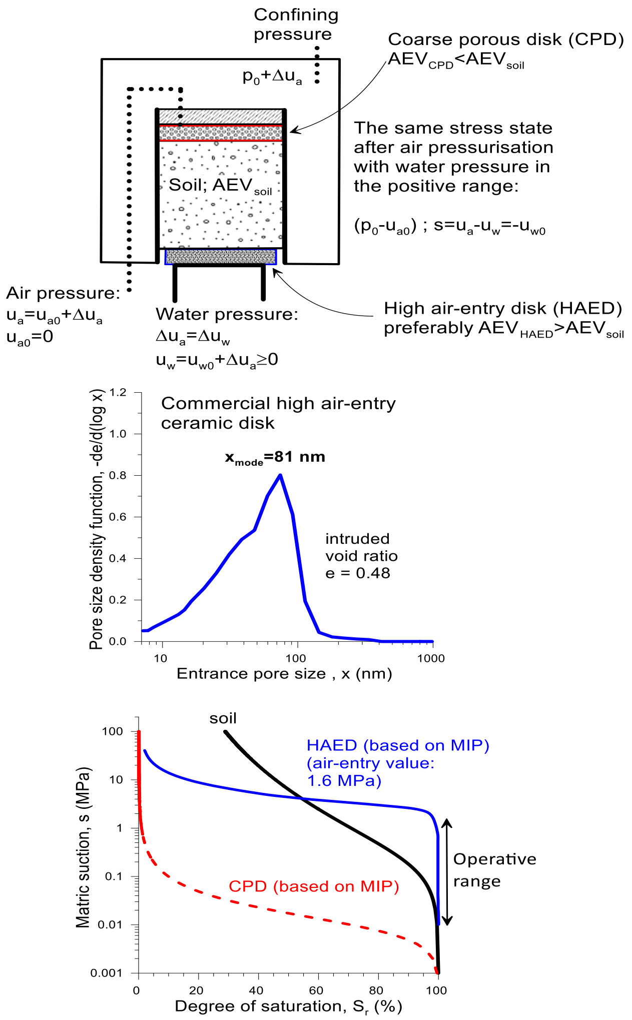


Figure 14. At the top: application of axis translation technique (isotropic stress conditions). The pore size density function of a commercial high air-entry ceramic disk (using mercury intrusion porosimetry) is at the middle of the figure. The water retention curves of a coarse porous disk, high air-entry ceramic disk and soil (operative range of axis translation) is presented at the bottom of the figure.

isothermal evolution (Castellan 1983). The psychrometric law (first expression at the bottom of the schematics) relates $\psi = \pi + s$ to RH where $M_w = 18.016$ kg/kmol is the molecular mass of water, R the molar gas constant, R the absolute temperature, and ρ_w the liquid water density.

From the perspective of experimentally controlling the matric suction with measurable variables, only the meniscus curvature (capillarity) is usually considered. Therefore, another method of handling the matric suction may be via the liquid phase transport by controlling the air and water pressures, $u_a$and u_w, respectively. In this case, matric suction is regulated by the pressure difference between non-wetting and wetting phases, $s = u_a - u_w$, acting on both sides of the curved meniscus interface. Air behaves as a mixture of several gases (dry air), and varying amounts of water vapour: $u_a = u_{da} + u_v$, where u_{da} is the dry air pressure and u_v the vapour pressure.

Two different procedures of liquid phase transfer can be used to control the water pressure at constant air pressure. The first one uses a chemically based control (osmotic technique), whereas the second procedure of controlling matric suction (capillary pressure) uses a hydraulic control with a hanging-water-column method for very low matric suctions (at $u_a = 0$) (Berliner et al. 1980, Vanapalli et al. 2008) or axis translation technique for higher matric suctions (at $u_a > 0$) (Delage et al. 2008, Romero et al. 2012b). The axis translation technique will be later discussed.

As previously indicated, the osmotic technique uses a chemically based control at $u_a = 0$ taking advantage of the properties of a pumped osmotic solution with high-molecular-mass polyethylene glycol PEG molecules (with activity a_{w2}) in contact with the soil and separated by a selective semi-permeable membrane, which allows solvent molecules and ionic species in the aqueous soil solution to pass. Description of the technique can be found in Blatz et al. (2008), Delage et al. (2008), Delage and Cui (2008) and Vandoorne et al. (2022). At equilibrium, a pore pressure deficiency (osmotic pressure), which depends on the PEG concentration (higher matric suctions at increasing concentrations), is induced on the pore water (soil water with activity $a_{w1} > a_{w2}$) that prevents its flow across the semi-permeable membrane.

$$u_w = \frac{\rho_w RT}{M_w} \ln\left(\frac{a_{w2}}{a_{w1}}\right) \tag{25}$$

Most calibrations relating PEG concentration and applied matric suction have been experimentally determined. A first intuitive approach is to measure the relative humidity (equivalent to a_{w2}) above solutions of PEG using psychrometers (Zur 1966, Williams and Shaykewich 1969) with calibration curves independent of PEG's molecular mass. Dineen and Burland (1995) and Tang et al. (2010) used a high capacity tensiometer directly on a sample/thin layer controlled by the osmotic technique through a selective membrane. However, an alternative approach has been recently proposed by Lieske et al. (2020) based on a modified Flory-Huggins thermodynamic polymer solution equation by knowing the average molar mass, the concentration, and the PEG-solvent interaction. Different selective membranes with different molecular weight cut-offs have been used, such as cellulose acetate (Tripathy et al. 2011) and more resistant polyethersulfone membranes (Slatter et al. 2000, Yuan et al. 2017). An advantage of the osmotic technique is that there is no need to apply any air pressure. In addition, high matric suction can be applied by using high concentration PEG solutions (maximum matric suction of 8.5 MPa has been used by Cuisinier and Masrouri 2005). Some of the problems related to this technique can be summarised as follows:

- PEG solution and mainly cellulosic membranes are susceptible to degradation during long-duration tests.
- Membrane effects tend to change the applied matric suction (concentration polarisation close to the membrane, membrane fouling and reverse solute draw; Vandoorne et al. 2022).
- Calibrations for long-term water volume changes, particularly with the stability of the weighing system (creep of the transducers of the electronic balance, vibrations of the lines connected to the peristaltic pump) and problems associated with water evaporation and circulation losses (Abbas et al. 2017).

Axis translation is a widely used technique for regulating matric suction, which stems from the pressure plate apparatus for pore water pressure measurement (Gardner 1956, Hilf 1956). However, since it controls non-wetting and wetting pressures, it can only regulate the matric suction's capillary pressure (meniscus curvature) and not the adsorption mechanism (Lu 2019). Figure 14 1at the top presents a schematic of its application in an isotropic cell. It involves the increase of the air pressure in which the soil is immersed $u_a = u_{a0} + \Delta u_a$, and the consequent increase of the pore water pressure $u_w = u_{w0} + \Delta u_a$, assuming that the curvature of the menisci is not significantly affected (Hilf 1956, Bocking and Fredlund 1980, Delage et al. 2008). In particular, this increase over atmospheric conditions has been criticised for not representing field conditions and strongly affecting (or suppressing) the water cavitation for capillary water (Baker and Frydman 2009, Lu 2019). A framework for soil water cavitation has been recently proposed and experimentally validated using axis translation and hygrometer-based methods by Luo et al. (2021). In addition, there are some doubts about how the air pressurisation process affects water held by adsorption mechanisms. A saturated high air-entry value HAEV interface in contact with the soil allows independent air and water pressure control (Figure 14 at the middle shows the pore size distribution by mercury intrusion porosimetry of a commercial HAEV ceramic disk of 1.5 MPa bubbling pressure). These interfaces, which limit air transport and are permeable to ionic species in the aqueous soil solution, are usually made of a) microporous membranes (polyether sulfone and acrylic copolymer) with air-entry values up to 250 kPa and prepared for improving the time required to reach equalisation (Nishimura 2013); b) ceramic-based disks (dominant pore sizes typically between 81 and 220 nm and air-entry values between 0.7 and 1.6 MPa, Romero et al. 2012b); c) cellulose acetate membranes (MWCO 3500 with largest pore size about 8 nm, Tripathy et al. 2011); and d) nanoporous glass interfaces (maximum pore sizes about 7 nm and maximum air-entry value around 7.3 MPa, Mendes et al. 2019). As depicted in Figure 14 at the bottom, the soil should preferably present a lower air-entry value than the corresponding one of the ceramic disk. In addition, to ensure that the top boundary presents no water flow (or storage) condition, the top coarse porous disk should display a very low air-entry value (this ensures that no water is stored when applying matric suction).

A benchmark aimed at comparing different experimental techniques for controlling/measuring suction (axis translation, osmotic technique, high capacity tensiometer and dew-point psychrometer) on a mixture of kaolinite, sodium bentonite and sand (reference soil) was presented by Tarantino et al. (2011). Different laboratories tested the two matric suction application techniques, and similar results were obtained in the range between 20 kPa and 1000 kPa, which gave further confidence in using the axis translation technique. Nevertheless, a specific discrepancy was observed in the medium range of matric suction (between 200 kPa and 400 kPa) between axis translation and a high capacity tensiometer (for the same water content, the suction measured by the tensiometer was lower). This discrepancy has been explained by Marinho et al. (2008), considering an increase in the meniscus curvature, and thus the matric suction, due to compression of entrapped air during water pressurisation. Discrepancies between water retention data by axis translation and dew point psychrometer for a silty soil have also been reported by Bittelli and Flury (2009) for matric suctions > 100 kPa, which were interpreted in terms of the suppression of cavitation (Lu 2019). Nevertheless, Boso et al. (2003) observed an excellent overlap between high capacity tensiometers (equilibrated and dynamic curves) and axis translation technique while drying clayey silt. In addition, Hoffmann et al. (2005) presented a good continuity between vapour equilibrium and transistor psychrometer results at elevated total suctions and nearly saturated states controlled with axis translation technique on compacted Febex bentonite. Axis translation has been successfully used by Mokni et al. (2014) to transfer liquids at different concentrations on a compacted clay and study osmotic suction effects on volume change behaviour and shear strength at different constant values of matric suction.

Experimental difficulties concerning its implementation are associated with the following aspects.

- The accumulation of diffused air beneath the saturated HAEV ceramic disk can induce the progressive loss of water continuity, water volume change errors in drained tests and pore-water pressure measurement errors in undrained tests (Airò Farulla and Ferrari 2005, Lawrence et al. 2005, Padilla et al. 2006). Increasing the water pressure is an efficient way to reduce air diffusion rates for specified matric suctions.
- The control of the relative humidity of the air chamber (around 95%) is required to minimise evaporative fluxes between the soil surface and the air chamber (Romero et al. 2012b).
- Applying air pressure at high degrees of saturation can induce irreversible arrangements in the soil skeleton due to pore fluid compression (entrapped air bubbles) (Bocking and Fredlund 1980). If nearly saturated states are expected to be reached during the hydraulic paths, it is preferable to increase air pressure when the continuity of air is ensured, and then maintain the continuous air phase at constant pressure.
- One crucial difficulty when using the axis translation technique is estimating the time required to reach matric suction equalisation (Oliveira and Marinho 2008, Romero et al. 2012b).

The vapour equilibrium technique, associated with the total suction, is implemented by controlling the relative humidity RH of a closed system where the soil is immersed (Figure 13). The RH of the reference system can be controlled by varying the chemical potential of different types of aqueous solutions (Delage et al. 1998, Tang and Cui 2005, Delage et al. 2008). The main drawback of this experimental technique is that the time to reach moisture equalisation is exceptionally long because vapour transfer depends on diffusion. Vapour transfer –through the sample or along the boundaries of the sample– can also be forced by a convection circuit driven by an air pump to speed up the process (Blatz and Graham 2000, Lloret et al. 2003, Alonso et al. 2005, Pineda et al. 2014c). The mass rate transfer of vapour by convection (assuming isothermal conditions and constant dry air pressure) can be expressed in terms of vapour density or mixing ratio differences between the reference vessel and the soil (Oldecop and Alonso 2004, Jotisankasa et al. 2007, Romero et al. 2012b).

One of the difficulties in using the vapour equilibrium technique is to maintain thermal equilibrium between the reference system and the sample. A way to minimise this thermal effect is achieved by disconnecting the reference system that regulates RH, and by allowing the equalisation of vapour in the remaining circuit and the soil. Another problem when using the forced convection system is the air pressure differences created along the circuit. This phenomenon makes the reference vessel's RH not be adequately controlled in the remaining circuit and the soil (Pintado et al. 2009a,b). These authors observed that forcing humid air reduced the equalisation time, but the results highlighted that this flow must be carefully applied to avoid reaching a different total suction than regulated.

A significant issue is to estimate the time required for equilibrating the total suction homogeneously. The conventional experimental criterion to define the hydraulic equalisation period is based on measuring volumetric strains and controlling strain rates below a specified value (typically below 0.1%/day, as suggested by Merchán et al. (2011) based on hydro-mechanical numerical simulations), due to the experimental difficulty in knowing the precise hydraulic status of the sample during vapour transfer (usually the evolution of vapour transferred to or from the sample is not measured).

4.3.2 *Suction measurement techniques*

This section will focus on the development of high capacity matric suction tensiometers and the most used psychrometers.

Significant progress has been made in developing high capacity tensiometers with fast response time since the probes used by Ridley and Burland (1993) and Tarantino and Mongiovì (2003). These probes share the same configuration comprising a tiny water reservoir, a sufficiently thick high air-entry interface and a pressure transducer. Therefore, the improvements have been focused on using different interfaces, pressure transducers, water reservoir volume, and interface saturation. Table 6 summarises the improvements made. Within this context, correct protocols should be followed to pressurise the chamber and ensure the saturation of the high air-entry interface with de-aired water (Mendes and Buzzi 2014). In addition, a soil paste/filter should be inserted between the stiff high air-entry interface and the soil to improve contact and delay evaporation of the interface. Different prototypes have been recently compared for long-term matric suction measurements by Mendes and Gallipoli (2020) regarding interfaces, pressure transducers, water reservoirs and protective casings. Probes incorporating a small water reservoir showed a more remarkable ability to sustain suction over a long period without cavitating. Mendes et al. 2019 used a nanoporous glass interface (96% SiO2 with maximum pore sizes about 7 nm), which allowed reaching matric suctions of 7.3 MPa.

Nevertheless, despite these different studies, the phenomenon of air diffusion through the saturated interface, driven by the gradient of air concentration, remains an open issue when considering the long-term stability of these probes and the possibility of the loss of water continuity. Air diffusion depends on the thickness of the interface and the pore water pressure in the chamber with an external atmospheric air (Airò Farulla and Ferrari 2005, Romero et al. 2012b).

Table 6. High capacity tensiometers (updated information based on Delage et al. 2008, Toll et al. 2013).

Reference	Air-entry value (kPa)	Water reservoir volume (mm^3)	Pressure transducer
Ridley & Burland 1993	1500	-	Modified/commercial (3.5 MPa)
Guan & Fredlund 1997	1500	Approx. 20	Modified/commercial (1.5 MPa)
Meilani et al. 2002	500 (1-mm thick ceramic)	-	Modified/commercial (1.5 MPa)
Tarantino & Mongiovi 2003	1500	< 4.5	Strain gauged diaphragm (4 MPa)
Take & Bolton 2003	300	-	Modified/commercial (0.7 MPa)
Ridley et al. 2003	1500	Approx. 3	Strain gauged diaphragm (8 MPa)
Lourenço et al 2006	1500	5	Ceramic transducer (2 MPa)
He et al. 2006	1500	-	Modified/commercial (3.5 MPa)
Mahler & Diene 2007	500-1500	5-112	Modified/commercial (tensiometer acrylic body)
Jotisankasa et al. 2007	500	60	Modified/commercial (piezoresistive sensor)
Oliveira & Marihno 2008	1500	-	Modified/commercial (3.5 MPa)
Mendes & Buzzi 2014	500-1500	4-800	Modified/commercial (3.5 MPa)
Mendes et al. 2019	7300 (nanoporous glass interface)	4	Modified/commercial (35 MPa)

A great effort has also been dedicated to extending psychrometers' working range. High-range alternatives to the widely used Peltier thermocouple psychrometry (Campbell 1979, Fredlund and Rahardjo 1993) were developed in the 90s and 2000s: a) transistor psychrometers (Soil Mechanics Instrumentation SMI type: Woodburn and Lucas 1995) with an upper limit of 70 MPa and long-term measuring time (1 hour), and b) chilled-mirror dew-point psychrometers with an upper limit of 300 MPa and involving a reduced time of reading (3 to 10 min). In particular, the latter device has been improved and widely used in recent years and has become a reference system to determine total suction (Leong et al. 2003, Cardoso et al. 2007, Ebrahimi-Birnag and Fredlund 2016). The chilled-mirror dew-point psychrometer measures the temperature at which condensation first appears (dew-point temperature). Then, in equilibrium with the surrounding air, a soil sample is placed in a housing chamber containing a mirror and a photoelectric detector of condensation on the mirror. A thermoelectric (Peltier) cooler precisely controls the mirror's temperature. The relative humidity is computed from the difference between the dew-point temperature and the temperature of the soil sample.

Cardoso et al. (2007) detected some discrepancies between transistor and chilled-mirror dew-point psychrometers in the high suction range (7 to 70 MPa) –systematically larger values were detected with the dew-point psychrometer–. These authors suggested that the hydraulic paths undergone by the soil during the measurement period inside each equipment chamber were different. The soil inside the chilled-mirror chamber undergoes some drying before reaching equalisation, and it will follow a main drying path during the measuring period.

4.3.3 *Hydro-mechanical cells and mock-up tests*

Standard cells for saturated soils have been modified to test unsaturated soils using different techniques to determine soil and water volume changes and apply or measure suction. The initial developments of these experimental systems are explained in detail in Delage (2004) and Delage et al. (2008). Table 7 summarises selected developments in different hydro-mechanical cells, in which diverse techniques for controlling total and matric suctions were implemented (vapour equilibrium technique VET, axis translation technique ATT, hanging-water-column HWC method at atmospheric air pressure, osmotic technique OMT, and constant water content CWC tests with suction measurement).

Mock-up tests are usually designed to allow larger-scale investigations of coupled processes before or simultaneously with in situ tests to complement data and demonstrate their feasibility under better-controlled boundary conditions while validating numerical models to predict the real scenarios better.

In the field of radioactive waste disposal, several mock-up experiments have been developed to study the chemo-thermo-hydro-mechanical behaviour of bentonite-based materials. These materials' initial phase of saturation generates the most relevant interest from an unsaturated soil mechanics viewpoint since it leads to complex stress/suction paths (Mesa-Alcantara 2021, Bian et al. 2020, Saba et al. 2014, Wang et al. 2013). Some mock-ups also consider the coupled thermal response mimicking heat emitting wastes (Chen et al. 2014, Pacovský et al. 2007, Šťástka 2014, Huertas et al. 2006, Villar et al. 2012, Martín and Barcala 2005). These tests rely on the installation of multiple sensors to locally measure temperatures, pore pressures, relative humidity or total suctions, total stresses, and displacements.

Also interesting are the infiltration column tests that are widely used for several applications, including capillary barriers (Zhou et al. 2021, McCartney and Zornberg 2010, Zhan et al. 2014, Tan et al. 2018, Yang et al. 2004), geosynthetic clay liners (Bathurst et al. 2007, Bathurst et al. 2009), road pavements and railway structures (Duong et al. 2013), contaminated soils (Rahardjo et al. 2018, Murakami et al. 2008) or landfills (Ng et al. 2016c), among others. The infiltration columns can be assessed as one-dimensional boundary condition problems in which several infiltration regimens can be applied. Sensors such as tensiometers, matric potential sensors, pressure transducers or time domain reflectrometry TDR probes for volumetric water

Table 7. Hydro-mechanical cells with suction control/measurement.

Reference	Technique	Improvement
Triaxial cells		
Romero et al. 1997	ATT	Local displacement transducers (laser-based transducer for radial displacements). Suction and temperature-controlled system
Blatz & Graham 2000	VET	Suction measurement by thermocouple psychrometry
Hoyos & Macari 2001	ATT	True triaxial apparatus
Toyota et al. 2001	ATT	Triaxial cell and hollow cylinder apparatus
Meilani et al. 2002	ATT	Mini-suction probe for matric suction measurement
Ng et al. 2002	ATT	Differential pressure transducer system to measure sample volume change
Toyota et al. 2004	ATT	Hollow cylinder torsional shear apparatus
Buenfil et al. 2005	ATT	Local displacement transducers (laser-based transducer for radial displacements)
Padilla et al. 2006	ATT	Double-wall cell and diffused-air flushing device
Jotisankasa et al. 2007	HWC	Continuous monitoring of water content and suction
Siemens & Blatz 2007	CWC	Xeritron sensor to measure suction in the centre of the sample
Alramahi et al. 2008	ATT	P and S-wave velocity
Rojas et al. 2008	ATT	Reduced testing time
Romero & Jommi 2008	ATT	Isotropic cell (laser-based transducer for radial displacements)
Uchaipichat & Khalili 2009	ATT	Temperature and suction-controlled cell
Perez-Ruiz 2009	ATT	True triaxial apparatus for cubical samples under constant-suction control
Chávez et al. 2009	VET	Large size cell for rockfill with local transducers
Biglari et al. 2011	ATT	Suction-controlled cyclic triaxial tests
Hoyos et al. 2011	ATT	Refined true triaxial apparatus
Mendes et al. 2012	ATT	Double-wall cell with glass to eliminate water absorption
Muñoz-Castelblanco et al 2012	-	Monitoring local changes in water content with an electrical resistivity probe
Cruz et al. 2012	ATT	Biaxial apparatus for plane strain conditions
Pineda et al. 2014c	VET	High-pressure apparatus with a forced convection system
Cai et al. 2014	ATT	Temperature and suction control triaxial testing
Li et al. 2015	CWC	Photogrammetry-based method to measure volume changes
Alsherif & McCartney 2015	VET	High suction and elevated temperatures
Zhang et al. 2015	CWC	Photogrammetry-based method to measure volume changes
Nishimura 2016	VET	Creep triaxial cells with cyclic relative humidity control system
Patil et al. 2016	VET & ATT	Control of matric or total suction during shearing
Mora Ortiz 2016	ATT & CWC	Isotropic cell with the matric suction measurement and local displacement transducer
Banerjee et al. 2020	ATT	Multi-stage triaxial to reduce testing time
Zhang et al. 2020	ATT	System to precise inject known amounts of water into the specimen
Direct shear apparatus		
Caruso & Tarantino 2004	CWC	Cell with a high capacity tensiometer
Ng at el. 2007	ATT & OMT	Comparison of ATT and OMT techniques
Hamid & Miller 2009	ATT	Direct shear interface tests
Jotisankasa & Mairaing 2010	CWC	Cell with a high capacity tensiometer
Kim et al. 2010	ATT	Low confining pressures
Nam et al. 2011	ATT	Multi-stage to reduce testing time
Hamidi et al. 2013	OMT	Higher suction range than with ATT
Borana et al. 2015	ATT	Shear strength of unsaturated soil-steel interfaces
Gallage & Uchimura 2016	HWC	Negative gauge pore-water pressure for a low suction range
Rosone et al. 2016	ATT	Adapted conventional cell
Ring shear apparatus		
Infante Sedano et al. 2007	ATT	Measurement of water content
Merchán et al. 2011	VET	Forced convection system to transport vapour
Oedometer cells		
Villar 1999	ATT/VET	Cell with a combined system for controlled suction
Romero et al. 2003	ATT	Cell with temperature control. Flushing system and evaporation control.
Oldecop & Alonso 2004	VET	Large cell for rockfill
Cuisinier & Masrouri 2005	OMT & VET	Large range of suction by combining two techniques
Tarantino & De Col 2008	CWC	Cell with a high capacity tensiometers
Airò Farulla et al. 2010	ATT	Wetting/drying cycles
Le et al. 2011	CWC	Cell with a high capacity tensiometer
Monroy et al. 2014	OMT	Miniature tensiometers and active radial stress system
Mokni et al. 2014	ATT	Oedometer and direct shear tests also controlling osmotic suction
Wijaya & Leong 2016	CWC	Cell with a high capacity tensiometer
Oldecop & Alonso 2017	VET	Measurement of lateral stress and friction
Cardoso et al. 2017	CWC	Microfabricated sol-gel relative humidity sensor
Maleksaeedi et al 2019	ATT & HWC	Continuous measurement of water exchange
Multi-functional cells		
Romero et al. 2001	ATT	Temperature control system for oedometer and triaxial cells
Aversa & Nicotera 2002	ATT	System for oedometer or triaxial cells
Lourenço et al. 2011	VET	Tensiometer based suction control system
Liu et al. 2021	ATT	Modular design for pressure plate test, oedometer test, direct shear test, and triaxial creep test

content are typically installed at different column heights to monitor the saturation front and measure suction and water content.

Cracking in desiccating soils due to droughts is receiving more attention since it strongly affects soils' hydraulic and mechanical behaviour and is relevant to many engineering applications (shallow foundations, dikes or soil covers for mining, industrial and municipal waste). However, using small-size laboratory samples has a clear drawback since the mechanical boundary conditions imposed by the containers have an impact on the process of crack formation and propagation (Cuadrado et al. 2019, Lakshmikantha et al. 2018). Therefore, there is increasing use of large-scale specimens in environmental chambers mimicking environmental variables such as wind velocity, air relative humidity or solar radiation (Ledesma 2016, Cui et al. 2013, Yesiller et al. 2000, Lakshmikantha et al. 2018, Rodriguez et al. 2007, Costa et al. 2013, Peron et al. 2009, Tang et el. 2011, Levatti et al. 2019, Tang et al. 2020, Garino et al. 2021). Different instrumentation is employed to understand crack formation and propagation and curling phenomena. Sanchez et al. (2013) presented an automatic 2D profile laser that allows scanning the overall surface of a drying soil (evolving crack aperture and depth) and an electronic balance to measure the water loss. A similar non-contact electro-optical laser-based technique was used by Zielinski et al. (2014) to observe soil curling phenomena precisely (evolution of the exposed surface of a natural soil during controlled drying conditions). Infrared thermal camera and PIV technique have also been used by Zeng et al. (2022) to track the evolution of desiccation cracking while monitoring the difference between the temperature of the evaporating surface and the atmosphere (soils at higher water contents display more significant temperature differences). Moreover, the implementation of ground-penetrating radar technique (Prat et al.

2013) has allowed detecting the formation of cracks inside the soil mass. Electric resistivity sensors (Borsic et al. 2005, Comina et al. 2008, Sentenac and Zielinski 2009, Kong et al. 2012) have also allowed monitoring spatial and time evolutions of soil fissuring.

4.3.4 *Microstructural techniques (multi-scale tests)*

Multi-scale studies associated with the pore network, the arrangement of grains and their interactions (inter-particle contacts/bonding, wettability) are increasingly used to improve the understanding of the hydro-mechanical response and multiphase flow properties. Regarding pore/fissure characterisation, these studies challenge unsaturated soil mechanics due to the wide range of pore sizes (from few nm to hundreds of μm) and their consequences on fluid storativity, permeability, diffusivity, single and multi-phase fluid flow properties. These studies include the following topics (e.g., Juang and Holtz 1986, Delage et al. 1996, Komine and Ogata 1999, Aung et al. 2001, Simms and Yanful 2002, Koliji et al. 2006, Yuan et al. 2020, Nguyen et al. 2021, Yuan et al. 2021):

- Pores size, morphology and orientation,
- Pore size distribution changes along thermo-hydro-mechanical and chemical paths, and gas injection processes,
- Discrimination of the type of the pore space (micropore/macropore, matrix/fissures, pore body/pore throat),
- Distribution of the pore volume (micropore volume/ macropore volume, volume of fissures, fluid occupancy effects during multiphase flow displacement processes),
- Connectivity of the pore space,
- Random distribution of porosity,
- Wettability issues and interaction with grains.

Among various techniques used to study porous geomaterials at the microstructural scale, mercury intrusion porosimetry MIP and scanning electron microscopy (particularly environmental ESEM with digital image analysis) were pioneering techniques that are still used. MIP is utilised for analysing the pore size distribution PSD of geomaterials with interconnected porosity (Delage et al. 1996, Romero and Simms 2008). This technique applies absolute pressure to a non-wetting liquid (mercury) to enter the empty pores. In addition, Washburn equation is adopted under equilibrated conditions (i.e., null penetration velocity of mercury and constant contact angle that does not vary with the advancing interface) to provide a relationship between the applied pressure and the entrance size of the intruded pores (e.g., Juang and Holtz 1986). Sample preparation requires emptying the sample of water, which can be dehydrated using controlled relative humidity-drying, oven-drying, freeze-drying or critical-point-drying technique (Delage and Pellerin 1984). For drying sensitive materials, freeze-drying is preferred (Delage and Pellerin 1984, Delage et al. 1996). The main limitations of MIP are a) enclosed porosity is not measured; b) pores that are accessible only through smaller ones are not detected until the smaller entrance pores are penetrated; c) the apparatus may not have enough capacity to enter the smallest pores (non-intruded porosity with entrance pore sizes below 7 nm); d) the minimum pressure which can be applied limits the maximum detected pore size (non-detected porosity with entrance pore sizes larger than 400 μm); and e) alteration in the pore volume of compressible materials before mercury penetration starts. MIP results are reported as pore size density function PSD, i.e., the log differential intrusion curve versus entrance or throat pore size, which aids visual detection of the dominant pore modes. Data from MIP can be complemented, for pore sizes below 60 nm, with nitrogen desorption isotherms (although using the adsorption branch is also possible). Data from the latter technique are interpreted using BJH model, based on which emptying of pores from condensed adsorptive at decreasing relative nitrogen pressure is re-interpreted by Kelvin equation (e.g., Webb and Orr 1977).

The environmental scanning electron microscope ESEM is a quantitative technique with minimal sample preparation requirement, and which allows subjecting the sample to hydraulic paths during observation (Komine and Ogata 1999, Montes-H et al. 2003a,b, Romero and Simms 2008). The ESEM is a particular type of SEM that works under controlled environmental conditions and requires no conductive coating on the specimen. Montes-H et al. (2003a,b) used ESEM jointly with a digital image analysis program to estimate bentonite's swelling/shrinkage behaviour at different total suctions at aggregate scale. Romero and Simms (2008) and Airò Farulla et al. (2010) used the same technique to study the effects of total suction changes on the volumetric behaviour at the microstructural level of different clays. Karatza et al. (2021) used ESEM to investigate the formation and evolution of meniscus structures (capillary bridges) in hydrophobic grain surfaces during wetting/drying cycles.

Regarding X-ray computer tomography CT, image analysis is based upon the ability of different materials to attenuate photons emitted by an X-ray source in different proportions, depending on multiple characteristics (i.e., bulk density, porosity, atomic number, chemical composition, water content) and the X-ray equipment (X-ray energy, intensity) (Hounsfield 1972). During an X-ray tomography scan, a 3D map is reconstructed consisting of voxels, each representing, through a grey value, the photon attenuation that the material provides at a specific point in the space. Such a technique has been successfully used in the investigations of unsaturated soils since voxels constituted by soil grains, water, or air can be easily distinguished due to their different attenuation capacities (Salager et al. 2014). Depending on the X-ray image resolution, voxels can include one or multiple phases: in this last case, the grey value will represent a linear combination of the attenuation values of those phases, weighted by the porosity and the degree of saturation of the material (Luo et al. 2008).

Apart from the extensive use of this technique for qualitative purposes in geotechnical engineering, X-ray CT is increasingly being used to observe and quantify features and additional phases with a high organic matter, such as bacteria, vegetation and grout binders, within partially saturated soils (Terzis and Laloui 2019, Fraccica 2019, Anselmucci et al. 2021). X-ray CT (i.e., with a resolution of the order of microns) has been successfully used to extend and complement the upper limit of soil's PSD coming from MIP (Münch and Holzer 2008, Yang et al. 2015, Fraccica et al. 2019). Such result has direct implications for understanding the hydro-mechanical behaviour of partially saturated soils, whose microstructure is modified by multi-physical phenomena (e.g., the evolution of root-induced fissures with suction, fissures generated by gas transport, arrangement and evolution of solidified grouts or calcite bonds at varying degrees of saturation within soil matrix). X-ray CT has also been used to achieve a deeper understanding of:

- The geometrical and spatial-temporal evolution of liquid bridges and wetting fronts, fundamental to trace adhesion forces between hydrophilic/hydrophobic grains and understand shear strength behaviour of partially saturated soils (Wildenschild et al. 2005; Berg et al. 2013; Pot et al. 2015), the impact of grain's shape and contacts on the phenomenological soil mechanical behaviour (Karatza et al. 2019).
- Strain fields and shear bands in unsaturated soils under different stress-paths (e.g., pioneering works of Desrues et al. 1996 and Otani et al. 2000 and recent works of Higo et al. 2013 and Takano et al. 2015).
- Desiccation cracks (Tang et al. 2019), injectability and effects of hydraulic states on grouted/bio-improved soils (Minto et al. 2017, Pedrotti et al. 2020).

- Ice contacts formation and evolution in soil, frost heave (Starkloff et al. 2017, Wang et al. 2018, Song et al. 2021).
- Multi-phase flows or gas/solute transport phenomena (Andrew et al. 2014, Larsbo et al. 2014, Gonzalez-Blanco et al. 2020).
- Methane hydrate processes (Kneafsey et al. 2007).
- Ground loss and granular flow for model tunnels or extraction advancement (Takano et al. 2006, Viggiani et al. 2015).
- Estimation of external stress transmission through the soil matrix (Naveed et al. 2016).
- Oil-water-air interfaces (Culligan et al. 2006; Gharbi and Blunt 2012).

When performing X-ray tomography, the first challenge is finding an adequate compromise between REV and resolution, as this last characteristic is strictly linked with the specimen's size (Li and Shao 2020). For this reason, partially saturated clays are still poorly investigated at the scale of pore and liquid bridge contact, even if research is pushing towards the development of miniature soil mechanics equipment (Cheng et al. 2020). Moreover, poor-quality X-ray CT of unsaturated soil with silty or clayey matrix might produce incorrect distinctions of porosity and grains if excessive noise (i.e., grey value's scattering) or edge/ring artefacts are present in the images. Despite the limitations of X-ray images (strictly linked with the experience in using the equipment), many investigations on partially saturated soils use this powerful technique. A selection is presented in Table 8 jointly with the type of soils investigated. Given that X-rays are more attenuated by soil grains than water and that neutron tomography offers a higher contrast in the visualization of the water phase, the two techniques have been recently used in a complementary way (Kim et al. 2013, 2016, Stavropoulou et al. 2020).

A good combination of microstructural techniques and phenomenological measuring tools can be successfully used to link microstructural features of partially saturated soils such as liquid bridges, through the evolution of their curvature, with grain's adhesion forces (Hueckel et al. 2020).

4.3.5 *1-g scaled experiments and centrifuge tests*

Table 9 summarises selected experimental 1-g scaled and centrifuge N-g (where N is the scale factor, and g is the Earth's gravity) set-ups proposed to study the unsaturated mechanical behaviour of geomaterials under distinct flow and water content regimes. Among 1-g testing, the well-established granular column collapse experiment was first proposed for the investigation of dry granular flows by Lajeunesse et al. 2004 and Lube et al. 2005. Unsaturated collapse experiments using natural materials are of interest though scarce (e.g., Fern and Soga 2017). Scale model embankments are built to investigate rainfall infiltration into various soil types and the consequences on stability. The performance of permeable geosynthetics on sandy soils was assessed by Garcia et al. (2007), instrumenting a soil box with pore pressure and moisture content sensors. Rainfall infiltration in gravelly soils was also studied with a 2D seepage set-up by Dong et al. (2017). Computed tomography was used to identify the main contributing effects to the observed infiltration rates. Other 1-g experiments were designed to characterise the unsaturated behaviour of transparent soils (Iskander 2018, Siemens et al. 2013) and hydrophobic soils (Karatza et al. 2021, Zhou et al. 2021), as well as the effect of freezing/thawing (Caicedo 2017).

The term centrifuge testing is used to describe the measurement of physical properties of unsaturated soils such as suction or permeability using geo-centrifuges (Timms et al. 2014), whereas centrifuge physical modelling refers to the extrapolation of reduced scale model tests to full-scale geotechnical applications (Caicedo and Thorel 2014). Another advantage of centrifuge modelling is observing the hydraulic

Table 8. X-ray tomography investigations in unsaturated soils.

Reference	Soil (USCS)	Applications
Li & Shao 2020	CL	HM behaviour: Soil bulk density; Pore-size distribution; Tortuosity; WRC; Strain fields and shear bands under mechanical stresses.; Evolution of liquid bridges/wettability/contact angle; Effects of grain size and shape on soil HM behavior; Desiccation cracks
Moscariello & Cuomo 2019	Artificial sands	
Tang et al. 2019	CH	
Karatza et al. 2019	Zeolite granules (d_{50} = 1.36 mm)	
Khaddour et al. 2018	SP	
Karatza et al. 2018	Caicos sand	
Mukunoki et al. 2016	SP	
Takano et al. 2015	Yamazuna Sand (d_{50} = 0.54 mm)	
Pot et al. 2015	Silt (56%) with sand (27%) and clay (17%)	
Yang et al 2015	Sandstone	
Berg et al. 2013	Sandstone	
Higo et al. 2013	SP	
Bull et al. 2022	Sand (71%) with fines (29%)	Bio-improved soils: Pore size distribution; WRC; Root volume and architecture; Root-induced fissures; Root-induced grains kinematics
Kemp et al. 2022	Clay/silt/sand/gravel layers	
Anselmucci et al. 2021	SP	
Fraccica 2019	SM	
Terzis & Laloui 2019	SP	
Koebernick et al. 2017	Sandy loam	
Minto et al. 2017	Sandstone	
Keyes et al. 2017	Sand (52.7%) and fines (47.3%)	
Carminati et al. 2013	Sand (92%) with fines	
Kim et al. 2021	Clay	Multiphase flows, gas/solute transport
Gonzalez-Blanco et al. 2020	Granular bentonite	
Andrew et al. 2014	Limestone	
Larsbo et al. 2014	Silty and clayey soils	
Pedrotti et al. 2020	Sand (d_{50} = 1.2 mm)	Grouted soils
Takano et al. 2006	Dry sand	Granular flows and ground loss processes
Viggiani et al. 2015	Fine sand (d_{50} = 0.2 mm)	
Song et al. 2021	Volcanic ash sand and terrace deposits	Ice formation/evolution in partially saturated soils
Wang et al. 2018	CL	
Starkloff et al. 2017	Sandy and silty soils	
Culligan et al. 2006	Glass beads	Oil-water-air interfaces
Gharbi & Blunt 2012	Carbonate limestones	
Kim et al. 2016	Sand	Projectile impact/penetration engineering

long-term soil response in reduced time frames (Dell'Avanzi et al. 2004). Scaling laws for centrifuge modelling of unsaturated soils were recently reviewed, focusing on soil water retention scaling (Mirshekari et al. 2018). The capillary rise was studied in sandy soils (d_{10}≤0.1 mm), for which the scaling is not affected by the effects of gravity on menisci shape, unlike in coarser soils (Rezzoug et al. 2004). Scaling laws are generally defined by dimensional analysis as in saturated soils, albeit showing discrepancies for unsaturated flow problems. Nevertheless,

Table 9. Classification of selected experimental set-ups for investigating the unsaturated mechanical behaviour of granular materials.

Reference	Flow regime		Saturation state	
	Steady	Transient	Pendular	Funicular/ Capillary
1-g (granular collapse tests)				
Artoni et al. 2013		●	●	
Gabrieli et al. 2013		●	●	
Morse et al. 2014		●	●	
Santomaso et al. 2018		●	●	
Torres-Serra et al. 2018	●		●	
Pinzón & Cabrera 2020		●	●	●
Taylor-Noonan et al. 2 020		●	●	●
N-g				
Ng et al. 2008		●	●	●
Askarinejad et al. 2014		●	●	●
Caicedo et al. 2015	●		●	●
Rotisciani et al. 2016	●			●
Lozada et al. 2018	●			●
Lalicata et al. 2020	●			●
Lucas et al. 2020		●	●	●
Escobar et al. 2021	●		●	

Table 10. Scaling factors for centrifuge modelling concerning length scale N of full-scale prototypes.

Quantity	Prototype/model
Length	N
Volume and mass	N^3
Time (diffusion)	N^2
Acceleration, gravity	N^{-1}
Force	N^{-2}
Stress, moduli and strength	*1*
Rezzoug et al. 2004	
Capillary rise	N
Dell'Avanzi et al. 2004[a]	
Discharge velocity	N^{-1}
Suction	*1*
Caicedo & Thorel 2014	
Rate of water content	N^{-2}
Rate of volumetric strain	N^{-2}
Heat flux	N^{-1}
Rain intensity	N^{-1}
Rain duration	N^2
Rain frequency	N^{-2}
Rate of evaporation	N^{-1}
Wind velocity	*1*
Vapour pressure deficit	N^{-1}
Askarinejad et al. 2014	
Seepage velocity (macro/micro)	N^{-1}
Seepage time (macroscale)	N^2
Seepage time (microscale)	N
Hydraulic gradient (macro/micro)	N^{-1}

[a] assuming centrifuge arm length much larger than the model length

analytic determination of scaling laws provides a consistent framework for scaling steady-state and unidimensional flow in generic hydraulic conductivity functions (Dell'Avanzi et al. 2004). Relevant scaling factors for unsaturated soil phenomena, including heat exchange, infiltration, evaporation, and rainfall (Askarinejad et al. 2014, Caicedo and Thorel 2014), are summarised in Table 10. The factors are given a length scaling factor N=L_prototype/L_model, where L_model is the length of the small scale physical model that is compared to the quantity in the full-scale prototype. In equivalent terms, it corresponds to an acceleration scaling N=a_model/g, where a_model is the centrifugal acceleration that scales up gravity g.

Geotechnical applications of centrifuge modelling can be found on:

- Characterization of the at-rest coefficient of lateral earth pressure of unsaturated soils (Li et al. 2014) and expansive clay swelling (Gaspar et al. 2019).
- Slope stability (Higo et al. 2015) including root-induced phenomena (Leung et al. 2017, Liang et al. 2017a) and live pole reinforcement (Ng et al. 2017; Kamchoom and Leung 2018).
- Climatic or environmental chambers (Archer and Ng 2018);.
- Rainfall (Bhattacherjee and Viswanadham 2018, Khan et al. 2018) and seepage (Beckett and Fourie 2018).
- Seismic induced settlement of shallow foundations on unsaturated soils (Borghei et al. 2020).

The leading experimental technologies applied in centrifuge investigation, also recently reviewed by Take (2018), include:

- Image analyses (Beckett and Fourie 2018) and especially the Particle Image Velocimetry (PIV) (Iskander 2018).
- 3D printing (Liang et al. 2017b).
- HR sensors (Takada et al. 2018), tensiometers (Basson et al. 2021, Jacobsz 2018, Kwa and Airey 2018), laser-based displacement and fibre optic strain sensors.

Image analysis techniques such as the PIV are widespread in the observation of flow kinematics (Morse et al. 2014, Pinzón and Cabrera 2020, Torres-Serra et al. 2020). The PIV is also known as Digital Image Correlation (DIC) in geomechanics frameworks (Hall 2012). In 2D-surface DIC analyses the displacement fields are recovered, starting from an initial reference configuration, by cross-correlation of the subsequent image pairs of the deforming material. Resolution of the PIV analysis depends on the size of the correlation windows, the pixel subsets being traced, and of the search region or region of interest ROI, as well as on the recording frame rate, defining the time spacing between analysis points. Recent advancements on the PIV technique include its enhancement to large displacement and deformation observations for landslide modelling in both 1-g and N-g (Pinyol et al. 2017). PIV is also currently used in the study of local frost deformation (Wang et al. 2020) and unsaturated soil-structure interaction (Vo et al. 2016, Shwan 2019, Speranza et al. 2020).

Novel imaging applications using infrared techniques are of special interest to the study of unsaturated materials. The saturation degree of soils can be measured by short wave infrared SWIR images from the different absorbance of incident light existing between water and the solid particles. SWIR imaging was thereby successfully used to observe near-surface water infiltration profiles for sandy soil column set-ups (Parera et al. 2020, Sadeghi et al. 2017). Hydraulic soil properties were also recently derived from SWIR measurements of volumetric water contents (Bandai et al. 2021).

5 FIELD INSTRUMENTATION

5.1 *Preamble*

The quantities required to be measured in the field, to contribute to the design and monitoring of engineering works, are associated with the type of mechanical demands that the structure

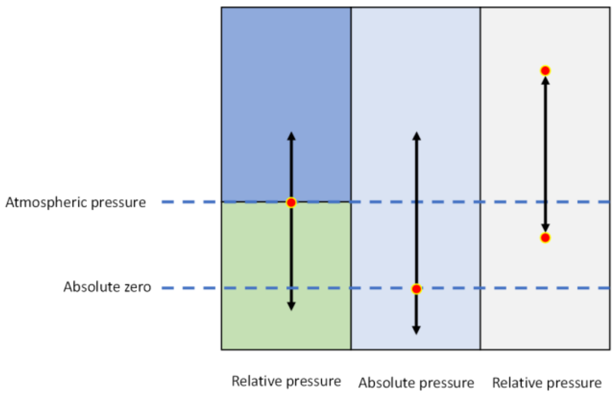


Figure 16. Pressure definitions and their references.

imposes on the soil as well as how the soil will react. It is the function of the engineer to understand the phenomena involved in each situation and thus define the quantities to be determined and/or monitored. When the soil is in a saturated state there are only two phases, namely, the solid phase and the liquid phase. In this situation the determination of the stress state, by means of pore pressure measurements, using piezometers, is quite common and relatively usual. When the problems involve soils in an unsaturated state, the mere presence of air in the voids, combined with the water, increases the difficulty of monitoring for physical reasons. This section will discuss how to choose and define the installation position of suction sensors in the field. The physical principles that are used in the measurements will be briefly given, in order to allow the reader not only to know the working principle of the sensors, but also to be able to choose, calibrate and correctly install the sensors. Examples of how soil suction is measured are presented, providing examples from the literature and highlighting for each type the most appropriate way to install the sensors and determine their optimal positioning.

5.2 *Pore Water Pressure measurement*

To understand the differences and similarities between positive and negative pore pressure measurements, it is essential to recall some fundamental concepts of pressure measurement in liquids. The measurement of pore water pressure is of utmost importance for the vast majority of engineering works. Furthermore, most of the problems that geotechnical structures face are associated with variations in the pore-pressure of water. In the early days Galileo Galilei made important observations for the knowledge of pressure in liquids. With the development of a hydraulic pump by Galileo at the end of the 16th century, he noted the limitation of his invention in raising water to a maximum height of 10m. This limitation was never explained by Galileo, who nevertheless recorded this observation. Studies related to atmospheric and liquid pressure measurement, developed by Galileo, Torricelli, Viviani, Perier, Von Gueriche and Hooke, led Boyle to establish the law between pressure and air volume at the same temperature. In 1849, Bourbon obtained a patent for what is known to this day as the Bourbon gauge pressure. But in 1733, an unprecedented pressure measurement was taken by Stephen Hales. He was the first to measure blood pressure, although it was on a horse. The measurement was made with what we know today as a piezometer (Figure 15). It is noteworthy that in this case the response time of the measurement system must be immediate.

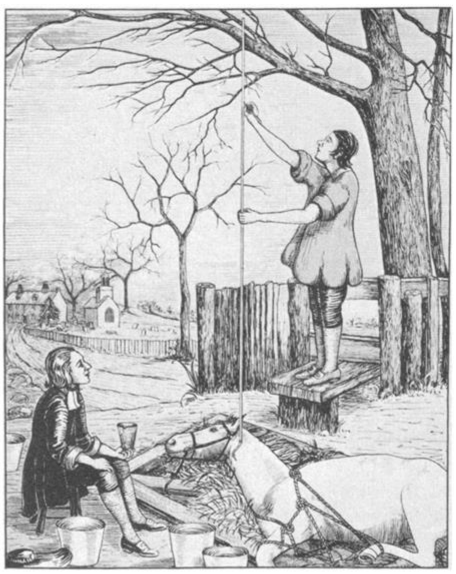
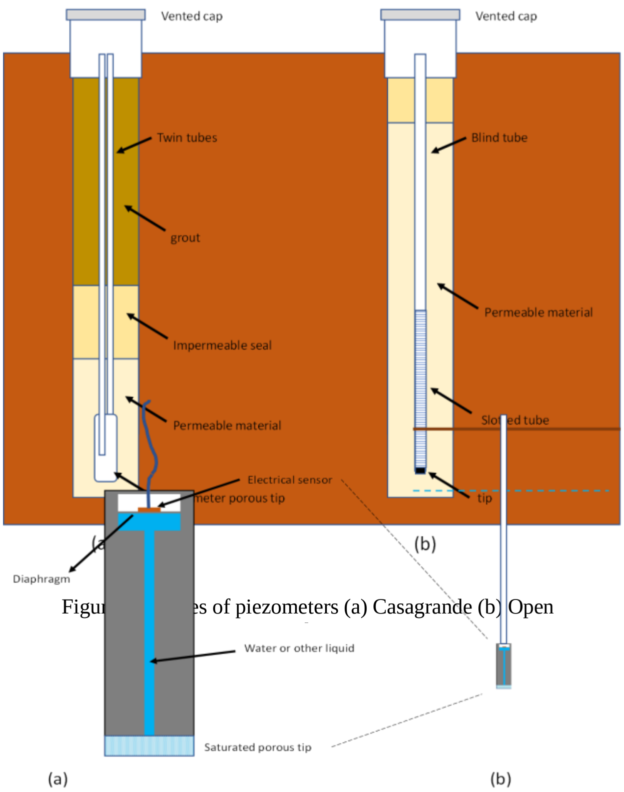


Figure 15. Illustration showing Stephen Hales performing a blood pressure measurement on a horse.

It is important to understand the principle enunciated by the one who, nowadays, gives the name of the pressure unit we use. Pascal's principle states that in a fluid within a closed system, any change in pressure at any point in the fluid will be transmitted to all points in the fluid as well as to the fluid-containing system. This principle is valid for both positive and negative pressures, keeping within the limitations associated with cavitation, observed, although not fully understood by Galileo.

Measured pressure values are referenced to absolute zero or atmospheric pressure. When measuring absolute pressure, the sensor must have vacuum at the back of the sensor. In the case of relative pressure measurement, the pressure varies from one location to another as the pressure is corrected for level conditions. Air (atmospheric) pressure decreases with increasing altitude. This pressure is the relative pressure or pressure relative to the level where the measurement is made. Figure 16 illustrates the different ways to interpret the measured values. Pressure



Figu… s of piezometers (a) Casagrande (b) Open

Figure 18. (a) Pressure transducer with a porous tip (b) piezometer installed in the field.

values below atmospheric pressure measured in porous materials such as soil are called suction and can exist even for values below absolute zero.

Positive pore water pressure measurement requires continuity between the sensor and the water. Casagrande or standpipe-type piezometers record the piezometric pressure or the water level, respectively. Measurements are made on the surface by measuring height, or the pressure at a given point. A schematic drawing of these piezometers can be seen in Figure 17. Piezometers without connection to the atmosphere measure pore water pressure using pressure transducers that require an interface between the soil pore water and the water of the transducer.

Figure 18 schematically illustrates the tip of a piezometer with a pressure transducer. This system is composed of a porous element that will establish contact between the soil pore water and the water within the transducer (for the case of a tensiometer).

The placement of piezometers for measuring positive pore water pressure is intuitively straightforward and requires the sensor to be positioned below the phreatic line. Bearing in mind that in situations where the piezometer is installed in unsaturated soil, but the expectation is to measure positive pressure, equipment that allows re-saturation of the system should be chosen. If the system has the presence of air, inaccurate values may be measured and the response time is impaired.

The piezometer that allows the measurement of negative (relative) pore water pressure is named tensiometer. According to Or (2001), the inventor of the tensiometer was the American plant physiologist Burton E. Livingston. In 1908, Livingston developed a system to control the soil moisture available to plants, which was perfected in 1918, and which has all the elements of a tensiometer, and used nowadays.

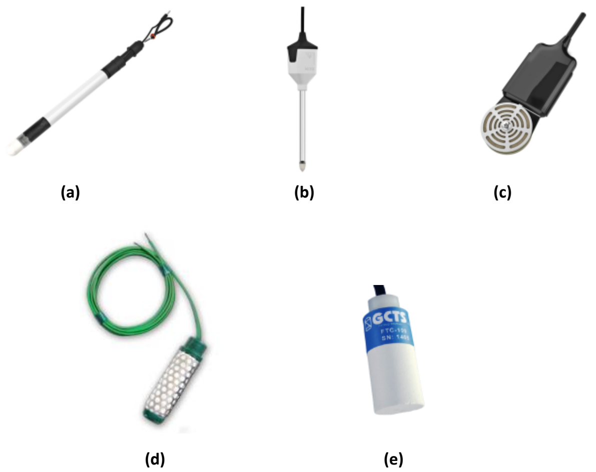


Figure 19. Main sensors for suction measurement and field use, (a) Conventional tensiometer (b) High capacity tensiometer (c) Capacitive sensor (d) Electro-resistive sensor (e) Thermal sensor.

Determining negative water pressure does not necessarily require the sensor to measure the water pressure directly, nor does it necessarily require the measurement to be taken with the sensor in physical contact with the soil. Field measurements generally measure matrix suction. The measurement principle can be divided into two:

- Direct measurement of water pressure using tensiometers that use a porous element as the interface between the pore water and the transducer water, in the same way as piezometers. These sensors, called tensiometers, generally do not allow suction measurements beyond 1 atm. There are some tensiometers which have been designed to allow suction measurements greater than 1 atm, as will be mentioned later. Other sensors that use the osmotic principle are also promising (e.g., Bakker et al., 2007; Rahardjo et al., 2021).
- Measurement of some characteristic (e.g., electrical conductivity, capacitance, thermal conductivity, etc.) associated with suction, measured not necessarily in the soil, but in a porous element with a well-defined characteristic that allows establishing a pressure equilibrium with the soil. These sensors use porous elements in which the quantities related to suction are measured. As these quantities, in general, require the material to be in an unsaturated condition, measurements for very low and very high suction values are compromised.

In all situations there will be a flow after installation, between the soil water and the sensor, or vice versa. It is this transfer process that determines the sensor response time and the applicability of the sensor to specific cases. Ideally a method for measuring the negative pressure of soil water (suction) should have the following characteristics:

- Calibration must be easy to verify and be reliable over time and for each sensor
- It must guarantee the measurement of the suction type to which it is specified
- It must allow measurements at suction levels suitable for each case
- It must be easy to use and economically viable
- It should require little or no maintenance.

Referring once again to Figure 16, we could separate the sensors that measure pressure below atmospheric pressure (they would be the tensiometers) from those that measure above (the piezometers). However, this separation would be too simplistic and would not take into account the advances that are already observed in the pore pressure gauges that are available. Of course, sensors that measure pressure below atmospheric pressure play the role of tensiometers. However, the same sensor can measure both positive and negative pressures. Tarantino et al. (2008) present a review of the types of piezometers and the development of sensors that allow measuring both values above and below atmospheric pressure and even below absolute zero. Details of the functioning of tensiometers that measure pressure below absolute zero can be found in the literature (e.g., Marinho et al. 2008).

5.3 *Sensors for Measuring Suction in the Field*

In pore pressure measurements, whether positive or negative, there will be an interaction between the sensor and the pore water. In both cases, the pore water characteristic can influence the measured value depending on the characteristics of the sensor used. However, as pointed out previously, one of the most important aspects for these sensors is the response time, which depends on the operating principle of the sensor, the hydraulic characteristics of the soil and the interface that facilitates the interaction between the soil water and the water in the sensor. Suction measurement for use in warning systems can only be considered if tensiometers are used. This is due to the response time of this type of sensor, which is the smallest of them all.

Table 11 presents the main sensors used to measure suction in the field, its range and working principle. All sensors listed are commercially available. The HCT can be found with a higher measurement capacity (up to 1.5 MPa), but it still requires some adjustments to allow its use in the field.

Figure 19 illustrates the five sensors referred to in Table 11. The tensiometer shown in Figure 19a has the transducer close to the ceramic capsule, however the other models use transducers that are on the surface. The high capacity tensiometer illustrated in Figure 19b allows for larger suction measurements than the conventional tensiometer. Other models can also be found with a greater measurement capacity. However, the process of saturation and maintenance of capacity in the field is still a challenge. Figure 19c shows a capacitive sensor, which has a low maintenance requirement and a very wide suction measurement range. The granular matrix sensor (Fig. 19d) is very robust and very interesting for use at great depths. Figure 19e shows the thermal conductivity sensor.

Table 11 – Suction measurement sensor for field measurement and its range.

Sensor	Suction range	Working principle
Tensiometer	0 to -1 atm	Water pressure
HCT	0 – 150 kPa	Water pressure
GMS	0 to – 200 kPa	Electrical resistivity
Teros 21	5 – 100 MPa	Capacitance
Thermal Conductivity	1 kPa to 1 MPa	Thermal conductivity

All sensors that use an indirect way of measuring suction, need the presence of a porous element that will serve as a reference to convert the measurements made according to the operating principle of each one. As such, the porous material incorporated into the sensor has an unknown amount of hydraulic hysteresis, causing differences in relationships between suction and the measured quantity (e.g. dielectric permittivity, resistivity) for wetting and drying. In general, the calibration used by the manufacturers is associated with the drying path. In this way, it is essential to check the hysteresis that each sensor of this type can have, and in particular the scanning curves. This verification must be carried out according to the level of seasonal variation that is expected in each case according to the position of the sensor in the active zone (further described). Some examples of studies and comments related to hysteresis in sensors that use ceramics or other porous material are presented by several authors (eg Campbell and Gee, 1986; Feng et al., 2002; Bulu and Leong, 2008; Yates and Russell, 2022; among others).

Suction is the negative soil pore water pressure; however, it is not necessary to measure the pressure directly, it is possible, and often necessary and useful, to determine some variable that is in equilibrium with the pore water pressure. For a deeper understanding of this concept, the reader is referred to the work of Edlefsen & Anderson (1943).

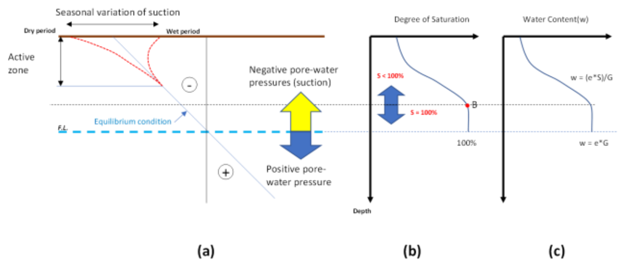


Figure 20. (a) soil suction profiles at equilibrium showing the profiles for dry and wet seasons (b) SWRC in terms of degree of saturation (b) SWRC in terms of water content.

5.4 *Fundamental Aspects for Monitoring Suction in Unsaturated Soils*

Selection of an appropriate sensor for the measurement of suction depends on the expected range of and location/depth at which suction is to be monitored. In the same way piezometers are installed in positions of interest for the project, the suction sensors must also be positioned depending on the interest of the project, but also on the expected response, so that the values do not deviate from the limits of the sensor used. It should always be kept in mind that the systematic maintenance of any sensor used to measure suction can compromise the monitoring project. Thus, it is essential that the choice of sensor and its positioning are analysed together.

Figure 20 presents, schematically, a soil profile indicating the pore-water pressure profile and the retention curve, in terms of degree of saturation and water content. In Figure 20a, in addition to the equilibrium profile (no infiltration and no evaporation), the hypothetical profiles associated with the dry and rainy seasons are also indicated. Linked with the profiles, Figure 20b shows the relationship between the degree of saturation and depth, while Figure 20c presents the variation of water content with depth. It should be noted that the depth in this case refers to the suction when the system is in equilibrium (suction = $-\gamma_w \times depth$).

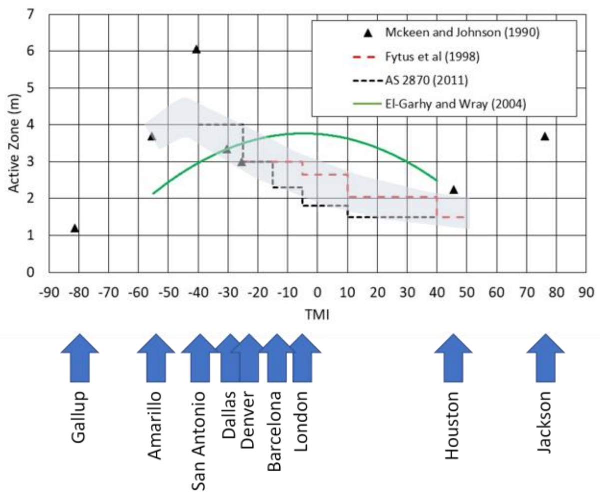


Figure 21. Association between TMI and active zone depth.

The importance of understanding the suction profiles and how they present themselves in each case is fundamental for the choice of the sensor and its positioning. As shown in Figure 20a below the phreatic surface the soil is saturated and with positive pressure and that above this line the soil has negative pore water pressure. It should be noted, however, that even above the phreatic surface the soil can remain saturated and with negative pressure. This makes pore pressure measurements unfeasible when using piezometers that are not capable of measuring negative pore pressure. The quantification of this capillary saturated range can be made using the SWRC (e.g., Nadai et al. 2022). Figure 20b illustrates how the separation point between the saturated and unsaturated condition is determined using the SWRC. The height of saturation by capillarity can reach tens of meters and the definition of this height can be fundamental for the instrumentation planning in projects. The capillary rise height is related to the air entry suction of the material, as indicated by Kumar and Malik (1990). As we approach the surface, the suction profile is affected by the seasonal weather variation, establishing what is called active zone, shown in Figure 20a. In order to better understand the sensor selection process, and its placement in the field, the following sections describe typical situations and how one should interpret them.

5.4.1 *Climatic condition and active zone*

It is critical to consider the expected flow pattern when positioning the suction sensors. Flow is caused by both infiltration and evaporation; and in some cases capillary rise. It is known that sensors very close to the surface can be subjected to suctions above those that the sensor can measure (see Figure 20a), generating the need for early maintenance, which in many cases can lead to sensor loss due to difficulties in removing them for maintenance. This behaviour depends not only on the structure to be monitored, but also on local climatic conditions. The climate and soil, as well as the geometry of the structure, induce the establishment of the active zone. The active zone (Hs) is the depth that undergoes seasonal variations, suction and/or moisture content, due to climate and/or vegetation. It should be noted, however, that this active zone can also be influenced by the variation of the water level, which in turn affects the height of capillary rise described below.

The choice of suction sensor positions must be such that it allows the chosen sensor to remain within the measurement range and permits the definition of suction profile over time or other information that may be relevant for the project such as the degree of saturation (via SWRC). When the phreatic surface is close to the surface (tens of meters or less) the suction profile can be inferred as illustrated in Figure 20a, and the suction can be estimated at the depth of the active zone. As we will see, the active zone varies between 2 and 4 meters in most cases. Therefore, it is possible to evaluate which type of sensor will allow the measurement of this suction. When positioning the sensor below this depth, little variation is expected with time.

Some authors present the relationship between the TMI (Thornthwaite Moisture Index) and the depth of the active zone. (e.g., Fityus et al, 1998; Mitchell, 2008) and this relationship is used in Australian standards, for example, relevant to foundations in expansive soils (AS2870, 2011). According to Mitchell (2008) climate changes, which will certainly change the TMI values in different locations, will induce a variation in the active zone that can create serious problems. The monitoring of these variations in terms of the active zone is fundamental and the measurement of suction becomes of extreme necessity in the short and medium term.

Figure 21 presents some relationships found in the literature correlating the depth of the active zone with the TMI. The purpose of presenting this relationship is to help define the ideal positioning for the installation of sensors for measuring suction. As previously mentioned, the active zone typically varies between 2 and 4 m for dry-sub-humid, wet sub-humid and humid climates, with a tendency to increase for dry-sub-humid climates. Positioning the sensor within the active zone will necessarily lead to fluctuations in measurements depending on local climatic aspects. It should be noted that vegetation, surface use and other aspects can affect near-surface oscillations.

It is usual to take the phreatic surface as a reference, as illustrated in Figure 20a. For simplification, we will adopt only two suction distribution situations with depth for discussion. One where the phreatic surface is known and another where it is absent or very deep. Situations where there may be a continuous flow of water from some source, or the existence of vegetation generate different profiles and may not establish an easily defined active zone.

The equilibrium profile of suction, as already mentioned, allows you to easily identify the suction value when there is no infiltration or evaporation. In this way, associated with the depth of the active zone, the suction values to be measured can be evaluated. Figure 22a reproduces the suction profile in the equilibrium condition, illustrating the schematic range of pore pressure variation and definition of the active zone. In principle, when placing a sensor below the active zone, the suction value is well established and directly related to the distance to the phreatic surface.

Some sensors, such as the tensiometer, generally have an installation depth limitation. Ordinary tensiometers have at their lower end a ceramic with high air entry pressure (e.g., 1 bar), and at the other end a pressure transducer or vacuum gauge. In this system, each meter of depth means a reduction in the capacity to measure suction of 9.81 kPa. In this way, the depth of installation of suction sensors must also be evaluated when choosing the type of instrument. When the tensiometer has the transducer together with the ceramic, as in the case of the tensiometer shown in Figure 19a, this problem does not exist.

As previously described, taking the water level as a reference, it is possible, in a first evaluation, to estimate the suction at the sensor installation point and thus better position the sensors, always keeping in mind the fluctuations due to the climate, in the active zone.

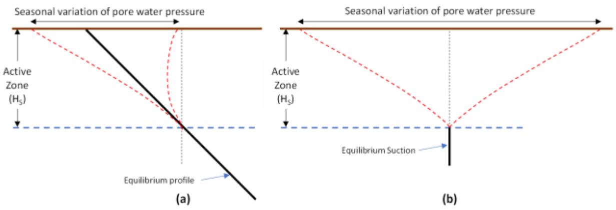


Figure 22. (a) Concept of equilibrium suction profile (b) Concept of equilibrium suction for regions with deep or absent phreatic level

When the objective is to measure suction in a location where the phreatic surface is very deep, which generally occurs in arid or semi-arid regions, the use of field sensors becomes almost impossible. Under these conditions the expected suction below the active zone is very high. Figure 22b schematically illustrates the suction distribution under these conditions and presents the equilibrium suction that is established, below the active zone. This suction is not related to the equilibrium profile described before. The equilibrium suction will still, in conditions where the phreatic surface is deep, have some variation, but as mentioned by Vann and Houston (2021) insignificant from a practical point of view. Neither these variations are significant to affect eventual settlement and shear strength calculations. In many cases where the equilibrium suction is very high, it may be more appropriate to take samples and measure the suction in the laboratory.

Russam and Coleman (1961) were among the first to establish a relationship between equilibrium suction and a climate index, in this case the TMI (Thornthwaite Moisture Index). However, as mentioned by Vann and Houston (2021), the relationship between TMI and suction is weak, as it depends on other factors such as soil type, profile heterogeneity and also surface condition. Furthermore, Karunarathne et al. (2012) and Sun (2015) showed that different TMI can be obtained through various approaches and hypotheses. Even considering all these difficulties, it will be interesting to examine the relationship between equilibrium suction and IMR. Figure 23 illustrates this relationship as obtained/estimated by several authors.

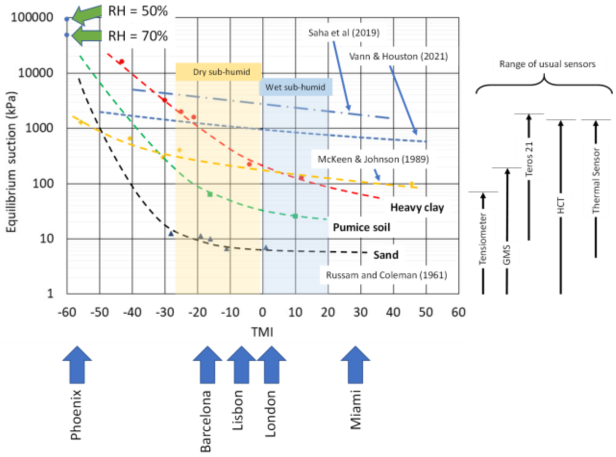


Figure 23. Relationship between equilibrium suction and TMI.

The data presented by Russam and Coleman (1961) separate the relationship according to soil type. Within this approach, it is possible to observe that more plastic soils have a higher equilibrium suction. The other authors present relationships that are associated with plastic soils and, more specifically, expansive soils. The data from McKeen and Johnson (1990), interpreted here, indicate suction values for dry sub-humid, wet sub-humid and humid climates, which range from approximately 400 to 100 kPa. As in the hypothetical condition of a very dry climate and with constant relative humidity (RH), the suction value has to come into equilibrium with the RH. The figure shows the suctions that are related to a 50% and 70% RH, which indicates that the data should tend towards these values, or another constant RH value. The curves presented by Saha et al. (2019) and Vann and Houston (2021) indicate suctions above 500 kPa in all climates. On the right side of Figure 23, the most used sensors and their measurement ranges are presented, allowing a direct evaluation of the sensor that can be used as a function of the expected equilibrium suction.

The equilibrium profile described above is partially formed by capillary effects. This, often ignored, can be of fundamental importance in the evaluation of the behaviour of structures. Depending on the material to be monitored, whether it is a landfill, a tailings bed, a dry stack or a natural soil, the presence of water at the bottom can induce a capillary rise that can reach tens of meters. It should also be noted that the phreatic surface may be tens of meters below the base of the landfill or natural slope. The association between saturated zone of the capillary rise (saturated fringe) and water infiltration can generate what can be called a pore water pressure bomb, since when the infiltration meets the capillary zone, the capillary height is converted into positive pore water pressure (e.g., Vaughan 1985, Jayatilaka and Gillham 1996). Suction measurement, performed correctly, using suitable sensors and correctly positioned, is essential to detect saturated regions with negative pore water pressures. These regions are prone to liquefaction if other features are present.

The association of water content measurement with suction measurement can significantly help in the interpretation of data and this is done using the SWRC or measuring the water content in the field. However, given the intrinsic differences between the two measurement processes, great care must be taken when installing sensors to measure water content. Two materials with different geotechnical characteristics may have equal suction values when in equilibrium, but different water content values. In this way, the suction measurement is independent of the soil on which the sensor is installed. Very heterogeneous materials can generate difficulties in the interpretation of suction values and moisture content.

Table 12. Examples of structures where suction was measured in the field

Type of sensor	Application	Reference
GMS	Railroad track	Castro et al. (2021)
GMS	Slope	Barbosa et al. (2010)
GMS	Slope	Mendes & Marinho (2010)
MPS-6	Natural ground	Tian et al. (2018)
Tensiometer	Landfill cover	Alam et al. (2019)
Tensiometer	Slope	Garg et al (2015)a
Tensiometer	Slope	Sestrem et al. (2018)
Tensiometer	Slope	Tu et al. (2009)
Tensiometer	Slope	Vieira & Marinho (2001)
Tensiometer	Slope	Yang et al. (2019)
Tensiometer - HC	Embankment	Mendes et al. (2008)
Tensiometer	Natural Ground	Silva Junior (2011)
Tensiometer - HC	Natural ground	Cui et al. (2008)
Tensiometer - HC	Slope	Toll et al. (2011)
Tensiometer - HC	Landfill cover	Maldaner & Marinho (2012)
Tensiometer and Thermal Sensors	Slope with vegetation	Garg et al. (2015)b

5.4.2 *Where to use the sensors and its requirements*

The vast majority of geotechnical structures involve soils with negative pore water pressure (regardless of whether it is saturated or not). In many of these cases, suction controls safety or at least induces additional safety, when properly designed. There are also situations where the saturation is guaranteed, as is the case with some mining tailings. Even though the suction measurement in the field is not common, understanding the importance of suction is a key factor for a proper instrumentation system. Table 12 lists some examples of use of suction measurement sensors in geotechnical structures.

5.5 *Summary remarks*

Monitoring engineering works with a focus on determining suction is an unusual procedure. However, more and more the climatic aspects have demanded monitoring to establish the suction profile. In addition, many situations require an understanding of the water pressure situation. This section presented the necessary aspects for the choice and positioning of suction sensors that can be applied to any type of engineering structure. The main aspects to be highlighted are:

- The available options of sensor for suction measurement allow measurement in a range well suited for engineering use.
- The position of the sensor is of fundamental importance not only for the proper functioning of the equipment but also for its correct interpretation.
- Understanding the possible suction profiles is one of the fundamental aspects for choosing and positioning the sensors.
- The local climate plays a key role in the suction limits to be measured. The preliminary assessment based on the possible profiles helps in the correct definition of the monitoring system.
- Of the sensors presented here, the tensiometer is the most accurate and precise. However, it is the one that requires the most maintenance when used in the field.

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