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Collapse settlement of unsaturated soil from effective stress and shear strength interaction of soil

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Abstract. Consolidation settlement is the volume change behaviour due to dissipation of excess pore water pressure in saturated conditions. However, in a partially saturated condition, suction plays an important role in governing settlement behaviour. Besides, it also influences the shear strength of the soil by providing additional cohesion (termed apparent cohesion, c_s) due to the existence of suction in the shear strength against suction axis. The shear strength decreases as the soil becomes fully saturated i.e. suction is reduced to zero, thence causes the apparent cohesion reduced to zero. Apparently, the shear strength has strong influence on the volume change behaviour. Therefore, a comprehensive review of the effective stress and shear strength interaction was conducted to verify this. The objective of this research is to verify the effective stress and shear strength interaction for both saturated and unsaturated soil conditions and to evaluate the complex settlement behaviour due to inundation at constant net stress, which has puzzled geotechnical practitioners. These conditions are tested and verified using the double wall triaxial test apparatus to determine the unsaturated shear strength of soil, the pressure plate extractor to produce Soil Water Characteristic Curve (SWCC) and modified large Rowe's cell apparatus to simulate the loading and wetting collapses of partially saturated soils.

1. Introduction

Terzaghi [1] was the first to introduce the effective stress concept for saturated soil. The consolidation theory expressed is the basis of applied pressure increase on the soil where volume change occurs due to the dissipation of excess pore water pressure causing the soil skeleton to experience increase in effective stress. However, the piston and spring analogy was then limited to the idea of one dimensional consolidation. In unsaturated soil, Biot [2] presented a three-dimensional consolidation theory based on the assumption that the soil is isotropic and linear elastic. Two constitutive relationships were proposed in order to describe the deformation state, which are in relations to the soil structure and water phase. Bishop [3] attempted to link the deformation behaviour of unsaturated soil with single valued effective stress equation, however, resulted in limited success [4]. It is reported by Fredlund and Rahardjo [5] that there is no unique relationship between volume change and effective stress for most soils, particularly below a critical degree of saturation from oedometer and all around compression tests performed on saturated and unsaturated soils. The objectives in this research are: i) to verify the effective stress and shear strength interaction for both saturated and unsaturated soil conditions and ii) to evaluate the complex settlement behaviour due to inundation at constant net stress.



1.1 Volume Change Behaviour of Unsaturated Soils

Coleman [6] separated the components of Bishop's effective stress equation, and proposed a set of constitutive relationships for the soil structure and for the water phase. The proposed volume change constitutive relation associated with the soil structure is given as:

$$-\frac{dV}{V} = -C_{21}(du_a - du_w) + C_{22}(d\sigma_m - du_w) + C_{23}(d\sigma_1 - d\sigma_3) \quad (1)$$

Where: dV is the overall volume change of a soil element.

V is the current overall volume of the soil element.

u_a is the pore air pressure.

u_w is the pore water pressure.

σ_1 is the total axial normal stress (i.e., major principal stress).

σ_3 is the total confining pressure (i.e., minor principal stress).

σ_m is the mean total normal stress [i.e., $1/3 (\sigma_1 + 2\sigma_3)$]

C_{21} , C_{22} , C_{23} is the soil parameters associated with the soil structure volume change.

Matyas and Radhakrishna [7] introduced the concept of state parameters for an unsaturated soil consisting of stress variables (e.g., $\sigma_m = (\sigma_1 + 2\sigma_3)/3 - u_a$, $(\sigma_1 - \sigma_3)$, and $(u_a - u_w)$ for triaxial compression, along with the initial void ratio and degree of saturation. For isotropic compression, the stress parameters reduced to $(\sigma_3 - u_a)$ and $(u_a - u_w)$ with the void ratio and degree of saturation used to represent the deformation state of the soil. The void ratio results [7] produced a single warped surface, with the soil structure always decreasing in volume as $(u_a - u_w)$ stress is decreased or as the $(\sigma - u_a)$ stress is increased as shown in Figure 1.

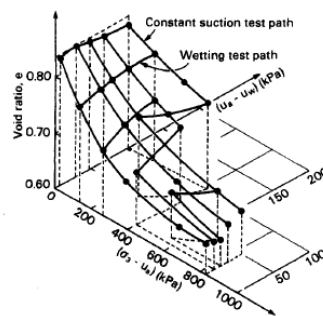


Figure 1. Void ratio constitutive surface under isotropic loading from Matyas and Radhakrishna [7].

1.2 Collapsible Soil

Alonso [8] stated that the microstructure is important to explain the behaviour of collapsible soil although it lacks simple quantitative description. Regarding the behaviour of collapsible soils, it is recognised that:

- Any type of soil compacted at its dry – optimum conditions and at low density, may develop collapsible fabric or metastable structure [9].
- A compacted and metastable soil structure is supported by microforces of shear strength i.e. bonding depending upon capillary action. With the increase of water content, the bond starts to lose strength and at full saturation the soil structure collapses [9].

- There is a gradual increase in compressibility and reduction of shear strength of collapsible soil during saturation [4] [9].
- The soil collapse progresses as the degree of saturation increases but at critical degree of saturation, the soil collapse negligibly regardless of the magnitude of prewetting overburden pressure [4].
- Collapse of a soil is associated with localised shear failures rather than an overall shear failure of the soil mass [10].
- The soil undergoes an increase in horizontal stresses during wetting-induced collapse at constant vertical load and under K_0 oedometer conditions [10].
- Under a triaxial stress state, the magnitude of volumetric strain due to the change in stress state or wetting depends on the mean normal total stress and is independent of the principle stress ratio. [11].

1.3 Loading Collapse

Alonso *et al.* [12] illustrated a suitable stress space in describing the isotropic stress states as (p, s) , where $p = \sigma_m - u_a$ as the excess of mean stress against air pressure and suction. Alonso *et al.* [12] stated that it is convenient to present a formal derivation of the yield function to underline ‘ab initio’ the relationship of yield states with observed volumetric behaviour of partially saturated soils and incorporating well established features of isotropic or oedometer compression of saturated soils.

Alonso *et al.* [12] introduced a yield curve named LC (after loading-collapse) as shown in Figure 2. A loading-collapse (LC) yield curve is when $p_o^* = p_c$, the LC yield curve becomes a straight line; therefore, any changes to suction will not result in plastic deformations. Most of the reported state surfaces have been obtained from stress paths involving reductions in suction from an initial value and a subsequent increment in net mean stress, p as shown in Figure 3.

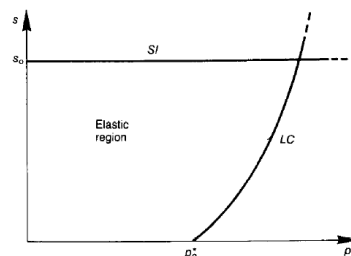


Figure 2. Loading-collapse (LC) and suction increase (SI) yield curves proposed by Alonso *et al.* [12].

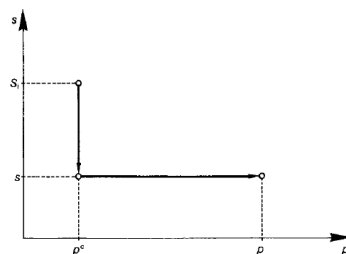


Figure 3. Wetting and loading stress path in (p, s) stress plane introduced by Alonso *et al.* [12].

1.4 Wetting Collapse

As detailed out by Md. Noor *et al.* [13] wetting collapse or inundation settlement is triggered when there is a shear strength decrease due to the loss of suction as the soil becomes fully saturated. Pereira and Fredlund [14] commonly refer collapsible soil as a metastable structured soil where an increase in pore

water pressure will result in a volume decrease for a metastable-structured soil. As specified by Barden *et al.* [9], there are four (4) factors required to produce collapse in the soil structure:

- an open, partially unstable, unsaturated fabric.
- a high enough net total stress that can cause the structure to become metastable.
- a bonding or cementing agent that stabilizes the soil in the unsaturated condition.
- the addition of water to the soil causing the bonding or cementing agent to be reduced and the inter-granular contacts to fail in shear, resulting in a reduction in the total volume of the soil mass.

2. Effective Stress and Shear Strength Interaction of Saturated and Unsaturated Soils

Md. Noor and Anderson [15] introduced comprehensive shear strength and volume change behaviour for saturated and unsaturated soils. This led to the three dimensional qualitative framework produced later by Md. Noor and Anderson [16]. Settlement has always been associated with shear strength due to the fact that shear strength governs changes to the resistance of the soil with the increase of effective stress (termed as loading collapse) or the decrease in effective stress (termed as wetting collapse). Therefore, it is vital to observe the interaction between effective stresses and shear strength which relates to collapse behaviour of both saturated and unsaturated soils.

A curved surface envelope soil shear strength model (CSESSM) was introduced by Md. Noor [17], where in the drained condition, at lower effective stress; the shear strength envelope is curvi-linear approaching towards zero at the shear strength, τ axis. Initially, this behaviour was discovered by Henkel [18], Bishop [19], Marsal [20] and Maksimovic [21] as shown in Figure 4. The term cohesion, appears to be misleading, where two (2) terms are associated with it, i.e. true cohesion and apparent cohesion. Whitlow [22] described true cohesion to be a historic cementation and overburden which happens in the pass causes the soil to develop it whereas apparent cohesion develops due to the negative pore pressure attraction that occur within the soil microstructure (termed as suction).

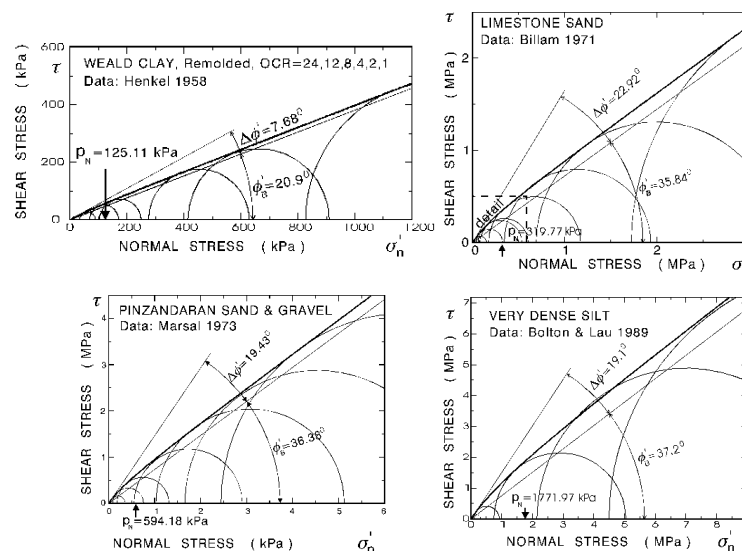


Figure 4. Nonlinear shear strength envelopes for various soils plotted by Maksimovic [21].

Interestingly, comprehensive study on the non-linearity of the shear strength against suction was conducted over large suction range using more elaborate testing equipment [23] whereby the shear strength increases at the same rate for an increase in total suction up to a certain suction value i.e. $\phi_b = \phi'$. Fredlund and Rahardjo [5] discovered that the angle ϕ_b is a function of matric suction ($u_a - u_w$) and the variation in ϕ_b is explained by considering the pore volume on which the pore water pressure acts.

Finally, Fredlund and Rahardjo [5] found that the shear strength with respect to suction increases equally to the air entry value of the soil obtained by Infante Sedano and Vanapalli [24] as shown in Figure 5.

2.1 Curved Surface Envelope Soil Shear Strength Model

In this case, the normal and shear stresses transferred to the soil grains from the applied external stress system that is the driving factor to cause settlement whilst the resisting shear stress developed between the soil grains derived from the combination of the applied external stress and the soil properties in which the mobilised internal friction angle, ϕ'_{mob} is the resisting factor. This angle is from the horizontal line from the origin tangential to the Mohr circle representing the stress state. The latter is referred to as the mobilised shear strength, τ_m , which is defined as the maximum shear stress mobilised in the soil mass enable the soil to carry applied stresses imposed to it.

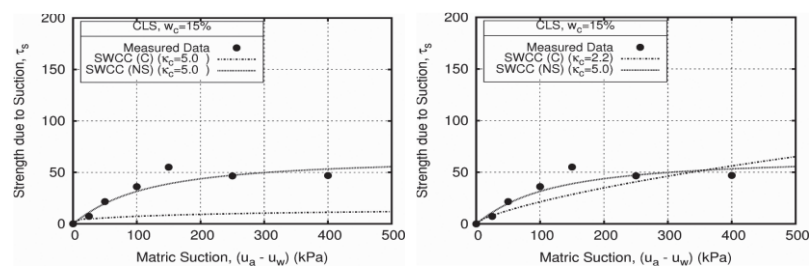


Figure 5. Measured shear strength envelope for a specimen prepared at $w_c = 15\%$ and sheared at constant load suction conditions [24].

The incorporation of the applied effective stress state and mobilised shear strength in the volume change framework will satisfy the requirement for interaction between the driving and resisting factor in the soil volume change behaviour. These values are represented by the applied effective stress Mohr circle and the mobilised shear strength envelope, respectively. Figure 6 illustrates the three-dimensional CSESSM developed by Md. Noor [17] from modified drained triaxial test conducted on gravel size soil sample.

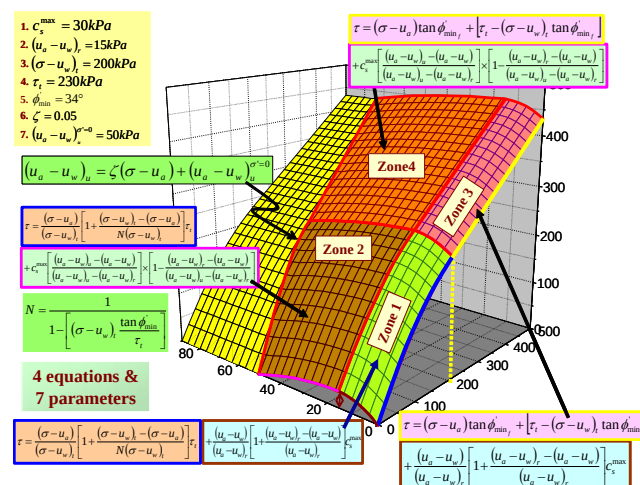


Figure 6. CSESSM developed by Md. Noor [17].

2.2 Rotational Multiple Yield Surface Framework of Saturated and Unsaturated Soils

Md. Noor [17] developed a hypothetical volume change framework to explain the volume change behaviour of coarse grained soils due to inundation and load increase. This later was proven by Md.

Noor and Anderson [25], Md. Noor and Mohamed Jais [26] and Md. Noor *et al.* [13] for Banting Clay and granite residual soil tested using modified large Rowe's cell. The Rotational Multiple Yield Surface Framework (RMYSF) takes into account the soil stress strain response and the volume change behaviour. The magnitude of deviator stress in the stress-strain response determines the major principal net stress while the net confining pressure determines the minor principal net stress and the axial strain is taken as the yield parameter.

Md. Noor [17] suggested applying the Mohr stress diagram, since it represents graphically the state of stress at any point in a soil mass by considering the combined effect of principal stresses and suction. The mobilized shear strength increases with mobilized friction angle, therefore the curvilinear shape of the mobilized shear strength envelope rotates about the suction axis towards the soil shear strength envelope at failure as soil structure is compressed. This framework is unique since the mobilized shear strength envelope acts as the yield locus since the axial strain along the locus is constant irrespective of the net stress or effective stress experienced by the soil. Soil compression occurs when the state of stress exceeds the mobilized shear strength; henceforth the stress state exceeds the yield surface.

2.2.1 Simulation of Loading Collapse. Md. Noor [17] anticipated that soil deformation is due to the combined action of shear and normal stress on the inclined planes. This was demonstrated based on the loose state of the soil, whereby the particles have room to adjust due to the nature of the voids, in the direction of shear as they are being displaced vertically downwards. However, when the particles experienced tightly packed arrangement, they have less freedom to move since it is in the interlocking state. Therefore, soil deformation is assumed to be one-dimensional. Hence, the action of shear and normal stress becomes the driving factor governing soil compression and it can be represented graphically by Mohr stress diagram. The mobilized shear strength is resisting the deformation and it can be represented graphically by the mobilized shear strength envelope. The deformation ceases when the mobilized shear strength equals or exceeds the stress state in the soil mass. Figure 7 demonstrates the loading collapse for unsaturated soil at a given suction.

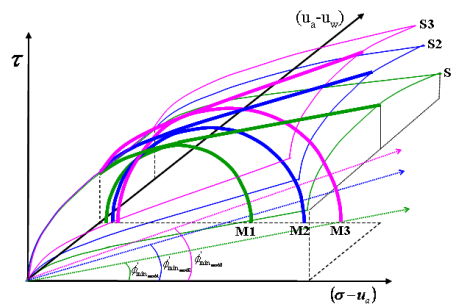


Figure 7. Modelling loading collapse using Rotational Multiple Yield Surface Framework of Md. Noor [17].

2.2.2 Simulation of Wetting Collapse. In the case of inundation for coarse-grained soils, this is equivalent to the constant diameter Mohr circle for the applied net stress moving parallel to the suction axis towards a lower suction. The turning of the shear surface envelope about the suction axis is immediate whenever the Mohr circle extends above the envelope. As the Mohr circle moves parallel to the suction axis towards the net stress axis in the case of wetting collapse, the yield surface envelope rotates and when suction is zero there is an abrupt rotation of the yield envelope since the envelope bends steeply downward illustrated in Figure 8. If the turning angle of the yield envelope represents the volume change this would then correspond to the massive volume change near saturation [17].

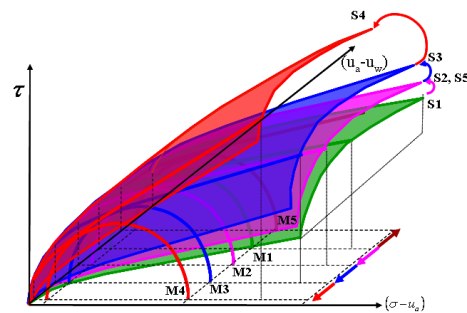


Figure 8. Rotated yield envelopes about the suction axis corresponding to the curvilinear lines in modelling wetting and drying collapse idealized by Md. Noor [17].

3. Results from Double Wall Triaxial and Modified Large Rowe's Cell Tests

The tests were conducted on granite residual soil obtained from Kuala Kubu Baharu to validate the use of CSESSM for the shear strength parameters and RMYSF for the validity of loading and wetting collapses of partially saturated soil. The physical properties were specified in detail by Mohamed Jais [27], Md. Noor and Mohamed Jais [26] and Md. Noor *et al.* [13]. The physical properties of Kuala Kubu Baharu (KKB) granite residual soil are given in Table 1. In fact, Mohamed Jais and Md. Noor [28] established an interesting information from the effective stress and shear strength interaction of saturated clay from Tanjung Bin, Johor and partially saturated granite taken from Rawang, Selangor. However, this paper will focus only for KKB granite residual soil.

Table 1. Physical characteristics of Kuala Kubu Baharu granite residual soil.

Soil physical properties	Description
Gravel content	19.7 %
Sand content	51.3 %
Silt content	16 %
Clay content	13 %
Average specific gravity	2.66
Average in situ moisture content	19.34 %
Average maximum dry density, ρ_{dry}	1.93 Mg/m ³
Average optimum moisture content, OMC	13.85 %

3.1 Validation of CSESSM from KKB Granite Residual Soil

The evaluation of saturated shear strength for Kuala Kubu Baharu granite residual soil has shown that at low stress level i.e. 30 and 50 kPa, the shear strength envelope approaches zero as shown in Figure 9. Hence, if the linear shear strength envelope is plotted, there will be an overestimation of the soil shear strength at low stress level.

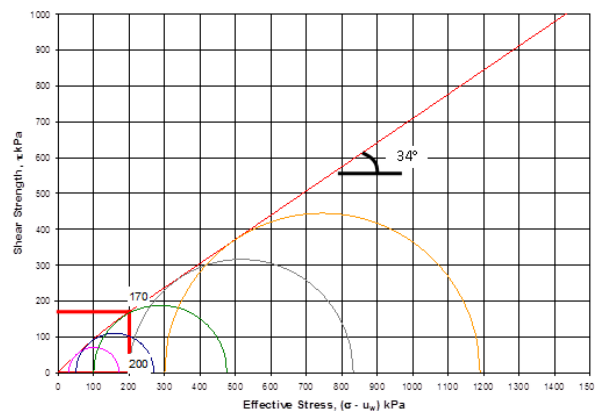


Figure 9. CSESSM for KKB granite residual soil.

The CSESSM was plotted for residual soil having mixture of fines at 29 % and coarse particles at 71 %. In fact, the minimum internal angle of friction at failure, $\phi'_{\min_f}$ and the transition effective stress, $(\sigma - u_w)_t$ are at 34° and 200 kPa, respectively. The transition shear stress, τ_t value is 170 kPa. The fine particles are usually flaky in nature, thus absorbing water and influence the plasticity of the soil. Coarse particles tend to be gritty and angular, which, increases the angle of friction in between the particles. Table 2 presents the CSESSM parameters obtained from three (3) different soil types.

The shear strength envelope with respect to suction is nonlinear throughout, whereas it increases towards residual suction and then gradually decreases towards zero as suction increases for unsaturated soils. From the model generated by Mohamed Jais [27], the unsaturated shear strength of KKB granite residual soil is in good agreements with CSESSM. These results from shear strengths tested at suctions of 40, 80, 100, 110 and 150 kPa proved that the CSESSM is applicable and in good agreements. The residual suction of 110 kPa is obtained from SWCC; therefore the unsaturated triaxial test was conducted at the said suction and yielded the maximum apparent cohesion, $c_s^{\max}$ of 72 kPa. When the unsaturated granite residual soil was tested at suction of 150 kPa, the apparent cohesion, c_s reduced to 70 kPa after passing residual suction clearly shown in Figure 10.

Table 2. CSESSM parameters for three different soil types at saturated condition [28].

Material type	Shear strength parameters		
	$\phi'_{\min_f}$	$(\sigma - u_w)_t$	τ_t
Blue Mountain limestone gravel (Md. Noor, 2006)	34	200	230
Tanjung Bin clay	10	165	110
Kuala Kubu Baharu granite residual soil	34	200	170

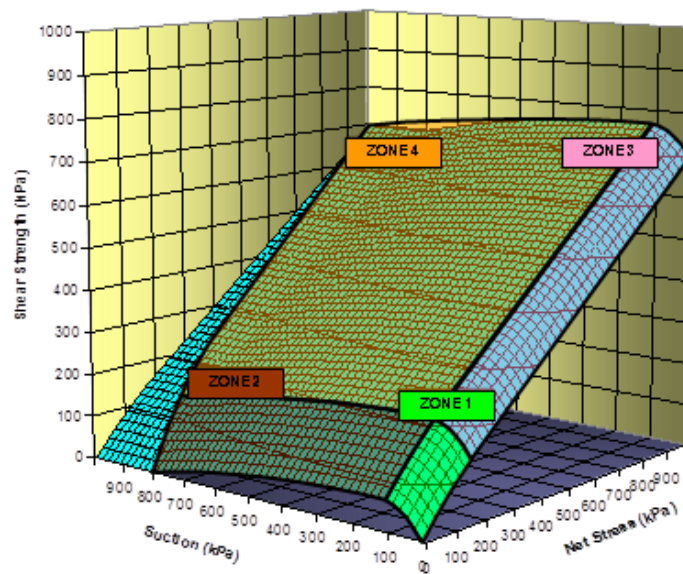


Figure 10. CSESSM obtained for Kuala Kubu Baharu granite residual soil.

From CSESSM parameters for KKB granite residual soil plotted previously in Figure 10, it showed increase of apparent cohesion until it approaches maximum apparent cohesion, $c_s^{\max}$ at residual suction of 110 kPa. Then the apparent cohesion starts to decrease gradually after reaching maximum apparent cohesion as shown clearly at suction 150 kPa illustrated in Figure 11. The plotted apparent cohesion values are similar to the findings of Fredlund *et al.* [29] and Infante Sedano and Vanapalli [24] where the shear strength against matric suction is nonlinear. However, the findings of Md. Noor [17] clearly showed that there is a maximum apparent cohesion at residual suction and then the apparent cohesion, $(u_a - u_w)_r$ gradually decrease as suction is increased.

3.2 Validation of Collapse Behaviour using RMYSF from KKB Granite Residual Soil

The RMYSF was established to explain the collapsibility of the soil due to loading and inundation, which was described in detail [27] for settlement due to both loading and wetting. The wetting collapse behaviour of soil due to inundation has been explained by Matyas and Radhakrishna [7], Fredlund and Morgenstern [30], Pereira and Fredlund [14] and Md. Noor and Anderson [16]. However, the mechanics are still complex and the settlement behaviour due to effective stress decrease has never been mentioned until it was found by Alonso *et al.* [12], Blanchfield and Anderson [31], Md. Noor and Anderson [16] and formulated by Mohamed Jais [27].

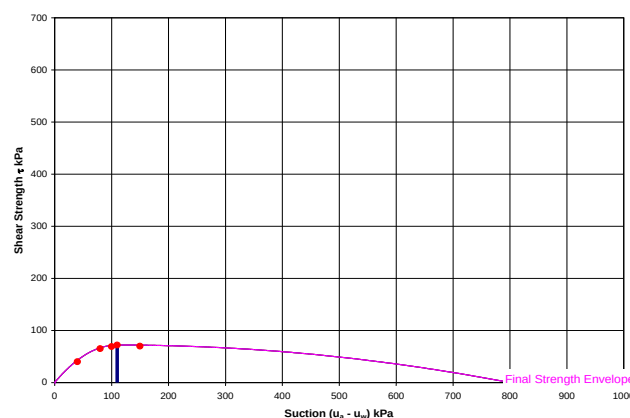


Figure 11. Values of apparent cohesion at different suction for KKB granite residual soil.

The volume change behaviour of KKB granite residual soil was extensively studied using the modified large Rowe's cell apparatus shown in Figure 12 and Figure 13 [27]. The test was conducted at suction of 100 kPa (water content at 13.85 %) and maximum dry density of 1.93 Mg/m³. The first series was the applied loading to determine the compressibility of the soil as controlled specimen for unsaturated KKB granite residual soil. Then two series of wetting at constant net stress of 150 kPa i.e. shallow foundation allowed bearing capacity and 300 kPa i.e. twice the allowable bearing pressure for shallow foundation. Mohamed Jais [27] found that at low net stress, inundation settlement occurred massively due to particle arrangement in the microstructure in order to achieve equilibrium and at high stress level; the inundation settlement is less as explained by Md. Noor and Anderson [25] which contradicts with the three dimensional model generated by Pereira and Fredlund [14]. The plot of void ratio with respect to net mean stress and suction showing these differences is shown in Figure 14.

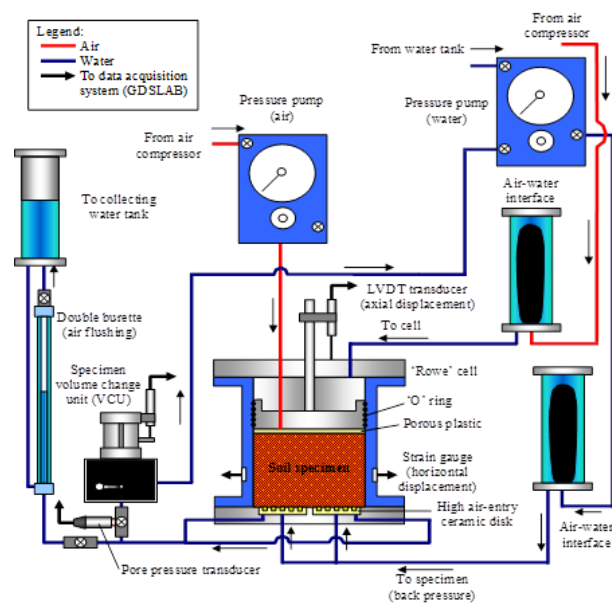


Figure 12. Schematic diagram of the modified large Rowe's cell.

3.3 Validation of Loading Collapse Settlement

The compression test using modified Rowe's cell apparatus is conducted to determine the loading collapse behaviour of KKB granite residual soil. The load was applied until there is no significant vertical displacement before the consequence stages were applied. A settlement comparison was made between the simulated settlement values using RMYSF and the laboratory modelling illustrated in Figure 15.

The simulation of loading collapse was done using RMYSF and the plot of mobilised shear strength envelopes based on the increasing Mohr circles are shown in Figure 16. This shows that settlements due to loading can be predicted based on the proposed framework and the proposed settlement equation as described by Mohamed Jais [27]. These validated that RMYSF could simulate loading collapse behaviour of unsaturated KKB granite residual soil and these simulations are based on the dimensions of the modified Rowe cell with the initial vertical stress of 150 kPa.



Figure 13. Modified large Rowe's cell for loading and wetting collapse modelling.

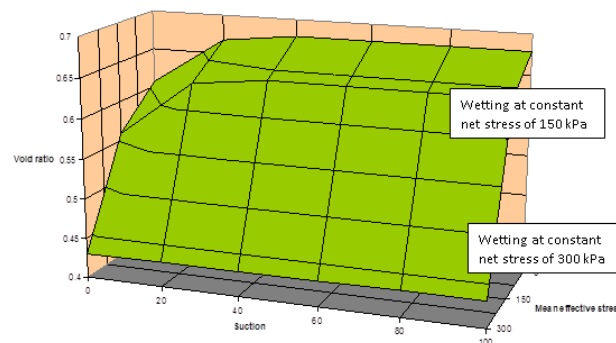


Figure 14. Wetting collapse of KKB granite residual soil at constant net stress of 150 kPa and 300 kPa generated 3D void ratio state surface.

3.4 Validation of Wetting Collapse Settlement

Md Noor [17] stated that inundation settlement occurred due to effective stress decrease where the soil experience loss of suction in the microstructure, hence reduction of stress occurred within the soil body, reducing the stiffness of the soil. This behaviour occurred mostly due to local wetting or inundation such as burst underground water pipe, settlement of embankment due to continuous wetting and inundation from adjacent rivers; and localised settlement underneath building structures. Therefore, this behaviour is investigated by laboratory modelling using the modified large Rowe's cell apparatus executed at different variations of constant applied stress i.e. 150 kPa and 300 kPa to validate the RMYSF.

The first stage of wetting at constant net stress of 150 kPa showed small settlement value during loading. During wetting at suction changes from 100 kPa to 60 kPa, small settlement values were recorded since the entrapped air dissolved within the water molecules and the microstructure rearranged to become more stable. However, when suction dropped to 20 kPa and finally at full saturation, massive collapse was recorded. This showed that effective stress decrease occurred causing the rotation of the mobilised shear strength envelope though there is no increase of applied stress shown in Figure 17. The visualization of rotating mobilised shear strength envelope due to loss of suction is shown in Figure 18 and Figure 19.

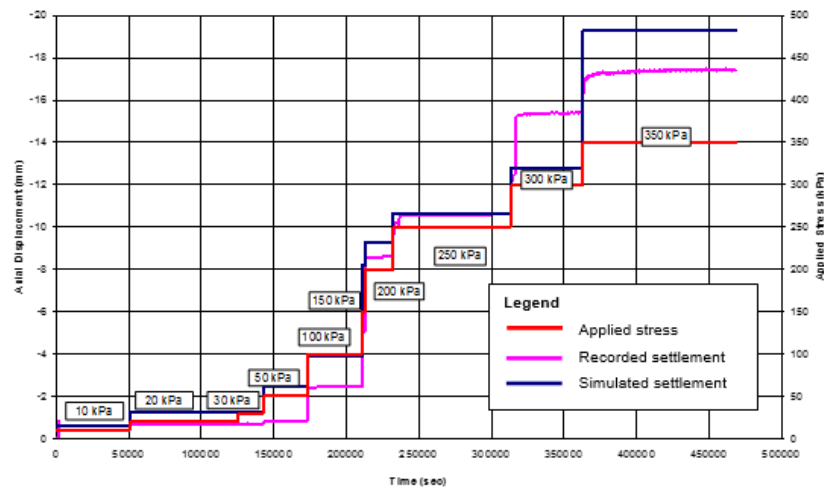


Figure 15. Simulated settlement using RMYSF and laboratory settlement using the modified Rowe's cell apparatus.

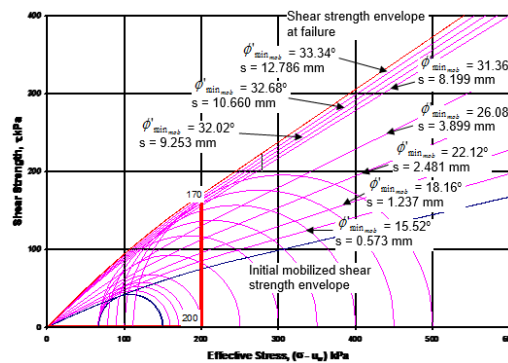


Figure 16. Simulation of loading collapse behaviour for KKB granite residual soil using RMYSF.

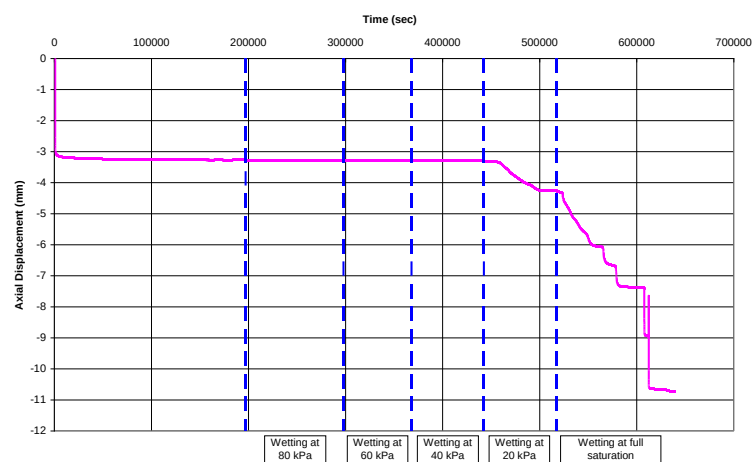


Figure 17. Wetting collapse settlement at constant applied stress of 150 kPa.

The second stage at constant net stress of 300 kPa showed massive loading settlement. However, when the soil was inundated up to full saturation, small amounts of settlement were recorded which

signified that the soil has experienced packing of the microstructure during compression. Md. Noor and Anderson [25] explained that minimal settlement occurred during wetting for higher applied stress since the soil has experience particle rearrangement and compression; hence the void ratio of the soil has become less as shown in Figure 20. There is wetting collapse settlement at high effective stress although, there was massive settlement observed after the soil has achieved full saturation. However, the value was rather small and this supported and agreed with the explanation by Md. Noor and Anderson [25]. The visualization of rotating mobilised shear strength envelope due to loss of suction is shown in Figure 21 and Figure 22.

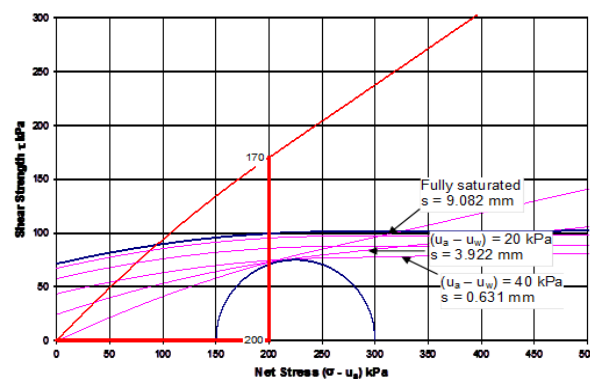


Figure 18. Simulation of wetting collapse settlement after the soil is loaded at constant applied stress of 150 kPa.

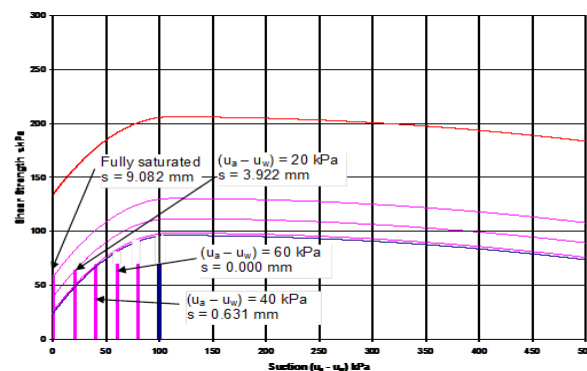


Figure 19. Suction decreases to saturation resulting in wetting collapse settlement at 150 kPa.

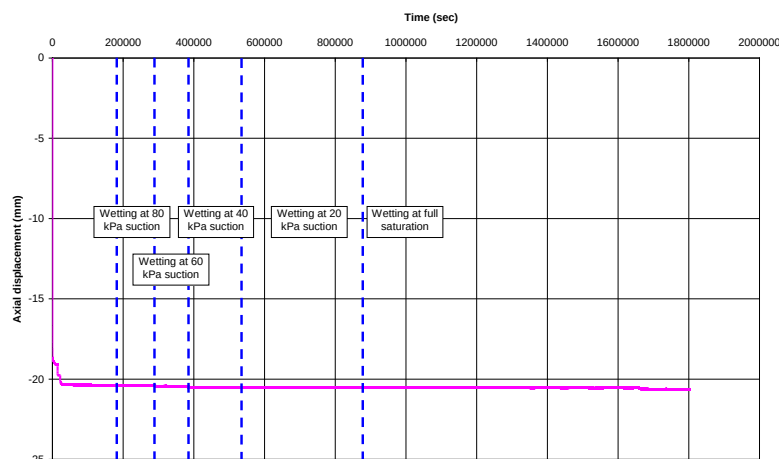
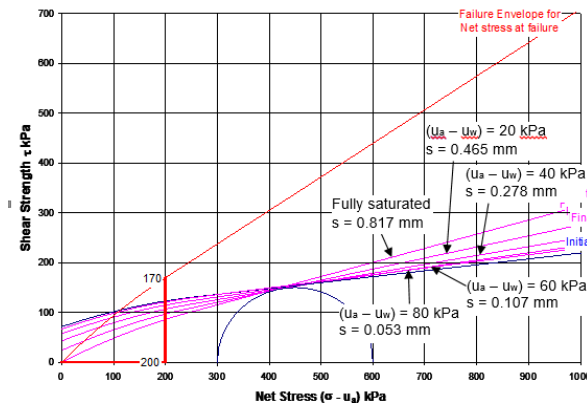
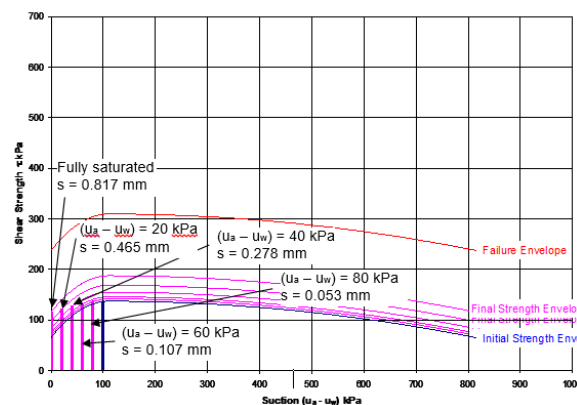


Figure 20. Wetting collapse settlement at constant applied stress of 300 kPa.**Figure 21.** Simulation of wetting collapse settlement after the soil is loaded at constant applied stress of 300 kPa.**Figure 22.** Suction decreases to saturation resulting in wetting collapse settlement at 300 kPa.

4. Conclusions

Based on the simulation for loading and wetting collapses, the RMYSF can simulate the actual soil behaviour based from the established CSESSM for KKB granite residual soil. The conclusions are:

- The formulation established from the RMYSF from CSESSM can simulate the complex wetting collapse settlement of unsaturated soil.
- From these formulations, the wetting collapse settlement due to effective stress decrease can be explained and simulated without any uncertainty and it can be explained from the stress paths observed from the mobilised shear strength envelope with respect to effective stress ($\sigma - u_w$) or net stress ($\sigma - u_a$) and suction ($u_a - u_w$).
- Collapse settlement physical modelling using modified large Rowe's cell apparatus verified the simulation using RMYSF and has proven the wetting collapse behaviour of the soil due to effective stress decrease exist.

5. References

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