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The Stress Strain Behaviour Envelope for Granitic Residual Soil in Mobilised Shear Strength Perceptive

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ABSTRACT

The concept of effective stress has been the principal concept in characterizing soil volume change behavior in soil mechanics, the settlement models developed using this concept have been empirical in nature. However, there remain certain unexplained soil volume change behaviors that cannot be explained using the effective stress concept, one such behaviour is the inundation settlement. Studies have begun to indicate the inevitable role of shear strength as a critical element to be incorporated in models to unravel the unexplained soil behaviours. One soil volume change model that applies the concept of effective stress and the shear strength interaction is the Rotational Multiple Yield Surface Framework (RMYSF) model. This model has been developed from the soil-strain behavior under anisotropic stress condition. Hence, the RMYSF actually measure the soil actual elasto-plastic response to stress rather than assuming it to be fully elastic or plastic as normally perceived by the industry. The frameworks measure the increase in the mobilize shear strength when the soil undergo anisotropic settlement.

1. Introduction

In order to determine the behaviour of soil volume change, it is important to understand the effective stress concept. It has been the principal concept in predicting soil settlements of the methods adopting this concept has been in nature. However, there are still certain soil volume change behaviors that cannot be explain by this concept, such as the inundation settlement. The inundation settlement appears to become bigger with a small effective stress and becomes smaller with a larger effective stress. Studies have begun to indicate the inevitable inclusion of shear strength as a parameter in models to address these unexplained soil volume change behaviours.

Lee [15], Chan and Chin [11], Ting and Ooi [31], Balasubramaniam *et al*., [6], Todo and Pauzi [32], Ramli [25], Suhaimi and Abdul [28], Kepli [14] and Anuar and Ali [5] studied the shear strength of granite residual soil in Malaysia and found that the soil shear strength is a major properties in

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geotechnical engineering. Geotechnical works involving shear strength includes slope stability, shallow or deep foundations, excavation, filling, earth dams, design of road bases and lateral resistance in retaining wall. Structures built must be stable and resistant to maximum loads applied to it, therefore the studies of shear strength and shear strength parameters need to be obtained for applying suitable design parameters.

Shear strength is based on two distinct parameters, the frictional resistance of the soil particles known as the angle of friction, ϕ and the cohesion of the soil, c. These parameters are affected by moisture content, pore pressure, structural influence, ground elevation, stress history, time, chemical reaction and the environment [7]. Chang and Broms [10] stated that the shear strength of residual soil is measured in the undrained shear strength condition due to its low permeability. Lumb [16], on the other hand, stated that the internal angle of friction, ϕ' is influenced by the percentage of clay content and Lumb [17] also stated that cohesion is influenced by the percentage of clay and the degree of saturation in the consolidated undrained triaxial test.

Shear strength in terms of total stress is related to saturated residual soil whereas shear strength in terms of effective stress is related to partially saturated residual soil. Since most residual soils in Malaysia are partially saturated, measurement in terms of total stress becomes inappropriate. In order to obtain the effective shear strength parameters, c' and ϕ', the specimen is required to be at its saturation stage. Based on Fookes [13], high cell pressures are required to saturate the soil specimen which increases the moisture content and degree of saturation, hence reduces the c' value due to loss of suction. However, Bressani and Vaughan [8], showed that ϕ' is not influenced by saturation of the soil and Brand [7] stated that the effective cohesion, c' measure is very small.

Research conducted in Malaysia by Todo *et al.,* [33] showed that the shear strength of residual soil in Malaysia range between 20 – 30 kPa. Rahardjo *et al.*, [23] found that the shear strength of residual soils in Singapore almost reached the value of 400 kPa. Table 1 shows shear strength parameters of residual soil from previous researchers using triaxial tests. Based on the triaxial test, the internal angle of friction, ϕ' range from 21° – 43°. In the unconsolidated undrained triaxial test, the undrained cohesion, c_u and undrained internal angle of friction ϕ_u range from 10 – 180 kPa and 1° – 11°, respectively, while in the consolidated undrained triaxial test, the effective cohesion, c' and effective angle of friction, ϕ' range between 0 -17 kPa and 25° – 41°, respectively.

However, Sinclair [27] conducted studies in Sri Lanka found that the effective cohesion, c' range between 80 – 275 kPa. Further studies on consolidated undrained and consolidated drained triaxial tests were conducted by Anuar and Ali [5] who found that the cohesion value is smaller and the friction angle is bigger in the drained test in comparison to the undrained test. According to Anuar and Ali [5], the high values of cohesion in the undrained triaxial test are due to the presence of high pore water pressure in the soil sample.

The stress-strain behaviour of soil is derived from the interaction between effective stress and shear strength. Figure 1 shows the interaction between effective stresses and shear strength envelope where the mobilised shear strength envelope rotates towards the shear strength envelope at failure as the Mohr circle grows during soil compression.

The inclination of the linear section for the shear strength envelope at failure represents the minimum friction angle at failure, $\phi'_{\min_f}$. The inclination of the linear section for the mobilised shear strength envelope is represented by the minimum mobilised friction angle, $\phi'_{\min_{mob}}$. The increase in $\phi'_{\min_{mob}}$, which is the change in position of the mobilised shear strength envelope represents a specific degree of compression or axial strain.

Table 1
Shear strength parameters of granitic residual soil

Source	Location	UU test		CU test		CD test	
		c_u (kPa)	$Ø_u$ (°)	c (kPa)	Ø (°)	c' (kPa)	Ø' (°)
Kepli [11]	Melaka	10 - 33	< 10	9.5 - 28	8 – 17.5		
Todo *et al.,* [33]	Kuala Lumpur	25 - 180					
Ramli [20]	Sungai Buloh, Jln Duta, Bukit Lanjan, Tapah and Skudai			0 - 40	14 - 36		
Todo and Pauzi [24]	Malaysia and Singapore	27 - 87	< 11				
Balasubramaniam *et al.,* [5]	Malaysia	73 - 117	1 – 9.5				
Ting and Ooi [23]	Malaysia	61.8 - 117	1 – 9.5				
Lee [12]	Cameron Highland			1 - 5	25 - 35		
Rahardjo, *et al.,* [23]	Yishun, Singapore			6 - 50	28 – 33		
	Mandai SIngapore			0 - 14	27 – 31		
Winn *et al.,* [34]	Bukit Timah, Singapore	10 - 150	< 10		20 - 40		
Deshmukh and Amonkar [12]	Goa, India	75	35	75	15	80	16
Sreekantiah [29]	West India			6 - 51	16 - 30		
Balasubramaniam *et al.,* [6]	Thailand						
Sinclair [27]	Sri Lanka			80 - 275			

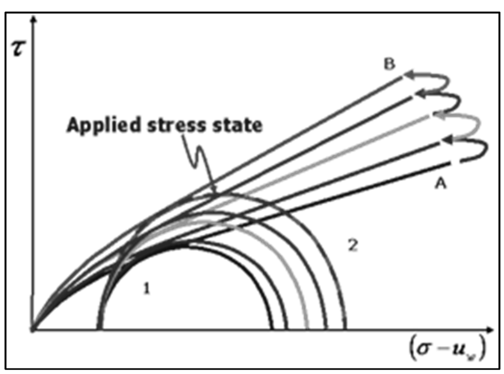

Fig. 1. Rotation of the mobilised shear strength envelope due to the enlargement of the effective stress Mohr-circle [18]

Md. Noor [16] reported that irrespective of the effective stress applied to a consolidated drained triaxial test, there is a unique relationship between $\phi'_{\min_{mob}}$ and ε_a. During soil compression, the mobilised shear strength envelope rotates towards shear strength envelope at failure. The location of the mobilised shear strength envelope represents a specific degree of compression, irrespective of the effective stresses. Therefore, if $\phi'_{\min_{mob}}$ is plotted against ε_a for tests at different effective stresses, the curves for all the tests to indicate the unique relationship will overlap. Figure 2 illustrates this unique relationship of saturated Blue Mountain Limestone. This unique relationship varies with different soil types.

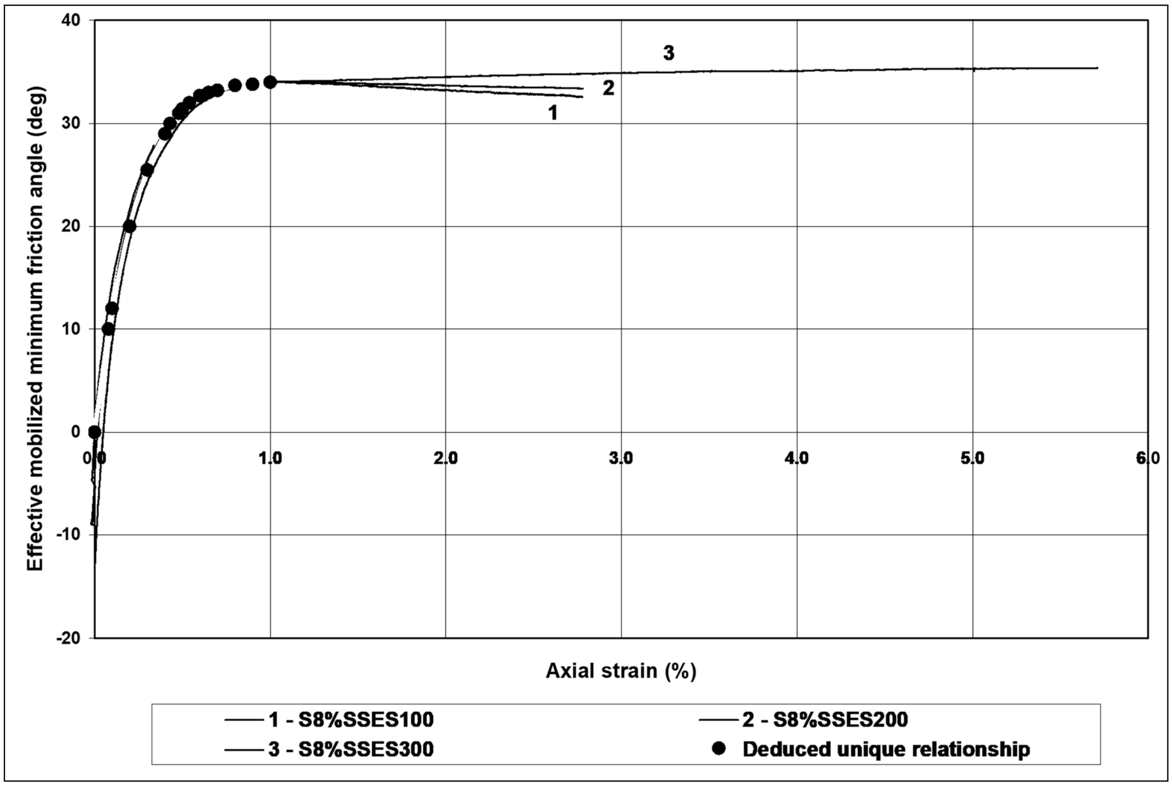

Fig. 2. Unique relationship of $\phi'_{\min_{mob}} - \varepsilon_a$ for triaxial compression tests on saturated specimens of Blue Mountain Limestone gravel [16]

Md. Noor [16] proposed a method to determine the minimum mobilised friction angle, $\phi'_{\min_{mob}}$ from the minimum friction angle at failure, $\phi'_{\min_f}$, mobilised friction angle at failure, ϕ'_{mob_f} and the mobilised friction angle, ϕ'_{mob}. The change in effective mobilised minimum friction angle, $\Delta\phi'_{\min_{mob}}$, is assumed to be equal to the change in the effective mobilised friction angle, $\Delta\phi'_{mob}$, as shown in Figure 3 where

$$\Delta\phi'_{mob} = \Delta\phi'_{\min_{mob}} \tag{1}$$

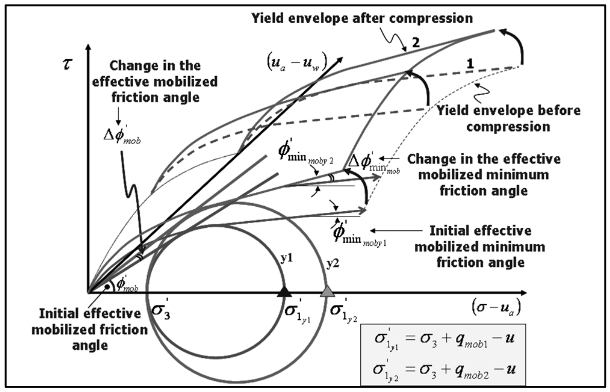

Fig. 3. The rotation of the yield surface envelope as the deviator stress increases from q_{mob1} to q_{mob2} and the assumption that $\Delta\phi'_{\min_{mob}} = \Delta\phi'_{mob}$ [16]

The expression for the change in effective mobilised minimum friction angle between the two yield surface envelopes is given by

$$\phi'_{\min_{moby2}} = \phi'_{\min_{moby1}} + \Delta\phi'_{\min_{mob}} \tag{2}$$

Similarly, if the variables are substituted, the expression is given by:

$$\phi'_{\min_{moby2}} = \phi'_{\min_{moby1}} + \Delta\phi'_{mob} \tag{3}$$

The effective mobilised minimum friction angle of the yield surface envelope, $\phi'_{\min_{mob}}$ with reference to the effective mobilised minimum friction angle at failure, $\phi'_{\min_f}$ (i.e. corresponding to the shear strength envelope) can be obtained as follows:

$$\phi'_{\min_{mob}} = \phi'_{\min_f} - \left(\phi'_{mob_f} - \phi'_{mob}\right) \tag{4}$$

Settlement of foundation is a geotechnical problem that is mainly caused by the increase of effective stress. Recently, numerous cases of foundation settlements occurred when the load was not increased but there was a sudden drop of effective stress due to the increase of groundwater table or inundation. This phenomenon is usually very rare and complex which cannot be explained by current practice of geotechnical engineering. In fact, most cases that occurred were either nearby a river or ex-mining area solely because the area is prone to groundwater fluctuation or the material is porous enough for water to flow upwards.

Modelling using finite element methods has been widely used in most engineering applications, but the significance of modelling using different models and parameters needs to be fully understood in order to refine the analysis such that it represents the actual condition. It is most important to understand the purpose of these soil models before adopting the correct values into the soil model parameters. In addition, certain conditions such as loss of suction, unsaturated and saturated soils are mostly not taken into account in the softwares available in the market. Therefore, this study hope to incorporating these conditions into these geotechnical engineering softwares.

Behaviour of saturated and unsaturated soils is not fully understood since there have been numerous soil models to explain the complex matter where loss of suction could either decrease or increase the stress behaviour of soils. Critical state soil mechanics proposed by Alonso *et al.,* [4] to predict the behaviour of unsaturated soils has been used worldwide and had led to the development of the modified cam clay model [21] to predict settlement behaviour of soft soil. In PLAXIS Finite Element (FE), Soft Soil Model (SS) and Soft Soil Creep Model (SSC) were implemented to predict the behaviour of soft clays and based on the modified critical state behaviour of soil that leads to extensive critical state parameters such as the modified compression index and modified swelling index that are used to predict the deformation. There are limitations to the models listed above with regards to stiffness where the behaviour is said to be in elastic-perfectly plastic manner but the derivation of the model is based on the spring stiffness with the elastic modulus or termed as Young's modulus, E, used in most of the soil models. Therefore, it is anticipated that the application of the curved surface shear strength envelope developed by Md. Noor and Anderson [17] would explain the behaviour of both saturated and unsaturated soil in terms of settlements and volume change.

2. Methodology

This research is an experimental study to determine the behavior and the strength of granitic residual soil. The procedures for laboratory tests are based on the British Standards (BS). The triaxial consolidated drained, CD test was used to determine the effective internal friction angle at failure, ϕ'_f, transition shear strength, τ_t and transition effective stress, $(\sigma\text{-}u_w)_t$ according to the curved-surface envelope shear strength model of Md. Noor and Anderson [17], and the linear envelope of Terzaghi [30].

The samples were taken at Kuala Kubu Baharu, Selangor with coordinates of 3°34'06.17"N; 101°41'51.50"E from a depth of 1.5m below the ground surface. Hence this sample is considered as a surface of original residual soil. The disturbed sample was taken and placed in polyethylene bags. To avoid the loss of moisture content, the bags containing the sample must be carefully tied and brought back to the soil laboratory as soon as possible. Laboratory tests were then conducted with the use of appropriate quantities of the residual soil sample collected. Important physical and engineering properties were determined using appropriate equipments in the laboratory. Figure 4 shows the location of samples at Kuala Kubu Baharu, Selangor.

Fig. 4. Location of samples collected at Kuala Kubu Baharu, Selangor

Remoulded soil were used to prepare specimens of 50mm diameter and 100mm in height. The moisture content and weight of the samples are kept constant for all samples and tested under different applied stresses of 50, 100, 200 and 300kPa. The specimens were then compacted using a rod of size 25mm in diameter and 350mm in height weighing 200g at three layer intervals. Twenty-five (25) number of blows approximately were applied at each layer.

The triaxial test will be used to determine the shear strength of the specimens. It is still one of the most versatile and widely performed geotechnical laboratory test, for determining the shear strength and stiffness of soil and rock. The triaxial test is used widely due to its simple procedure, its functions to control the specimen drainage and to take measurement of pore water pressure.

3. Experimental Results and Discussion

Table 1 shows the effective shear strength parameters incorporates mobilized shear strength envelope at failure. Result shows that, the friction angle of the specimens at failure is 33°.

Table 2
Effective shear stress parameters at failure of granitic residual soil

Effective Stress, kPa	Condition of Failure			Shear Strength Parameters		
	DS (kPa)	PWP (kPa)	CP (kPa)	ϕ'_f	τ_t kPa	$(\sigma-U_w)_t$ kPa
50	155	439	500	33°	183	124
100	278	443	550			
200	488	450	650			
300	716	451	750			

Stress-strain graphs were plotted using data from the triaxial tests to determine the effective internal friction angle at failure, ϕ'_f of the specimens. Figure 5 shows the recorded maximum deviator stress in stress-strain of the saturated specimens at failure. The maximum value recorded from the graph was 716 kPa. The failure envelope for saturated specimens can be drawn using the maximum values from the stress-strain envelope. Four failure envelopes could be easily be established and the failure line of the graph can be achieved as shown in Figure 6.

The minimum mobilised friction angles, ϕ'_{min} obtained for every deviator stress represents a particular amount of strain at any state of effective stress. This unique relationship was established from Md. Noor [15] which in turn relates to the change of strain or volume change of the soil tested. Figure 7, Figure 8, Figure 9 and Figure 10 shows the minimum mobilised shear strength envelopes obtain with the applied stresses at 50kPa, 100 kPa, 200 kPa and at 300 kPa respectively. With this plot, the deviator stresses represent strains in the stress train curves, therefore with the change in minimum mobilised friction angle, the amount of strain it represents can be plotted and the coefficient of anisotropic compression can be assumed.

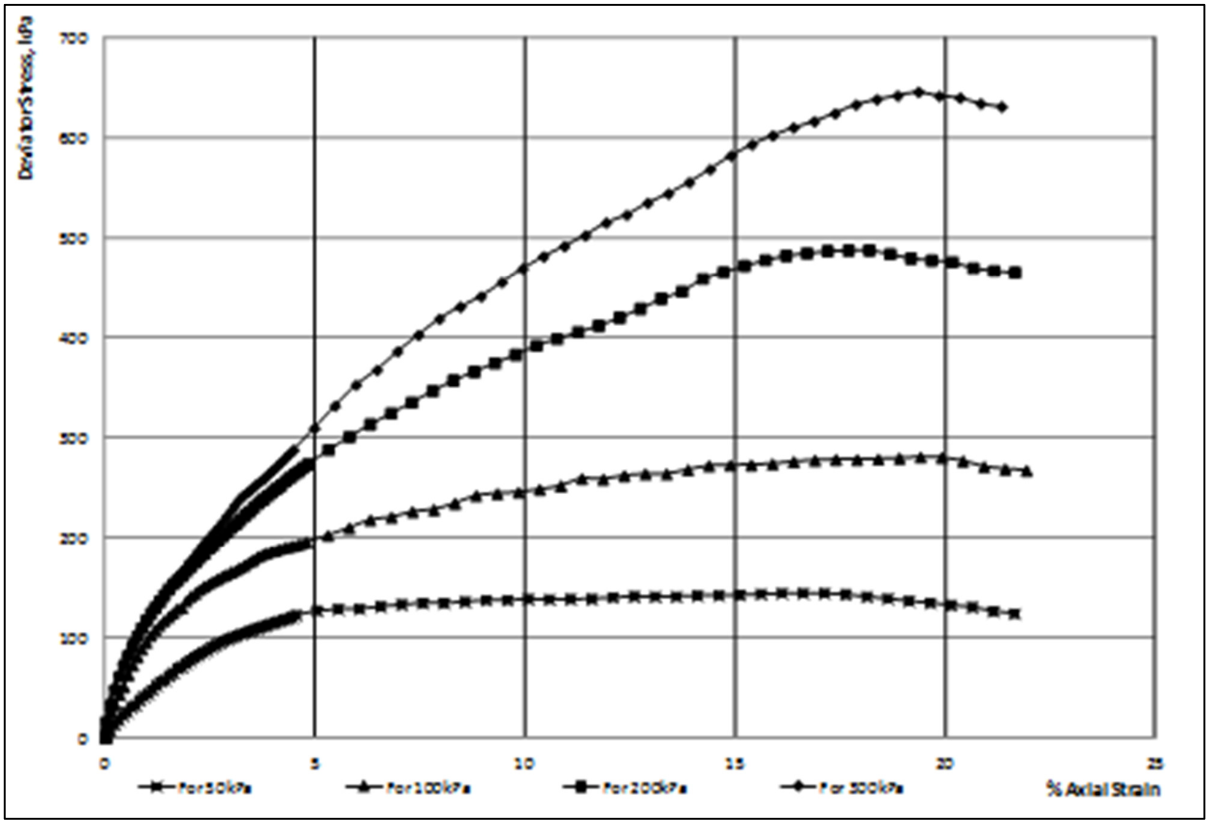

Fig. 5. Stress-strain for saturated specimens at 50, 100, 200 and 300kPa applied stress

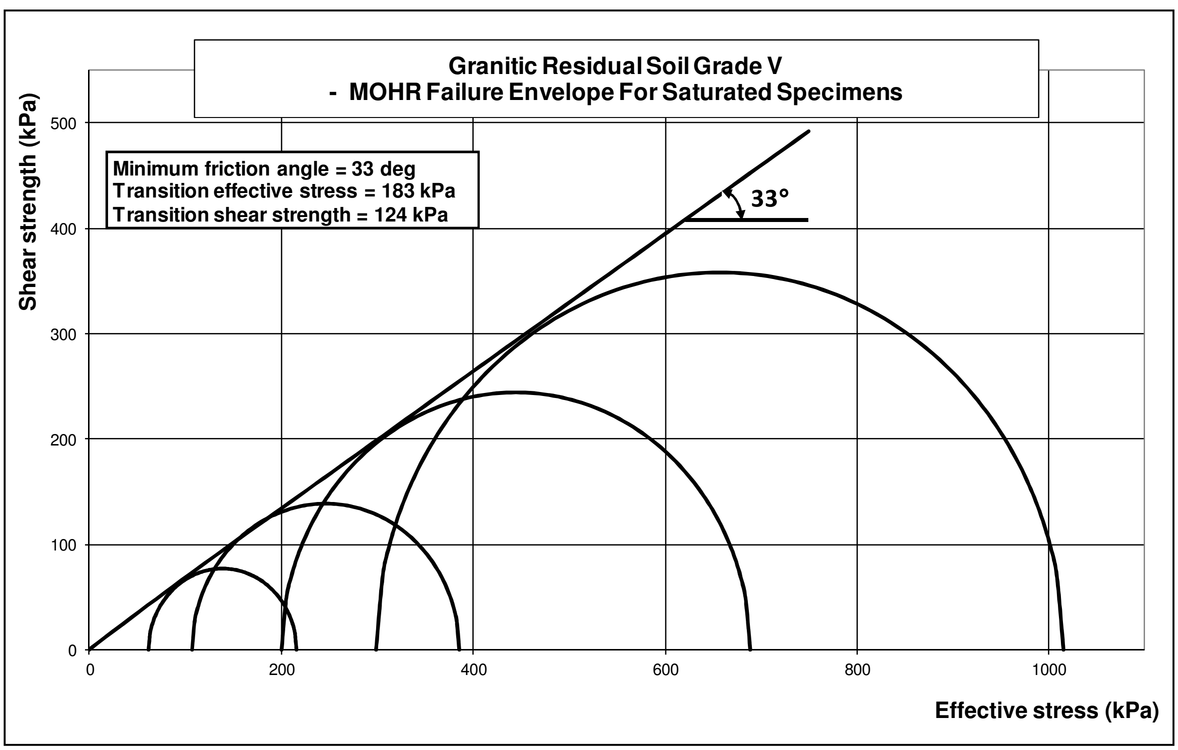

Fig. 6. Curvi-linear shear strength envelope for saturated specimens

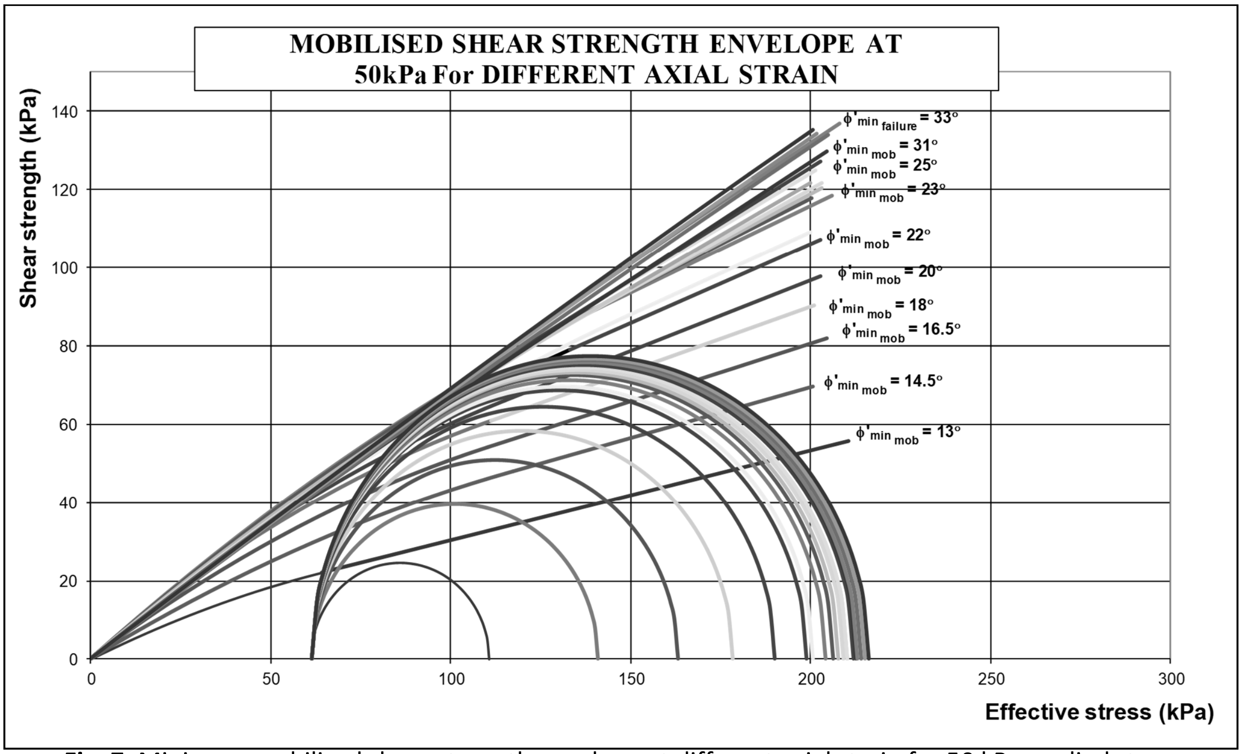

Fig. 7. Minimum mobilised shear strength envelope at different axial strain for 50 kPa applied stress

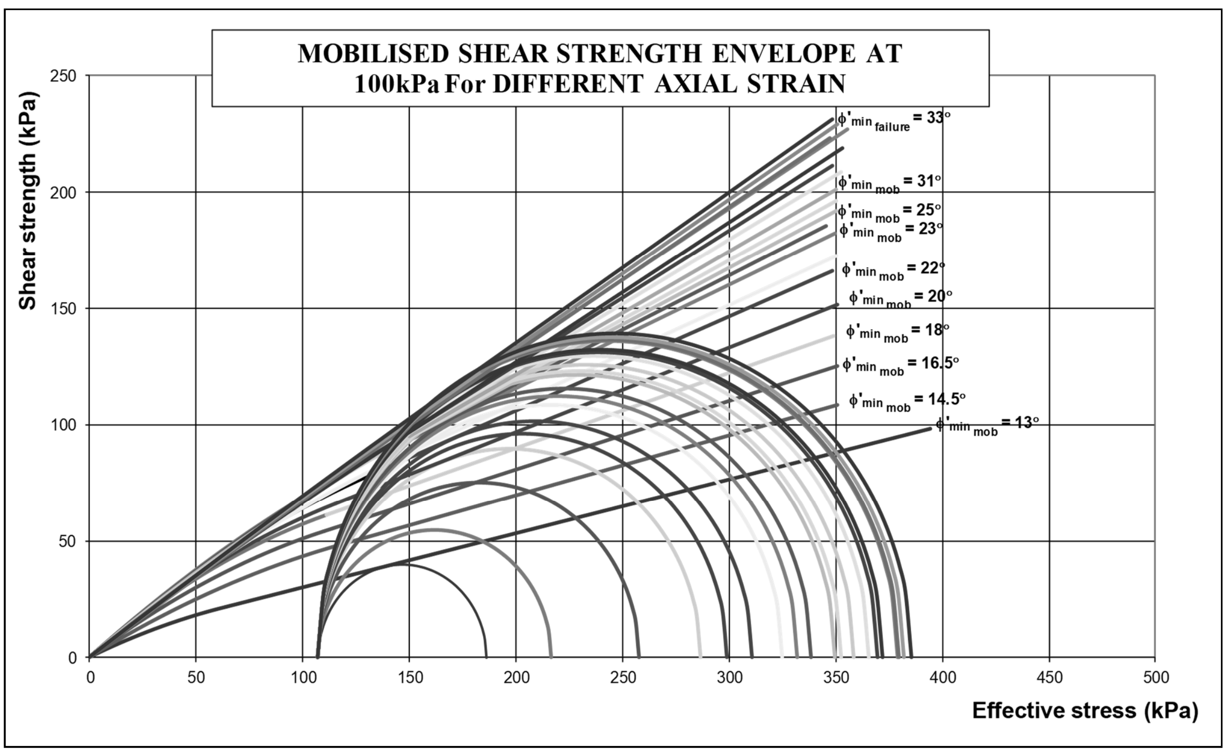

Fig. 8. Minimum mobilised shear strength envelope at different axial strain for 100 kPa applied stress

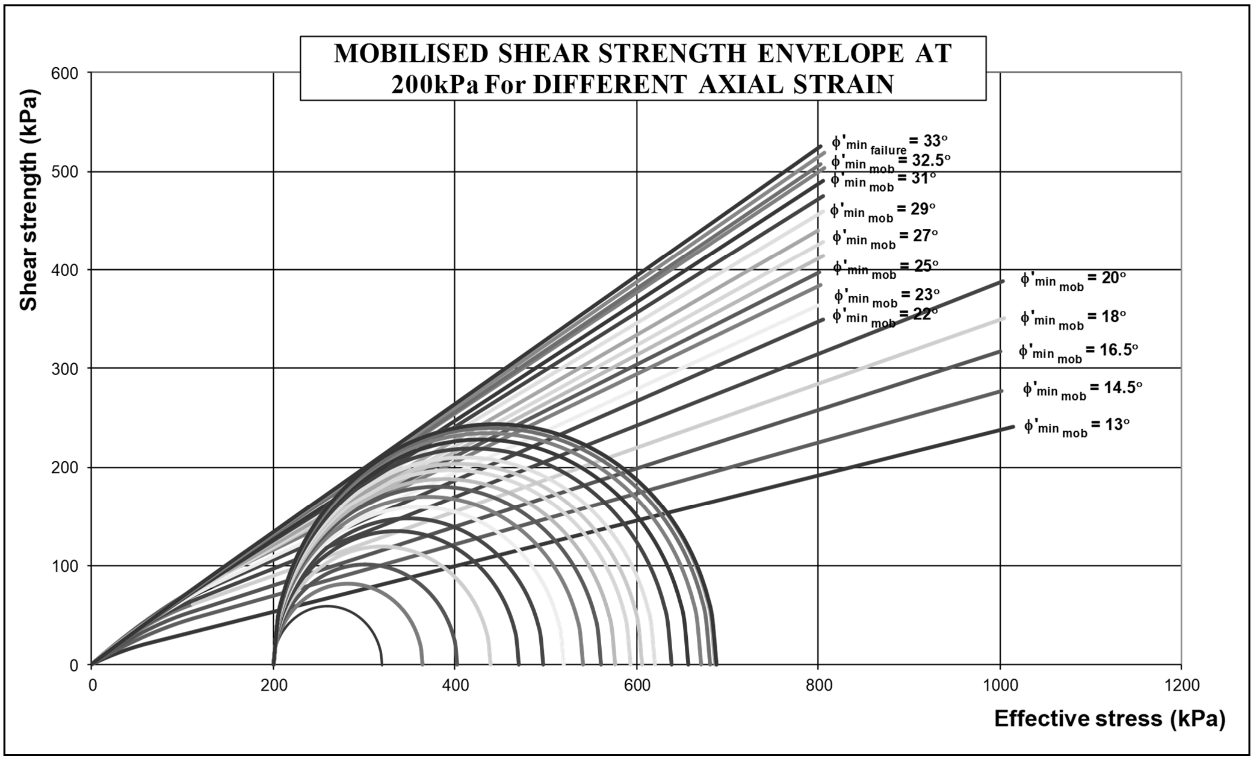

Fig. 9. Minimum mobilised shear strength envelope at different axial strain for 200 kPa applied stress

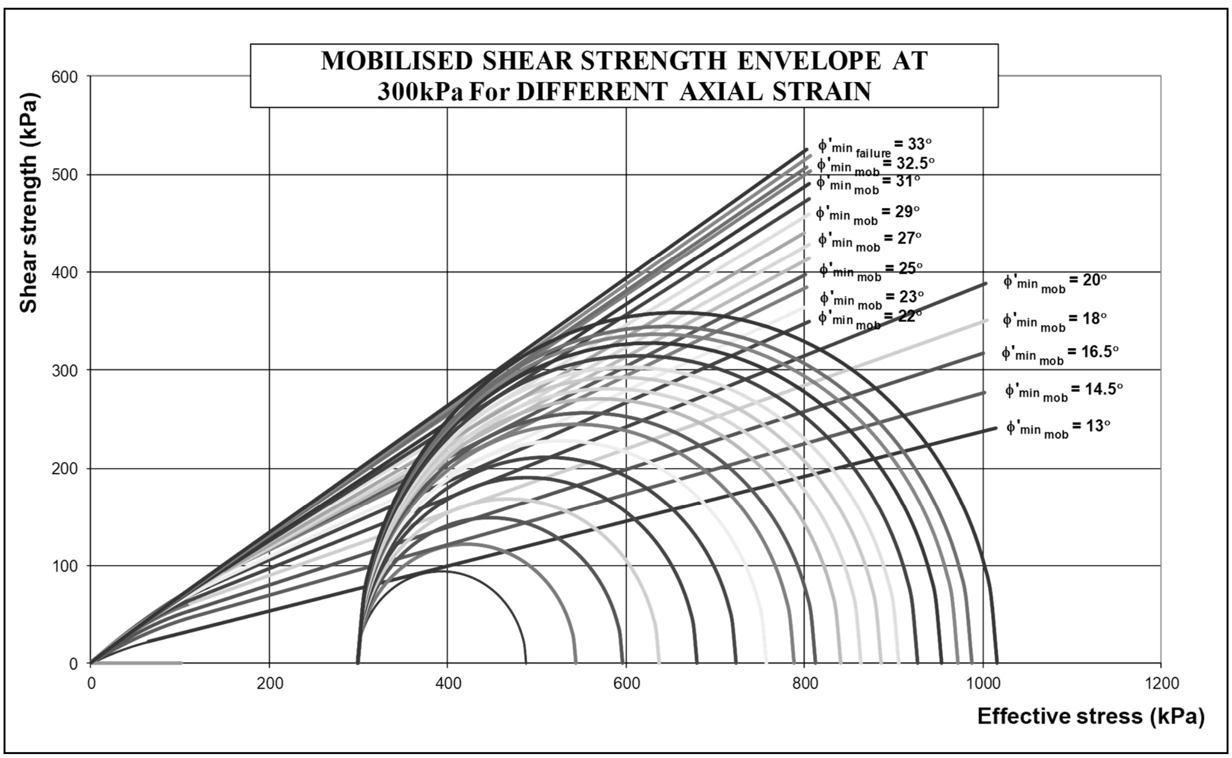

Fig. 10. Minimum mobilised shear strength envelope at different axial strain for 300 kPa applied stress

From the minimum mobilised shear strength envelope plots, the strain values relating to the changes in deviator stresses are plotted as a unique relationship between minimum mobilised friction angle and axial strain for the soil, as shown in Figure 11. Figure 12 on the other hand shows the minimum mobilised shear strength envelopes at different axial strain. Interestingly, the unique relationship can map the minimum mobilised friction angle in relation to the strain of the soil, and this reflects the initial volume change of the soil or compression once the soil has achieved full consolidation. The change of strain from an initial strain up to the applied load, changes the soil compression, allowing settlement of the ground to be determined.

Table 2 presents the minimum mobilised shear strength envelopes corresponding to the peak deviator stress and their relation to the strains for the soil specimens.

Table 2
Deviator stresses, minimum mobilised friction angles and corresponding axial strain values obtained from the mobilised shear strength envelopes

Axial Strain (%)	Deviator stress (kPa)				Minimum mobilised friction angle
	50	100	200	300	
1%	49.41	79.17	119.45	189.63	13.0
2%	79.49	109.44	164.01	244.86	14.5
3%	101.78	150.60	202.81	297.57	16.5
4%	116.88	179.61	238.41	338.15	18.0
5%	128.70	191.81	269.87	380.63	20.0
6%	137.54	203.40	296.53	423.73	22.0
7%	139.13	217.50	319.61	457.64	23.0
8%	142.79	224.56	340.70	490.06	24.0
9%	144.95	231.47	360.58	512.97	25.0
10%	146.23	242.65	376.90	541.10	26.0
11%	147.90	245.73	393.43	563.63	27.0
12%	148.68	251.37	406.20	586.87	28.0
13%	148.90	258.61	419.86	607.04	29.0
14%	150.48	262.72	437.94	628.54	30.0
15%	151.40	264.75	456.07	654.98	31.0
16%	151.66	271.81	470.21	654.98	31.5
17%	152.56	272.70	480.70	654.98	32.0
18%	153.66	275.08	486.16	654.98	32.5
19%	154.80	278.27	488.29	654.98	33.0

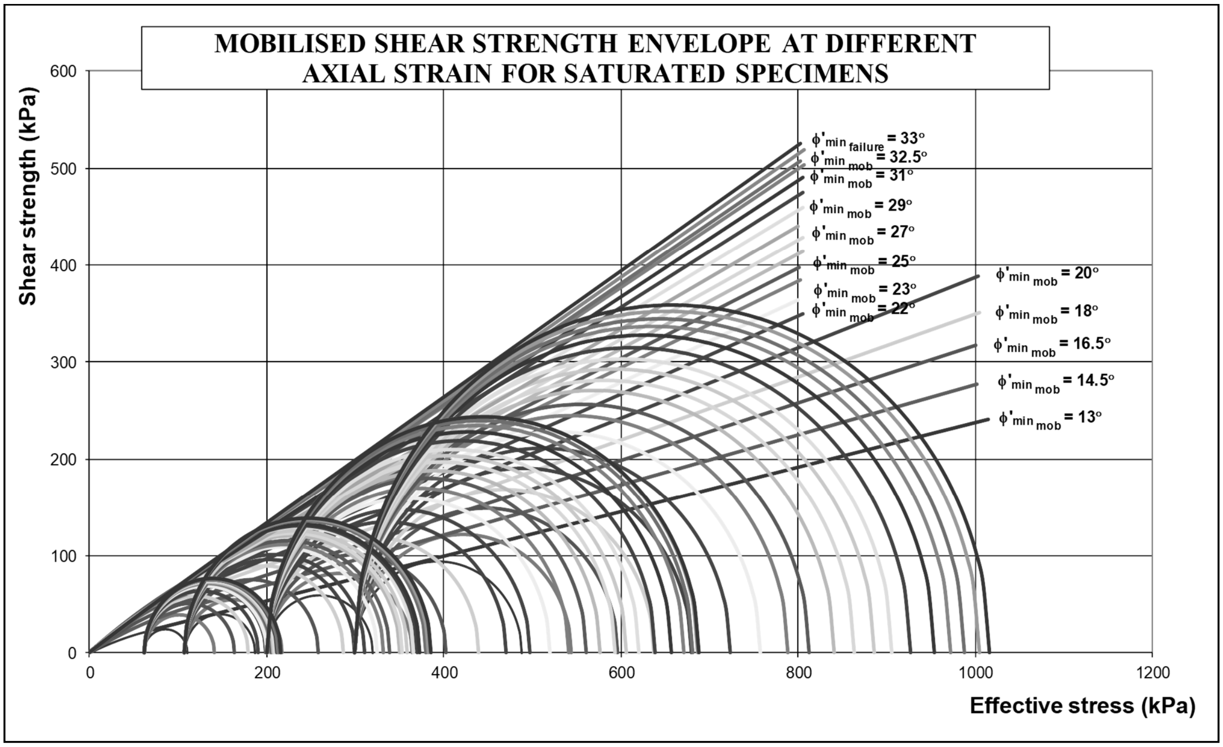

Fig. 11. Minimum mobilised shear strength envelope at different axial strain for increasing deviator stress

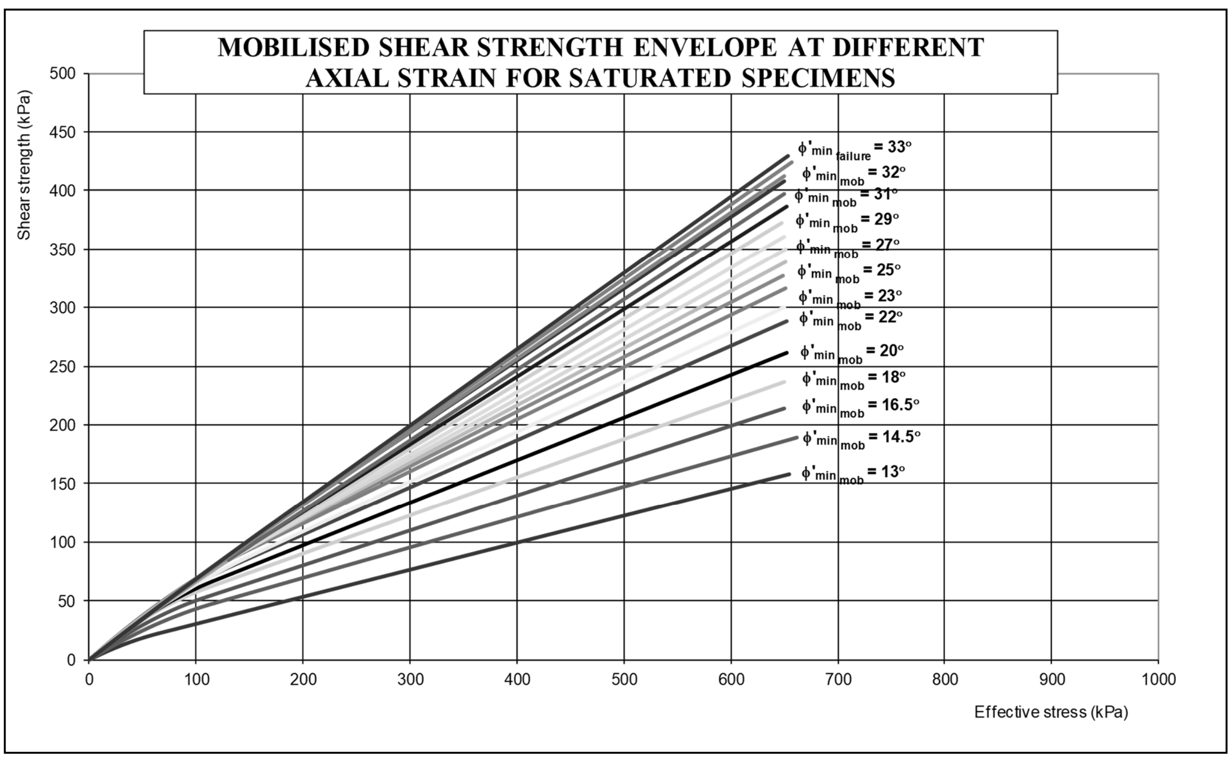

Fig. 12. Minimum mobilised shear strength envelope at different axial strain without Mohr Circle

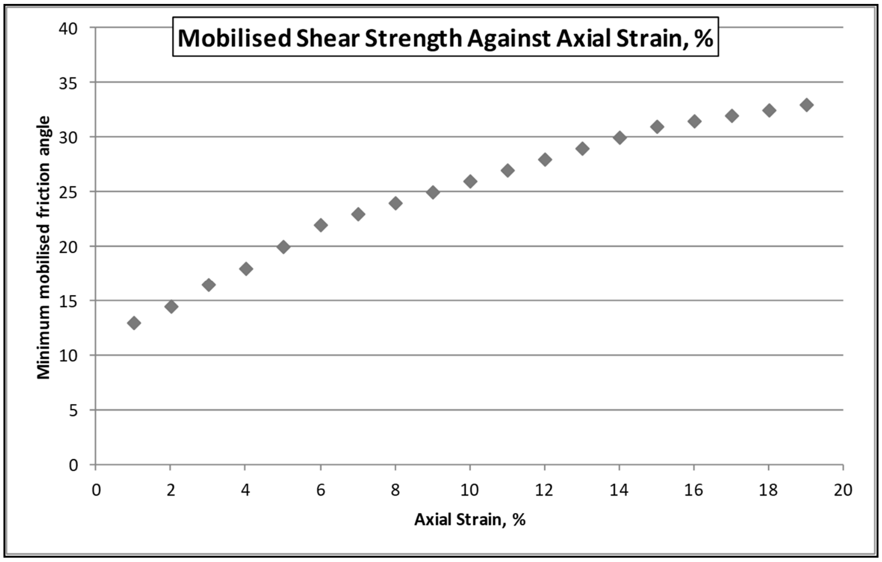

Fig. 13. Mobilised shear strength envelope against Axial Strain

4. Conclusion

By using the unique relationship of $\phi'_{\min_{mob}} - \varepsilon_a$ during analysis, it found that, there are simultaneous rotation axis during the compression of the specimens in shearing stage. This unique relationship in this research can be a milestone in order to incorporates the Rotational Multiple Yield Surface Framework to extend and reflect the true behaviours of stress-strain properties of the soil. Without the existence of this unique relationship the framework would not be valid. The most important advantage of having this unique relationship is that it can be used to predict the soil stress-strain behaviour at any effective or net stress. Furthermore, to establish the Rotational Multiple Yield Surface Framework, a lot of data need to be verified using this method and thus to proof the theoretical framework of these is successful and validate.

As a recommendation, it is important to extend the laboratory testing from saturated specimens to unsaturated specimens using double wall triaxial and compare the findings between these two soils condition by replicated the Rotational Multiple Yield Surface Framework in order to obtain more accurate results and interpretation of the shear strength of the soil.

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