Shear Strength in Terms of Coulomb C-Intercept

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Abstract

 Foundation design is conventionally practiced by deriving the allowable bearing capacity in terms of shear strength and compressibility. Shear strength refers to the individual or combined contribution of drained angle of internal friction and undrained cohesion, at one of the three instances of peak (failure), residual, or critical (softened) states. Compressibility refers to the elastic and consolidation settlements; the predominance of which is a factor of soil permeability. Soil is conventionally examined by confined stressing on the wet stress-state surface, for the coefficient of volume compressibility to be determined. Ultimate bearing capacity is then reduced to a certain total settlement, and an arbitrary factor of safety of 3 is finally applied to cover uncertainties. Such a conservative approach however does not always successfully restrict the settlements beneath foundation. Although the shear strength has become a much-practiced subject, there has been a certain lack of emphasis on a fundamental understanding of shear strength parameters, and even greater neglect of the correct choice of stress path for individual soil types so to derive such parameters in the most critical stress-hydraulic environment. For that reason, this work aims to critically review the shear strength literature for a better understanding of the internal angle of friction and c-intercept.

Key words: Cohesion, Internal angle of friction, Understanding, Practice

1. Introduction

 The measurement of shear strength parameters by in-situ and laboratory examinations has been the endeavour of many over the past two and a half centuries. Shear strength is proportional to the normal pressure applied to a soil, and a pair of diagrammatically developed phi and c' indices, which afford a ready measure by simple laboratory set-up. Yet, a fundamental understanding of shear strength components, the bonding forces and mechanisms is a matter of uncertainties. This has led engineers to apply a suite of arbitrary safety factors, which may or may not satisfy the structural requirements, but certainly challenges the finance and sustainability of development projects. For a better understanding of such indices, a state-of-the-art review of theories is brought in this paper, illustrating the efforts applied to the problem since 1776, spanning the early geological classics to geotechnical classics, critical state and the micromechanical perspectives. In particular, a fundamental understating of the c-intercept is underlined.

2. Cohesive and Frictional Relations

2.1 Clastic Mechanics Perspective

Load is naturally carried by elements of relatively higher stiffness. This principle laid the foundation for a handful of geotechnical concepts, in theory and in practice, including consolidation (Lambe & Whitman 1969), grains' continuous breakage (Bolton 1999), and relevance of the ratio of ground anchors' length to wall surface area with reinforced slopes (Assadi et al., 2009).

In a soil system under stress, water first takes up the stress due to its higher stiffness than that of solids. Stress in water flows in the form of an excess pore water pressure. This stress gradually is delivered to solids, made up of quartz grains paired together by clays, salts, oxides, and capillary water, by means of drag forces (Assadi & Jefferson 2014). Stress in solids flow in the form of skeletal stresses. Finer particles take larger shares of force, while coarser particles take up lesser shares of force (i.e. same stress operating on a reduced area). Therefore, fines continuously split to finer fragments of more angular texture (Assadi et al. 2013, Assadi & Jefferson 2013, O'Hara-Dhand et al. 2013). This agrees with the work of Burkalow (1945), where he showed the inverse relationship between the angle of repose (as introduced in Coulomb (1776)) and grain's size. Continuous breakage of fine fragments gives rise to the angle of repose and hence the internal angle of friction (Lawson 1915, Smith 1968). An application of such concept is the design of support systems for failed retaining structures. Although such structures (e.g. soil nail wall) carry a minimal residual undrained cohesion, an enhanced drained degree of friction can be considered in models.

2.2 Geochemical Perspective

Since the triggering mechanism of subsidence is largely associated with the modification of particle contacts, and given the significant contribution of bonding forces to the integrity of contacts (Assadi & Jefferson, 2014), the 'true cohesion' deserves to receive first consideration. Seven levels of cohesion are briefly outlined: (1) Inter-crystalline cohesion in clay, (2) adhesion between quartz and clay, (3) adhesion between quartz and oxide (4) adhesion between oxide and sulphate, (5) adhesion between clay and oxide, (6) adhesion between clay and carbonates, and (7) chemically weathered clay.

2.2.1 Clay-Clay

Whilst water film tension 'partially' confers plasticity on wet clayey (or fine quartz) particles, plastic clay particles are capable of adhering one to another. Hardy (1925) gave details of the random arrangement of surface molecules of hydrophilous clay flakes and the consequent surface negative net charge. The resulting high adhesion energy in presence of cations or water dipoles accounts for the mutual coherence between moist clay particles. Clay-to-clay adhesion takes momentum with an increase in the exchangeable ion charge (Saejiew et al. 2004) at the loosely bound adsorption, viscous capillary, and free bulk water phase. Lal (2005) explained the latter process by describing the coagulation when pairs of clay flakes approach together (also see Van Olphen & Waxman (1958)). This encourages the electric double layers to either overlap or repel. Overlapped cation clouds (when cloud is pushed to the clay flakes' surface) form when electric conductivity (EC) is increased (by an increase in electrolyte concentration after Waxman & Smits (1968)) or when a divalent cation replaces a monovalent cation (e.g. the ingress of Ca^{2+}-enriched water into Na^+-enriched soil solution). The reduction in the cloud thickness results in an increase in attraction forces. At a critical EC (or a critical flocculation/coagulation concentration abbreviated as CFC or CCC), at which the Van der Waals attraction forces overcome the repulsive forces (Van Olphen 1977), rapid coagulation takes place.

2.2.2 Quartz-Clay

Bonding forces are not solely limited to clay inter-platelet forces, since these also develop through the adhesion between clay and quartz minerals. According to Hardy (1925), no properly inert solid surface exists. Crystalline quartz in comparison with amorphous quartz or hydrated silica exhibits much lesser adhesion surface energy due to its more regular orientation of atomic groups on its boundary surface and consequently the limited extents of electrical surface charges. However, common atomic groups and thus actual chemical/structural relationship between quartz and clay substances provide some extents of binding, which potentially results in the formation bonding.

2.2.3 Quartz-Oxide

Quartz and amorphous silica interacted physically. Silica coats act as inter-particle beddings. According to microscopic observations of the author, fast re-precipitation of silica (i.e. in a hot environment) form spherical sub-rounded flocs (in agreement with Singh & Gilkes (1993)). These were accommodated in inter-particle trapdoors and macro-pore spaces. Flocs (see Fig. 1) are highly porous and thus highly permeable.
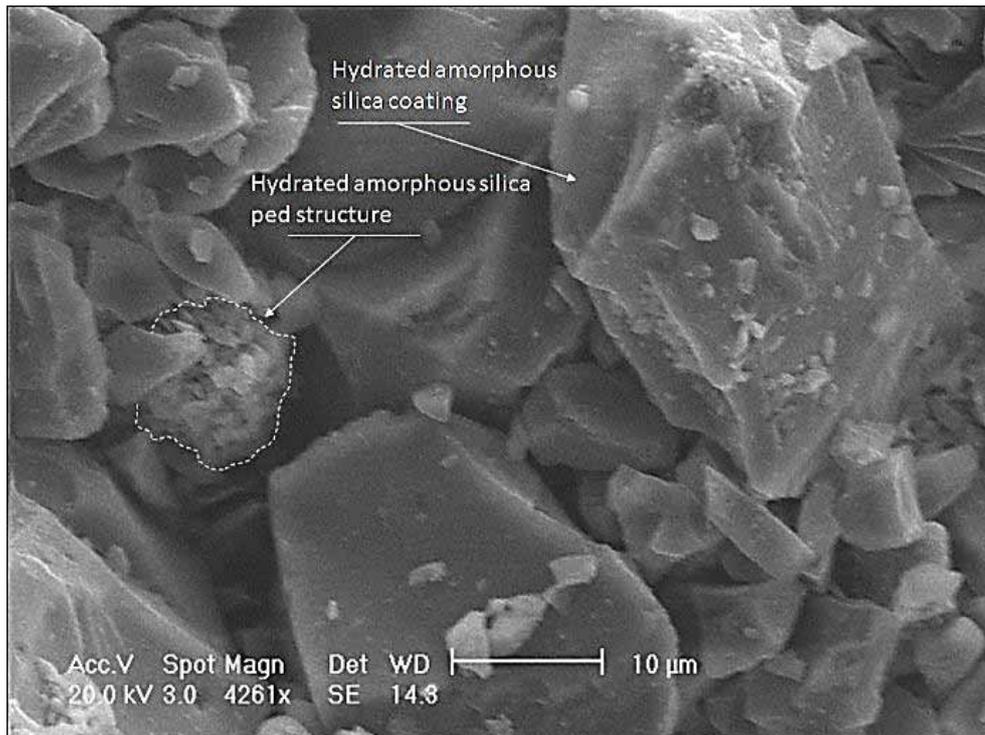
Fig. 1 Horizontal alignment, silica-indurate silt in hot environment

Flocs carried larger skeletal forces than laminated silica units, as skeletal stresses convert into compressive stresses on the flocs' spherical surface. Upon an increase in the stress level, grains rotate, and hence shear stresses at grains' contact points with stationary neighbouring flocs develop. The stress then travels to the adjacent out-of-plane grains through sets of compressive surface stresses. In contrast, slow re-precipitation of silica (under mild climate) form laminated coat units. Overgrowth of coats generates buttress structures, which are much susceptible to shear failure (Fig. 2).
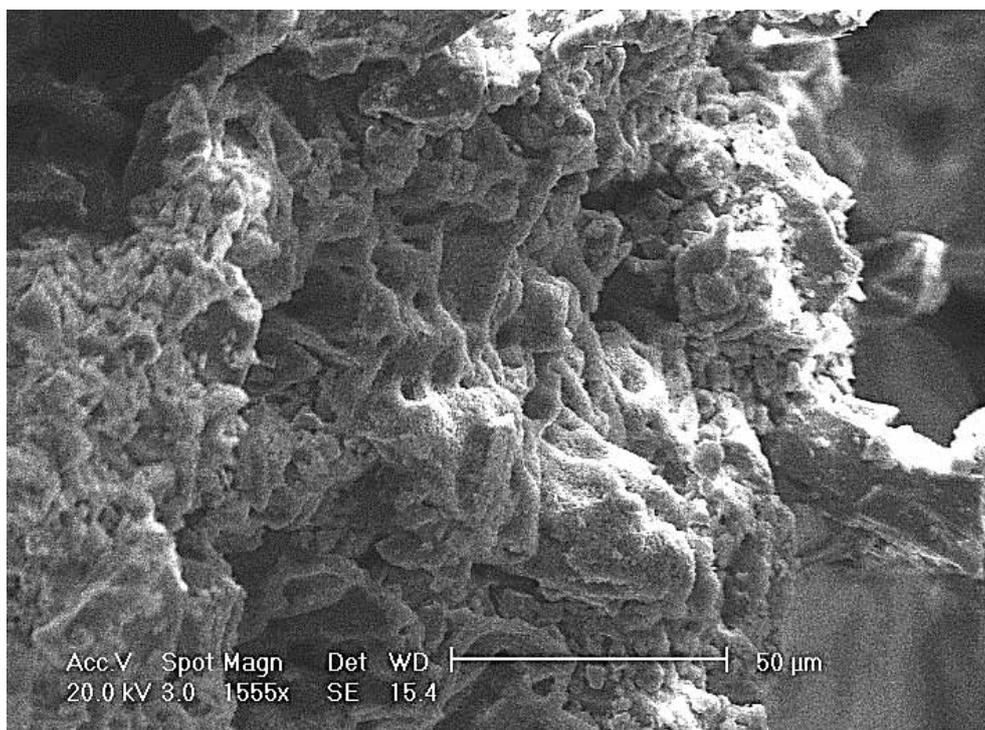
Fig. 2 Vertical alignment of silica-indurate silt in mild environment

2.2.4 Oxide-Sulphates

In presence of $[M - SO_4^{2-}]$ phases, sulphates bridged either fully hydrated $[H_4SiO_4]$ or partially hydrated $[SiO_2.OH]_n$ clusters: While neither Na+, Ca2+, and Mg2+ ions (M) could be exchanged with H+ of hydroxyl tail (cation exchange), the [-OH] can get released and replaced with $[SO_4^{2-}]$ via a ligand exchange (anion exchange). The release of hydroxyl gives rise to the pH. The basic environment encourages the de-protonation of the silica – sulphate chain. Consequently, donation of H+ ion by the hydroxyls slightly decreases the pH. Presence of free Na+ ions in the soil solution keeps the environment slightly alkaline.

The net negative charge of the resulting compound and the slight alkalinity of the soil solution needs to be balanced. Therefore, upon a second ligand exchange ([-OH] with $[SO_4^{2-}]$), and the sodium-oxygen bond formation, the charge balances and the environment attains a neutral pH.

2.2.5 Oxide-Clay

Layers of stacked H4Si2O6 orthosilicate anion share bond with kaolinite Chrystal (see also Tossell & Vaughan, (1992)). As demonstrated in Eq. 1 (dashed area), silicon cannot share covalent bonds in reality and hence carries a net negative charge, which allows it to share chains of Si-O-Si (also see Fig. 3).
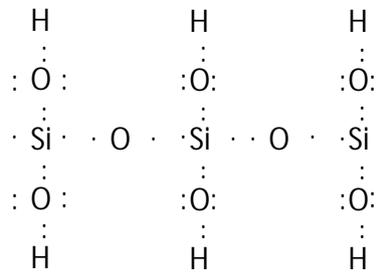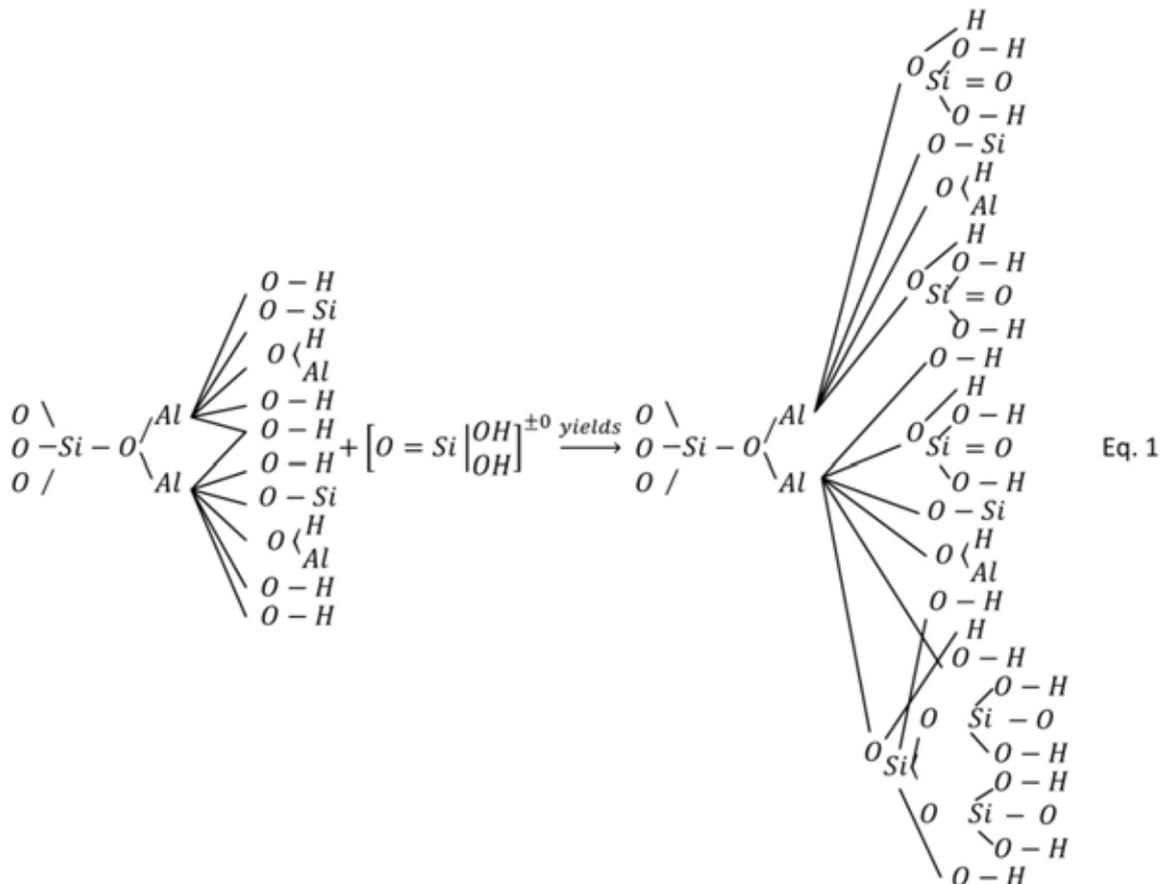

Fig. 3 Si-O-Si-O Rings on Lewis structure scale

Eq. 1

2.2.6 Carbonate-Clay

The ingress of calcium-enriched electrolytes into a kaolinite matrix triggers short-term and long-term pozzolanic reactions (Broms & Boman 1977, Little 1995). Pozzolanic reaction is a suite of ion exchange events, specifically associated with pH-dependent charges on clays' edge: The pH-dependent positive charge on the edge of Kaolinite flakes can change into negative after an increase in the pH of soil solution and the subsequent deprotonation at hydroxyl sites (Van Olphen 1977). Van Olphen (1964) and later Mitchell & Soga (2005) suggested that, these charges are responsible for clay platelets' associations.

Any pozzolanic reaction needs a pozzolan, which is a source of silica or alumina. Deprotonated Kaolinite (after donation of hydrogen atom by the hydroxyl tail in an alkaline environment) is a pozzolan. The reaction also needs a source of calcium ion. This can be supplied by the residual calcium within the soil. $Ca2+$ exchanges with monovalent $Na+$ at the stern layer. Liberated $Na+$ reacts with the free hydroxyl anions, giving rise to the pH of the environment. High pH encourages the liberation of hydrogen atoms from clay's hydroxyl tail (i.e. deprotonation). Excess calcium ions then attach to exposed silica sheets to form Calcium Silicate Hydrate (CSH) gels, as well as exposed alumina sheets to form Calcium Aluminate Hydrate (CAH) gels. These strongly cement the clay clusters.

2.2.7 Weathered Clay - Quartz

Chemical weathering of residual clay may result in the formation of aluminium polymers and secondary sesquioxides. Krebs et al. (1960) showed that the interaction of acids with inactive clayey inter-particle bonds (e.g. Kaolinite $Al_2Si_2O_5(OH)_4$) may lead to precipitation of aluminum ions and basic salt polymers at the location of parent clayey bonds. Boardman et al. (2001) explained the interaction of Kaolinite with cations in alkaline environments. Assadi & Yasrobi (2007) showed the delamination of aluminosilicate sheets in active clays in acidic environments, which then produced aluminum/magnesium based salts. From a different perspective, Boardman et al. (2001) showed the Liquid Limit and plasticity increase in either calcareous or non-calcareous loess when treated with a solution of iron chloride and sodium chloride. Enhanced plasticity improves the bonding role of sesquioxides in the loess.

2.3 Physical Perspective

The yield of a retaining wall is associated with the sliding of a wedge shaped body of soil. When a wall is pulled out, a talus of soil with a certain slope forms in the absence of water. Any increase in the slope of the meta-stable talus results further movements. Given the classic definition of friction ($F_{friction} = \mu_s . F_N = \mu_s . mg . \cos\theta$, $F_N = mg . \sin\theta$), Coulomb (1776) concluded the tangent of angle of repose (θ, relative to horizontal surface) equals the coefficient of static friction (μ_s), $\tan\theta = \mu_s$. Smith (1968) noted that with a gentle increase in θ to a state at which sliding appears imminent, θ gets the φ value, where φ is the angle of friction (also known as the Lawson hypothesis in Lawson (1915)).

Since the shear resistance equals the friction resistance ($\frac{S}{A} = \frac{F_f}{A} = \mu_s . \frac{F_N}{A}$), and given the fact that shear resistance of water is zero ($\frac{S}{A} - 0 = \mu_s . \left[\frac{F_N}{A} - 0\right]$, or $\tau - u_w = \mu_s . [\sigma - u_w]$), the linear relation between shear stress and effective stress can be phrased as $\tau = \mu_s . \sigma'$, where σ' is the normal stress on the surface of sliding (Smith 1968).

The equation shows the significance of the 'apparent cohesion' in terms of negative effective stress. Under zero external net stress ($\sigma = 0$), the suction (i.e. water potential or the negative pore water pressure) provides the positive effective stress ($\sigma' = 0 - -u_w$). This is analogous to a contact-level bonding force, which induces a certain degree of shear resistance. The bonding effect of water films was initially appreciated in the work of Terzaghi (1920a), where he showed the contribution of a >50 x 100 μm capillary film to the shear strength of an impermeable clay. Considering his 1938 definition of cohesive soils, it may be concluded that a soil functions as a cohesive material only at certain low water contents. The effect of negative pore water pressure can also be re-written in terms of the c'-intercept. The intercept can be added to the linear form of shear stress – normal stress equation ($\tau = \mu_s . \sigma' + c'$). For silts and clays, Smith (1968) defined the c-intercept as the shear strength under zero normal stress which stands for the mutual attraction between soil particles. Burkalow (1945) however argued the equal angle of repose and angle of 'sliding' friction, stating that this assumption only applies to physics but not geology. Burkalow (1945) showed distinctly lower values of friction angle than angle of repose for identical soils. Many attempts have been reported since then to correlate the angle of repose to the angle of friction (see Ghazavi et al. (2008)). The assumption was also argued in the work of Terzaghi (1920b), where he stressed on the effect of grains' rotation (i.e. movement) at the moment of sliding, which is strictly under-reported

in Coulomb's theory. Rotational frustration (see Santamarina (2003)) increases the angle of repose.

2.4 Continuum Mechanics Perspective

Stress can be defined at a geometric point in a given continuous medium (continuum). Such medium does not contain irregularities in molecular structure, gaps or empty spaces (Malvern 1969). Considering an axially symmetric stress-strain state ($\sigma'_x = \sigma'_y$ and $\varepsilon_x = \varepsilon_y$), normal and shear stresses can apply to a given point, the value of which varies with the orientation the plane which passes the point. Mohr circle is used to illustrate the stresses on all the possible orientations. For a given point in soil, stress state can be shown on σ - τ space via two points of ($\sigma = \sigma_h; \tau = 0,$) and ($\sigma = \sigma_v; \tau = 0$). A fitting circle to these points is termed the Mohr circle. A plane at 'α' degrees orientation to the horizon in specimen can be illustrated by a plane at '2α' degrees orientation to the horizon on the Mohr circle, carrying a set of normal and shear stress, which actually operates on the point of interest in the specimen. 'α' is termed 'angle of repose'. At a constant $\sigma_h = \sigma_{cell}$, any increase in σ_v generates a larger circle, and thus a higher maximum shear stress. However, not every combination of $\sigma_1 = \sigma_h + \sigma_v$, $\sigma_3 = \sigma_h$ can result in an α as large as φ (internal angle of friction). Under a correct combination of σ_1 and σ_3, an inclined plane of $\tan\theta$ slope gives an α value of φ content, and the $\sigma_N - \tau$ falls on the Mohr strength envelope, representing the shear strength of the material. The angle of repose at failure ($\alpha = \varphi$ angle of friction) and shear plane inclination (θ) are directly proportional through the expression $\theta = \frac{\varphi}{2} + \frac{\pi}{4}$. The shear plane inclination is applied in many engineering tasks such as the degree to which an active slip line is considered in the design of the anchored support of excavation walls (FHWA 1999).

3. Shear Strength in Classic Soil Mechanics
3.1 Drained Condition

Drained condition is conventionally believed to provide higher levels of shear strength in soils. The drained behaviour of soil was initially discussed in Terzaghi 1920 (Skempton 1960), where he stressed on the necessity of a hydraulic equilibrium state for the mobilization of maximum shear strength. Full consolidation prior to shearing provides significantly higher contents of φ' (drained internal friction angle) at the expense of significantly lower contents of C'. Terzaghi later in a series of articles (1925, 1927 and 1938) described further the term 'full

consolidation'. In 1925, he acknowledged the relevancy of internal friction angle with water content in clay (Skempton 1960). Terzaghi and Janiczek in 1927 restricted the direct relationship between φ' and water content (consolidation history) to normally consolidated clays (also in Terzaghi (1938)). With granular soils, he linked the internal angle of friction to density instead of water content. The FHWA code recommends drained testing condition for stiff to hard over-consolidated clays for the design of both temporary and permanent soldier walls. The standard then suggests the use of fully softened shear parameters ($c = 0, \varphi_{res} < \varphi' < \varphi_p$) as the 'C' intercept at the peak value rapidly levels down with a small increase in the shear strains beyond peak.

Within a micro-mechanical perspective, mobilization of skeletal forces through dissipation of the pore water pressure at micro-pore space (see Khalili et al. (2005) and Assadi & Jefferson (2014)) and therefore the drag force, accounts for higher inter-locking effect (the β component of internal friction angle). In addition to the contribution of skeletal forces, void ratio loss upon drained shearing gives rise to the coordination number (Santamarina 2003) and thus the α component of the internal friction. C-intercept on the other hand records lower contents as the excess pore water pressure disappears. According to Rao (1996), drained cohesion in unsaturated (i.e. under pore water pressures less than the atmospheric pressure) soils (i.e. Cd) could be expressed by two components of true cohesion (i.e. effective cohesion at saturated state when pore water pressure equals atmospheric pressure, C'), and apparent cohesion (i.e. matric suction induced cohesion, Capp). Apparent cohesion is manifested by the increased shear strength from matric suction effect, which acts as a modified effective stress in the unsaturated medium. Apparent cohesion may disappear throughout wetting. True cohesion at grains' contact points in dry systems was explained in Smalley (1970) as a low-order chemical bond of the Van der Waals type, although its exact nature is still in doubt. With the particular reference to the clayey silts (i.e. loess systems), Smalley acknowledged the complexity of particle interactions and a fundamental difference in the nature of the contact between silt particles and between clay particles. Rao (1996) emphasised that in soils with either long-range clayey indirect bonds (i.e. high clay fraction), or medium to hardly soluble sesquioxides, true cohesion has the contribution of cements to shearing resistance is evident. The significance of true cohesion is such that even a complete loss of the apparent cohesion does not

cause the shear strength to reach a limiting equilibrium state. Boonsinsuk & Yong (1982) studied two scenarios and introduced boundary conditions for the average cohesion in unsaturated soils throughout the wetting. Within the first scenario, cohesion merely was assumed to consist of the apparent component, which conforms to the framework of critical state soil mechanics. Within the second scenario, Boonsinsuk and Yong assumed that the apparent cohesion tends to zero with an increase in soil saturation ratio while true cohesion remains independent of the degree of saturation. This is partially coincident with views in the classic soil mechanics. However, given the varied soil composition, the true cohesion may diminish upon wetting, after the dissolution and/or softening of bonds followed by fine migration, which eventually causes the collapse of trapdoors and the pore network. Non-soluble cements may also modify and migrate (Assadi & Jefferson 2013) into macro-pore spaces upon wetting. This modification of bonds upon wetting led Derbyshire et al. (1995) to study the role of true cohesion associated with chemical cements, at high suction values in loess deposits. They divided total cohesion into 'primary' cohesion and 'consolidation-induced' cohesion. Primary cohesion was attributed to the act of inter-particle clayey bonds while consolidation-induced cohesion was ascribed to depositional and post-depositional formed chemical cements including calcium carbonate, magnesium sulphate, gypsum, and sodium chloride. Derbyshire and colleagues claimed that high matric suction enhances both the consolidation-induced and apparent cohesion. On the other side, whilst suction decrease initially mobilizes the apparent cohesion, it eventually reduces both the consolidation-induced and apparent cohesion. Although the drained condition was shown responsible for the enhanced frictional behaviour, it was earlier discussed that such condition mostly applies to normally consolidated soils. The micro-mechanical background for this in centred on the permeability. The cohesive behaviour of over-consolidated soils is felt to be the effect of lowered permeability which remarkably retains the pore water within the pore network and thus preserves the early high content of drag forces at the expense of lower contents of skeletal forces.

3.2. Undrained Condition

The c-intercept under undrained conditions is made up of true cohesion (Cu). Shearing at high strain rates (flow rates more than the permeability) slows down the dissipation of the drag forces (FD) and thus mobilization of skeletal forces. Given the fact that inter-locking force is a component of skeletal force, less frictional behaviour would be evident. According to Skempton (1960), Terzaghi in 1925 clearly stated that the pressures associated with undrained loading do not carry any static frictional component and that the frictional resistance emerge only after full dissipation of the excess pore water pressure (terms slightly changed to adapt to present-day knowledge). The zero content of the internal angle of friction can be explained through the undrained mechanism discussed above. Water acts as a lubricating agent, minimizing the inter-locking effect while quartz grains (and aggregated pellets) benefit from the all-round excess pore pressure which further reduces the contact stresses.

4. Shear Strength in Critical States Soil Mechanics

Within the framework of critical state soil mechanics (CSSM), soil is a two- or three-phase essentially particulate material, with or without negative pore water pressure, with negligible degrees of cementation and thus ignorable true cohesion. In other words, the intercept value of peak strength does not indicate the true cohesion component.

Schofield attributed the true cohesion to particles' interlocking in medium to dense sands and heavily over consolidated (HOC) clays (Schofield 1999). In 1998, Schofield suggested that an apparent cohesion may form in remoulded soft soils when small clay particles and chemicals give rise to the suction. Under zero total stress, suction in pores operate similar to that of positive effective stress, providing an apparent cohesion of suction times friction angle (see also Ho & Fredlund (1982)).

Attributing the true and apparent cohesion to particles' interlocking and matric suction, considering clay as a particulate frictional material and overshadowing the cementation, the bonding role of clay can be explained through:

(1) Small size of clayey particles supplies lower degrees of hydraulic conductivity, lower rate of saturation, and therefore higher durability of apparent cohesion.

(2) Small size of clayey particles supplies higher coordination number, higher number of trapdoors hosting the water menisci, and therefore higher levels of apparent cohesion.

(3) Net negative charge on clay flakes provides higher affinity for water, more water menisci units forming at contact points, and therefore higher levels of apparent cohesion.

As an example of CSSM modelling, for loess soil on the wet stress-state surface, soil is analogous to

normally consolidated (to lightly over consolidated) clay. In contrast, loess on the dry stress-state is analogous to heavily over consolidated clay. In other words, depending on the stress path, loess may either lie on the wet side of the critical state line exhibiting a long-range ductile behaviour or lie on the dry side of the critical state line exhibiting a short-range brittle behaviour. Maximum mobilized cohesion of either wet or dry loess, whether originated from direct grain-to-grain contact relations or indirect clayey-chemical bonds, should correspond with the critical state or peak shear resistance, respectively.

5. Closing Remarks

Angle of internal friction is not an intrinsic property of quartz. 'α' is a factor of grading, degree of internal crystalline defects, energy input and energy duration. Therefore, for granular young self-supporting slopes, the drained angle of friction might be slightly more than that measured on the foot or at the top of the slope. α-component of angle of friction is a factor of coordination number. Failure of slopes lead to grain crush and hence an increase in α. However, β component is a factor of inter-locking forces. β component increases from an initially low value as the excess pore water pressure (induced by stressing or water inflow) dissipates. In the light of the non-intrinsic feature of internal angle of friction components, such design value needs to be measured on correct site samples, and under correct laboratory set-up.

A correct estimation of consolidation volume change and hence shear strength parameters needs a good understanding of geochemistry of soil's bonding materials and the service environment. These factors include:

(1) The Cation Exchange Capacity and valence of clay fraction

(2) Geomorphology of the study area (e.g. a Calcium enriched solution shall be used to measure the Coefficient of Volume Compressibility for an accumulated deposit in a ditch overlaying permeable sand layer and underlying calcareous clayey silt with flint)

(3) Presence of poly-mineral bonding elements in soil (e.g. oxides, sesquioxides, carbonates, sulphates), which alter the liquid limit and therefore the critical collapse pressure. Such soils do not necessarily compress to their maximum possible under a self-weight flooding and the subsequent K0-stressing condition.

A correct draining set-up in oedometer tests requires a good understanding of soil's grading, permeability, and stress history. The c-intercept derived from a drain shear test does not represent a degree of true cohesion, but a consequence of capillary water contribution. Therefore, the c'-intercept should not be considered in engineering design.

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