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Critical State shear strength of an unsaturated artificially cemented sand

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ABSTRACT

This paper presents the results of a set of 22 triaxial tests on an unsaturated artificially cemented sand. The results are used to explore the applicability of a number of unsaturated soil frameworks for interpreting the shear strength. Constant water content triaxial tests were carried out on unsaturated specimens, using the axis translation technique to measure suctions during shearing. The test results on the unsaturated material were referenced against a series of drained and undrained triaxial tests that were carried out on saturated specimens. The results of the unsaturated tests were analysed to investigate the effect of the suction and degree of saturation on the shear strength at the Critical State. The results show that the Critical State stress ratio in terms of net stress (M_a) was found to be larger than the saturated critical state stress ratio (M_s). It was also found that the stress ratio in terms of suction (M_b) reduced as suction increased (when the degree of saturation reduced below 30%). Interestingly, during the initial desaturation phase when the degree of saturation reduces considerably, the stress ratio M_b was largely unaffected by desaturation. It was only when the suction increased sufficiently that the micro-voids within the cementing material could start to desaturate that a reduction in M_b was seen. This occurred at suctions in excess of the residual suction when the global degree of saturation was changing very little. This implies that the suction is contributing to the strength of the cementing material itself.

Keywords: Partial saturation; Suction; Fabric/structure of soils; Sands; Shear strength;

NOTATION

c'	cohesion (in terms of effective stress)
c''	cohesion when the two stress variables $(\sigma - u_a)$ and $(u_a - u_w)$ are zero
e	void ratio
k	parameter describing the increase in cohesion with suction (in the Barcelona Basic Model)
M	critical state stress ratio (in terms of effective stress)
M_a	critical state stress ratio with respect to net mean stress $(p - u_a)$
M_b	critical state stress ratio with respect to matric suction $(u_a - u_w)$
M_s	critical state ratio for saturated conditions
p	mean total stress $(\sigma_1 + \sigma_2 + \sigma_3)/3$
$p\text{-}u_a$	mean net stress
q	deviator stress $(\sigma_1 - \sigma_3)$
S_r	degree of saturation
u_a	pore air pressure
u_w	pore water pressure
$u_a - u_w$	matric suction
ϕ'	angle of shearing resistance (in terms of effective stress)
ϕ'_c	critical state angle of shearing resistance (in terms of effective stress)
ϕ^a	angle of shearing resistance with respect to net stress $(\sigma - u_a)$
ϕ^b	angle of shearing resistance with respect to matric suction $(u_a - u_w)$
$(\phi^a)_c$	critical state angle of shearing resistance with respect to $(\sigma - u_a)$
$(\phi^b)_c$	critical state angle of shearing resistance with respect to $(u_a - u_w)$
σ	total stress
σ'	effective stress
$\sigma - u_a$	net stress
τ_f	shear strength
χ	Bishop's factor related to the degree of saturation

1 INTRODUCTION

In many parts of the world, especially in tropical and arid areas, soils exist in an unsaturated state due to the climatic conditions. It is also common for tropical residual soils or sabkahs to have a cemented or bonded structure. Therefore, the behaviour of bonded soils in unsaturated conditions needs to be explained and clarified. Vaughan (1985) and Fredlund (1998) identified the need to develop a framework for describing and clarifying the engineering properties of unsaturated soils. The effect of bonded structure on soil behaviour in a saturated state is well established but there is still limited information on soil behaviour in an unsaturated state.

Many tropical and arid soils are "structured soils" (Leroueil and Vaughan, 1990). The term *structure* is used here to refer to the combination of *fabric*, meaning the geometrical arrangement of particles within a soil, and inter-particle *bonding* that results from cementation and physico-chemical interactions (Yong and Warkentin, 1975). This is not a universal definition and in many instances the terms structure, microstructure and fabric are used interchangeably to refer to the geometric particle arrangements (Toll and Ali Rahman, 2010). However, to distinguish between the two aspects of structure (i.e. fabric and bonding) is valuable.

Due to difficulties in conserving the weakly bonded structure and because of the variability in void ratio and bond strength in natural residual soils, many experimental studies have been performed on artificially bonded samples (Clough et al., 1981; Maccarini, 1987; Bressani, 1990; Coop and Atkinson, 1993; Malandraki, 1994; Cuccovillo and Coop, 1999; Asghari et al, 2003). As many theoretical models of soil behaviour have been developed largely from studying the behaviour of remoulded and reconstituted soils, it is appropriate to use artificially cemented soils to develop such frameworks for structured soils.

This study considers the Critical State framework for soil behaviour and looks at approaches proposed for extending this to unsaturated soils. Test results are presented for an artificial soil produced by mixing sand and kaolin and then firing the mixture so that the kaolin formed a permanent bond between the sand particles. Twenty two constant water content triaxial tests were carried out on unsaturated specimens, using the axis translation technique to measure suctions during shearing. These are referenced against a series of drained and undrained triaxial tests that were carried out on saturated specimens. The results are analysed to investigate the effect of the suction and degree of saturation on the shear strength at the Critical State for a cemented sand.

2 FRAMEWORKS FOR UNSATURATED SOILS

The first attempt to explain the shear behaviour of unsaturated soils was presented by Bishop (1959). Bishop adopted an effective stress approach which was expressed as:

$$\sigma' = \sigma - u_a + \chi\ (u_a - u_w) \qquad [1]$$

The χ variable was an empirical factor that varied between 0 and 1 as a function of degree of saturation, with χ=1 coinciding with full saturation. If χ=1 the equation reduces to the effective stress equation for saturated soils, so this provided a simple transition between saturated and unsaturated conditions.

Khalili and Khabbaz (1998) proposed that χ could be expressed as a function of suction (related to the air entry value of the soil) rather than degree of saturation. They suggested that expressing χ in this way allowed a unique value of χ to be defined.

Fredlund et al. (1978) argued that it was better to separate the effects of net stress and suction, rather than combine them into a single "effective stress" and gave the shear strength equation for unsaturated soils as:

$$\tau_f = c'' + (\sigma - u_a) \tan \phi^a + (u_a - u_w) \tan \phi^b \qquad [2]$$

Fredlund et al. (1978) went on to suggest that, when the matric suction is zero, the $(\sigma - u_a)$ plane will have the same angle of shearing resistance as the $(\sigma - u_w)$ plane. Therefore, they suggested that ϕ^a is the same as ϕ' (the angle of shearing resistance with respect to effective stress for saturated conditions). Fredlund and Rahardjo (1993: p. 238) suggest that the angle of shearing resistance ϕ^a "appears to be essentially equal to the effective angle of internal friction obtained from shear strength tests on saturated soil specimens". Fredlund et al. (1978) also suggested that c'' is the same as c' (the effective cohesion).

Making these two assumptions, equation [2] becomes the following:

$$\tau_f = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \tan \phi^b \qquad [3]$$

and it is this equation that is commonly quoted (e.g. Fredlund and Rahardjo, 1993).

However, Toll (2000) has argued that ϕ^a is not necessarily the same as ϕ' and therefore we should use the general form of the equation represented by equation [2]. For the particular conditions of the Critical State, equation [2] can be rewritten as:

$$\tau_c = (\sigma - u_a) \tan (\phi^a)_c + (u_a - u_w) \tan (\phi^b)_c \qquad [4]$$

and taking the cohesion intercept to be zero for critical state conditions (Atkinson, 1993: p. 116).

Frameworks for the Critical State for unsaturated soils have been proposed by Alonso et al. (1990) and Toll (1990). Toll (1990) and Toll & Ong (2003) expressed the unsaturated critical state concept (represented by equation [4]) in the more general stress invariants:

$$q = M_a (p - u_a) + M_b (u_a - u_w) \qquad [5]$$

For comparison with the “effective stress” approach (Bishop, 1959; Khalili and Khabbaz, 1998) the Critical State would be given by:

$$q = M p' \qquad [6]$$

$$q = M [(p - u_a) + \chi (u_a - u_w)] \qquad [7]$$

$$q = M (p - u_a) + \chi M (u_a - u_w) \qquad [8]$$

Therefore, for comparison with Toll’s approach, the “effective stress” approach implies:

$$M_a = M \qquad [9]$$

$$M_b = \chi M \qquad [10]$$

For comparison with the Barcelona Basic Model (BBM) (Alonso et al., 1990), the BBM assumes that the contribution from net stress is constant and equal to the saturated critical state stress ratio, M. In the BBM the contribution from matric suction is represented as a decrease in the intercept of the Critical State Line (CSL) on the p-u_a axis defined by a parameter k. Therefore the relationships in the BBM are:

$$M_a = M \qquad [11]$$

$$M_b = kM \qquad [12]$$

The major difference in the Toll (1990) approach is that M_a and M_b change with degree of saturation or fabric of the soil. The Khalili and Khabbaz approach assumes that M (and hence M_a) is a constant but that χ, and hence M_b, varies as a function of suction. The BBM approach assumes that M and k are constants (implying M_a and M_b are both constant).

The validity of these three approaches will be examined with reference to the test results on an unsaturated structured soil.

3 MATERIALS AND TEST PROCEDURES

The artificial cemented sand was made from sand and kaolin. A mixture of sand:kaolin (87%:13%) was fired at 500°C for 5 hours. The sand used was Leighton Buzzard sand, classified as uniform coarse sand (

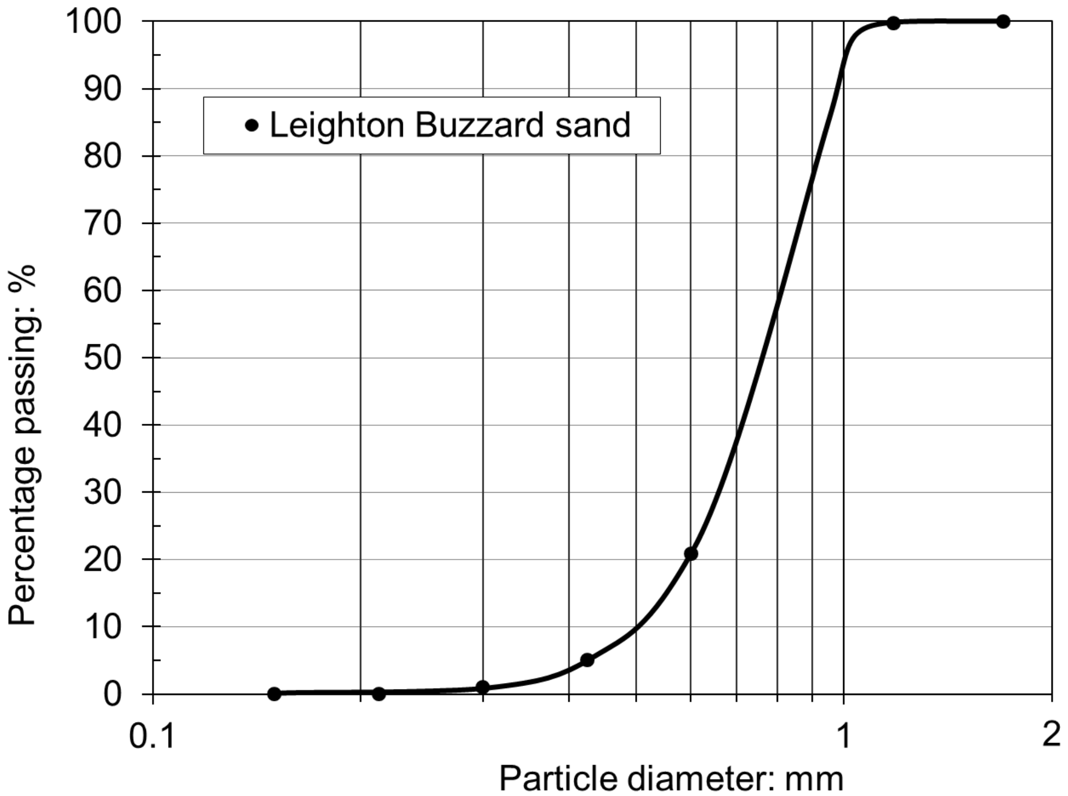
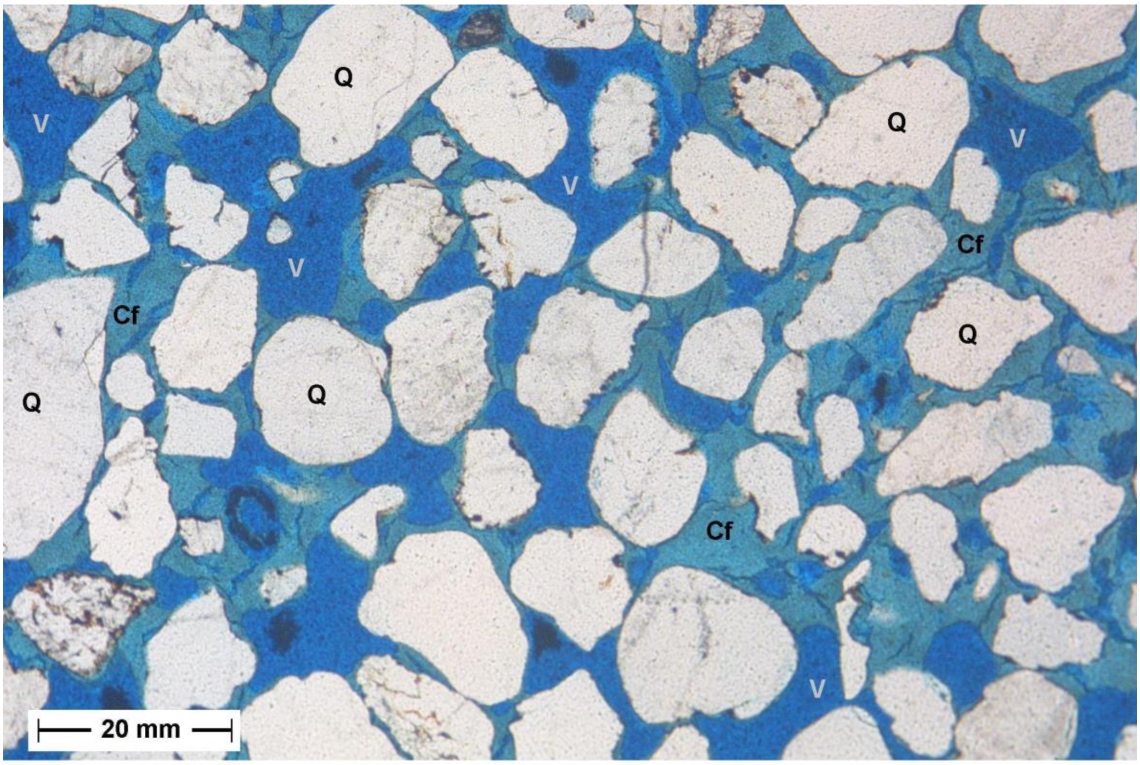


Figure 1). The dry sand and kaolin were mixed initially, and then distilled water was added to allow the wet kaolin to attach to the sand particles. The sand and kaolin were stirred with a spatula to get a uniform mixture. Firing the kaolin at 500^{o}C changes the nature of the kaolin (as used in china making) and creates bonds between the sand particles. This technique was first adopted by Maccarini (1987) to produce weakly bonded soils in the laboratory.

Micro-structural observations clearly indicated that fired kaolin established bonded “bridges” between particles (



Notes:
Q - quartz Cf - fired kaolin V - void

Figure 2) although in some parts of the section, it coated whole sand particles. Very few sand particles had direct particle to particle contact at least as far as can be seen in a single thin-section. The voids can be isolated (when surrounded by fired kaolin) or connected to each other to form larger voids (Ali Rahman et al., 2010).

The advantage of this technique for preparation of bonded soils is that the bond does not change over time and no curing period is required. Details of the technique for preparing the soil samples in this study are given by Ali Rahman (2008).

All samples tested in this study (38 mm diameter by 76 mm high) were prepared at a constant initial void ratio (e = 0.6) and with a uniform strength of bond (defined by the firing temperature of 500^{o}C and the period of 5 hours). A series of triaxial tests in drained and undrained conditions on saturated specimens of the same cemented sand were reported by Ali Rahman (2008) and Ali Rahman et al. (2010). This paper presents the results of 22 constant water content triaxial tests for testing samples in unsaturated conditions. Samples were initially saturated and then left to air-dry at laboratory temperature (22°C) and humidity (typically 35-45%) to achieve the required water contents.

The water retention curve for the artificial cemented sand samples is shown in

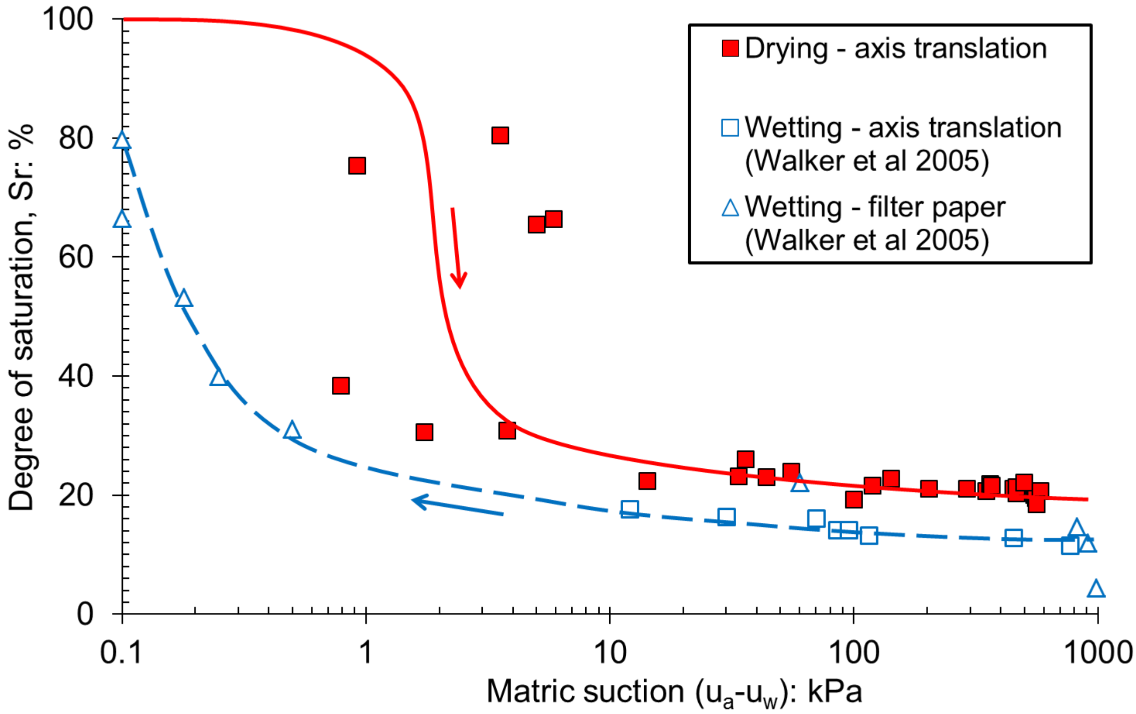


Figure 3. The curve is extremely steep in the small suction range of 2-4kPa (with some scatter in the data, which is emphasised by the logarithmic scale), followed by an almost flat section once the degree of saturation falls below 25%. The initial portion can be defined as the desaturation zone (capillary zone) where free water within the macropores is removed by drying. The coarse sand desaturates very rapidly over a very small suction range as has been observed in other studies (e.g. Toker et al., 2004; Li and Standing, 2014). The flatter portion below S_r = 25% represents the residual zone, where it is harder to remove water that is held in the micropores of the fired kaolin. The suction at which the residual zone is entered is called the residual suction. As the water retention curve is very steep within the initial desaturation zone, it is quite difficult to control suction at small values for this type of

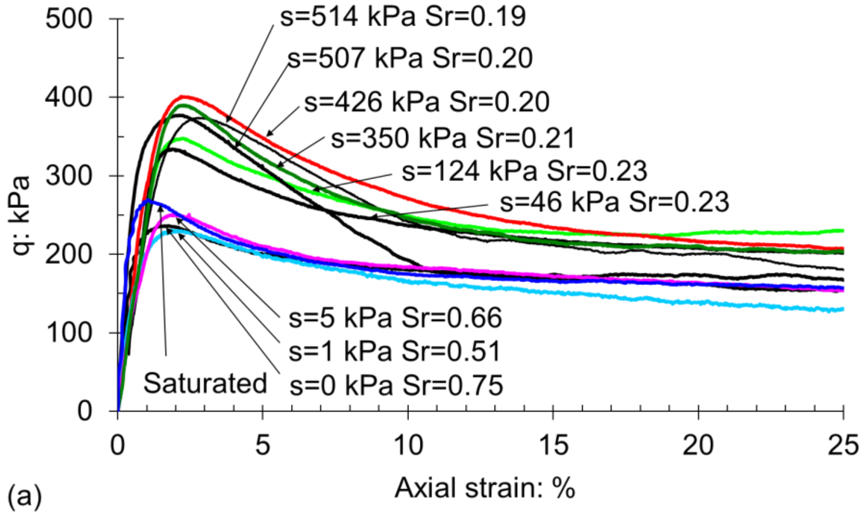
material due to the sharp change in water content (degree of saturation) for small changes in suction.

Samples prepared at the required water contents were set up in a triaxial cell equipped with a 500kPa high air entry ceramic disc fitted in the pedestal for measurement of pore water pressure. The pore air pressure was controlled through an air-line connected to a coarse filter at the top of the sample. The air pressure was slowly increased to 595kPa while also increasing the cell pressure to 600kPa in order to maintain a small net stress (σ-u_a) of 5kPa. The sample was allowed to equilibrate under this small net stress (and constant water content conditions) until a stable value of pore water pressure was observed. The initial matric suctions were then determined from the difference between the imposed air pressure and the measured water pressure (u_a-u_w).

The net stress was then increased to the desired value (50, 100 or 300kPa) by reducing the pore air pressure at constant cell pressure under constant water content conditions (allowing volume change to occur due to air flow, but preventing any flow of water). The changes in pore water pressure and volume were observed until no further volume change was observed. Samples were then sheared in triaxial compression under constant water content conditions. Measurements of pore water pressure and volume change were made during shearing.

4 CONSTANT WATER CONTENT TRIAXIAL TESTS ON UNSATURATED SAMPLES

Stress-strain curves for the unsaturated constant water content triaxial tests on the cemented sand are shown in



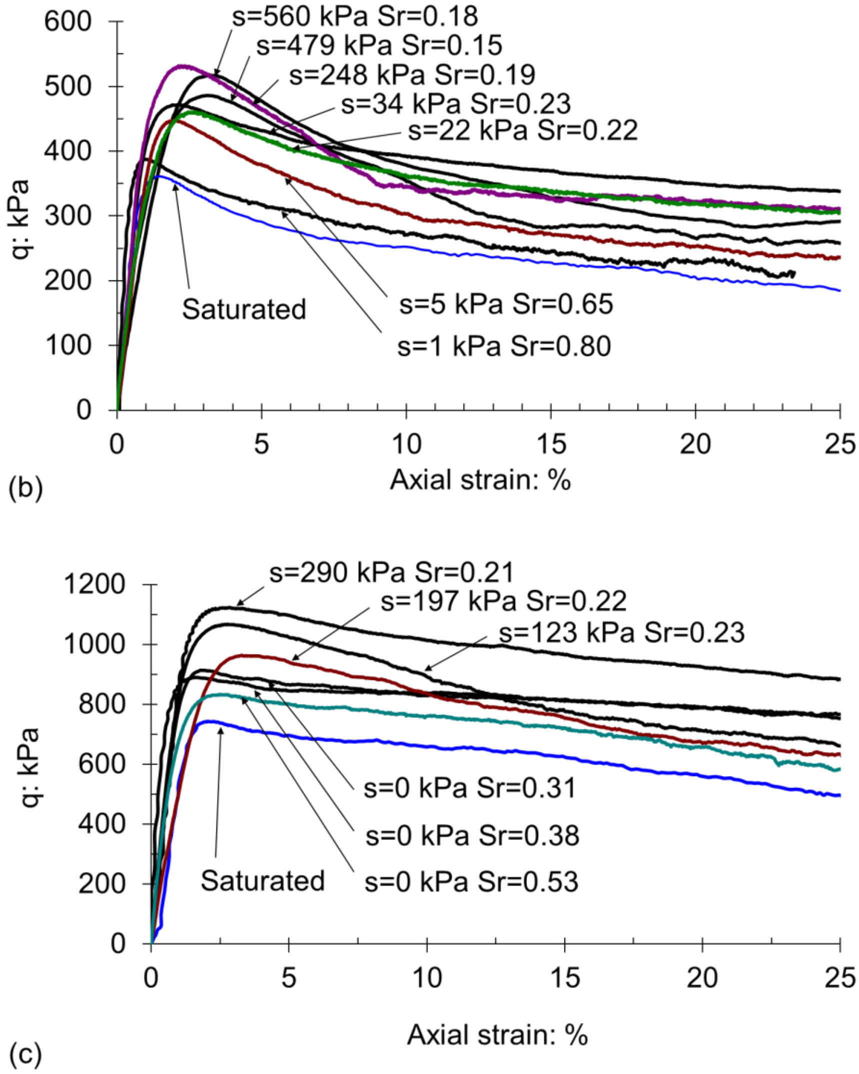
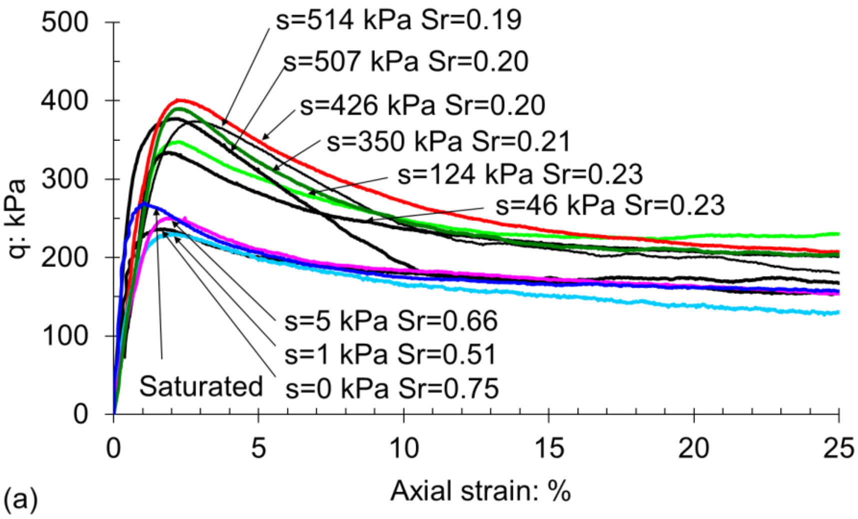
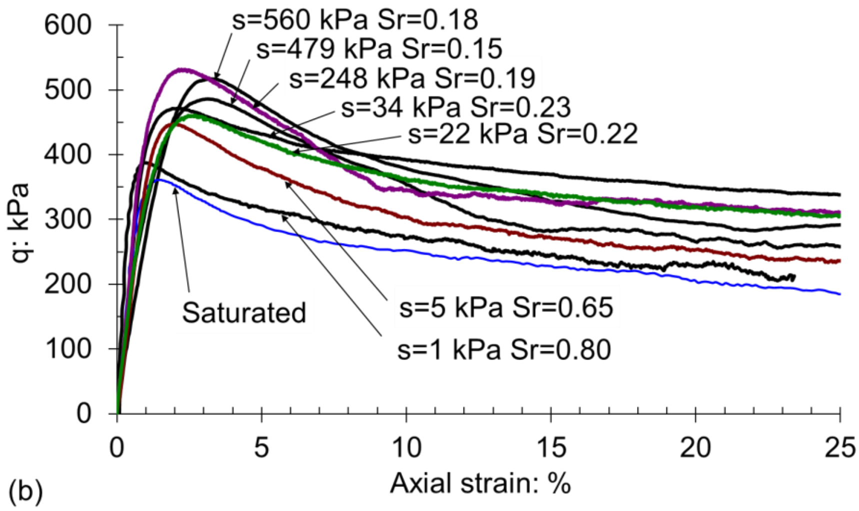


Figure 4(a)-(c) (for net stresses of p-u_a = 50, 100 and 300 kPa respectively). The values for suction (s) and degree of saturation (S_r) at the start of shearing are identified for each test. The equivalent saturated drained test at an initial mean stress of 50, 100 or 300 kPa is also shown in each plot (marked as “Saturated”).

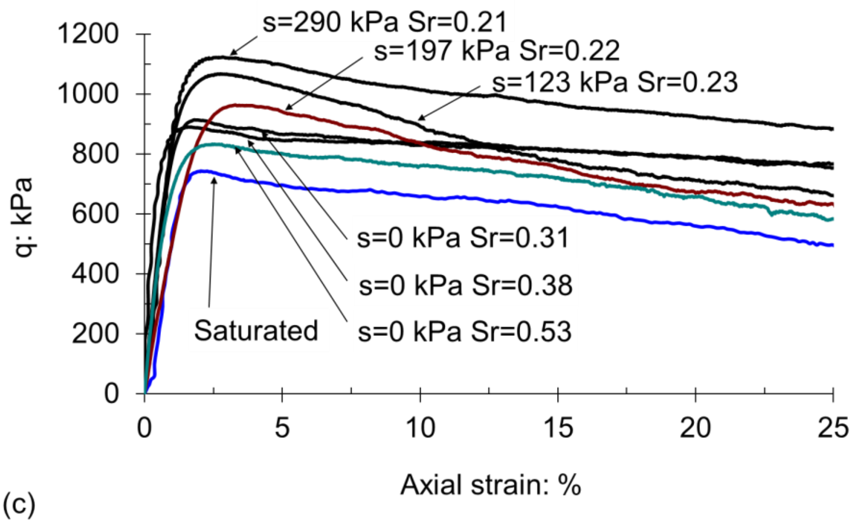


(a)

It will be seen from



(b)



(c)

Figure 4(a)-(c), that there are limited data for degrees of saturation between 22% and 50% and suctions between 5 kPa and 30 kPa. As noted earlier, it was difficult to control the values obtained due to the sharp change in degree of saturation for small changes in suction in this region (

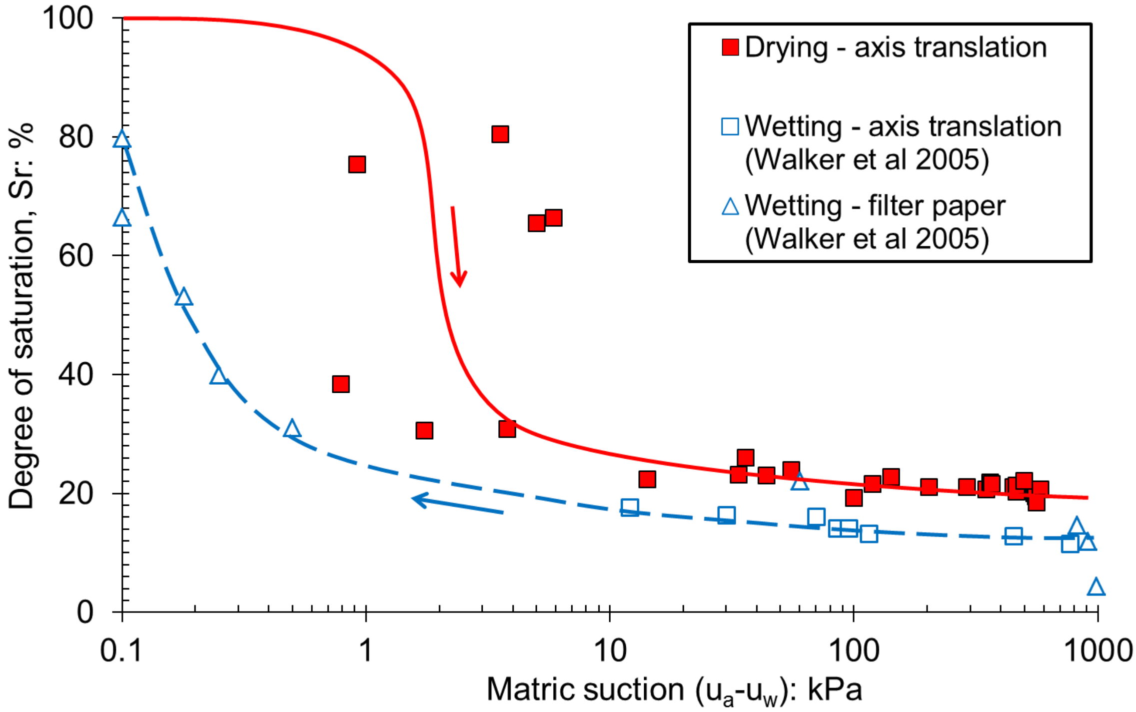


Figure 3).

Generally the tests on unsaturated samples show strengths higher than the equivalent "Saturated" test due to the presence of suction (

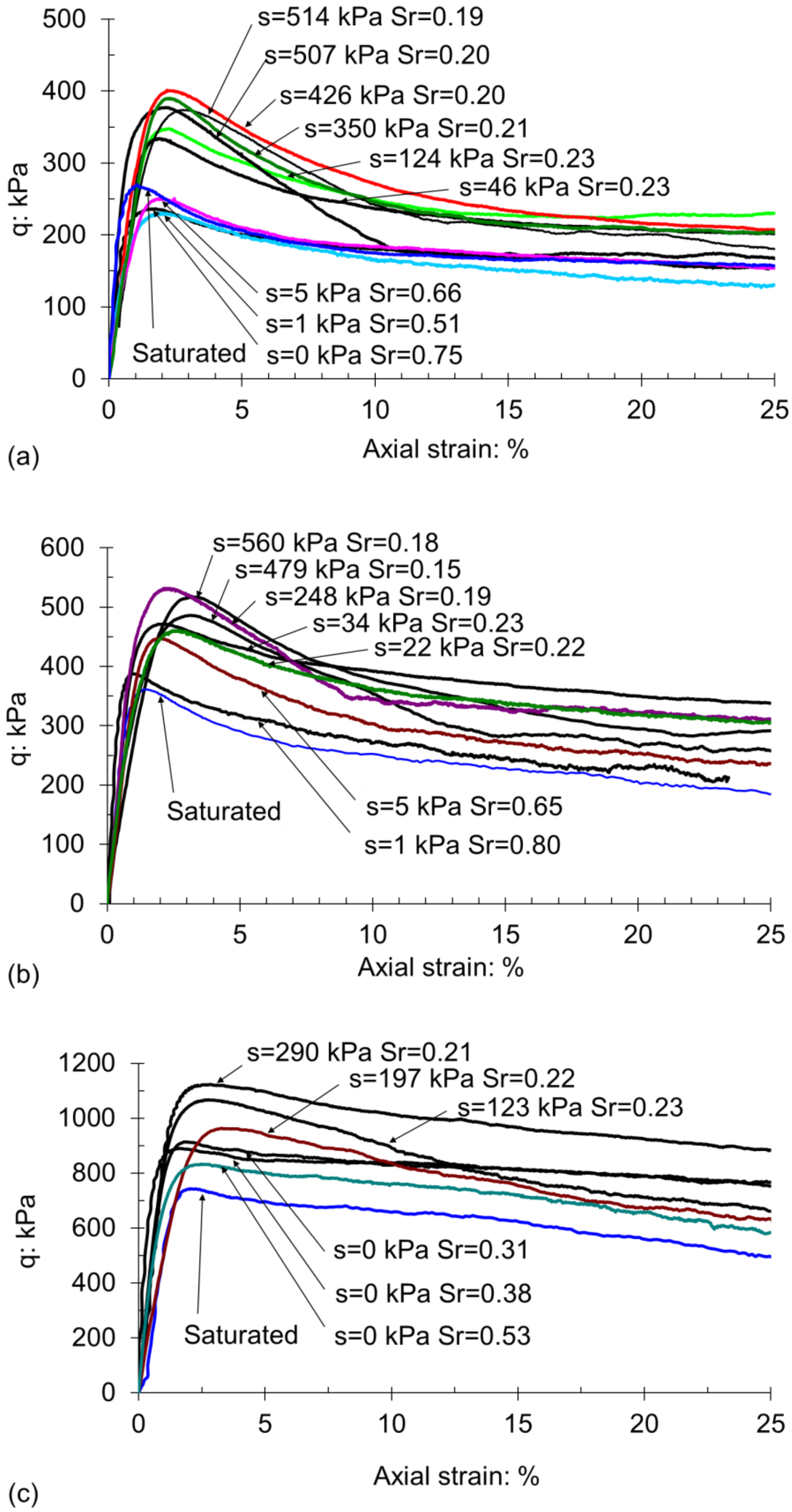


Figure 4(a)-(c)). The increase in strength can be as much as 1.5-2 times the equivalent saturated strength. Some of the low suction tests (suction <5 kPa) at a net stress of 50 kPa do show results quite similar to the saturated reference test tests (

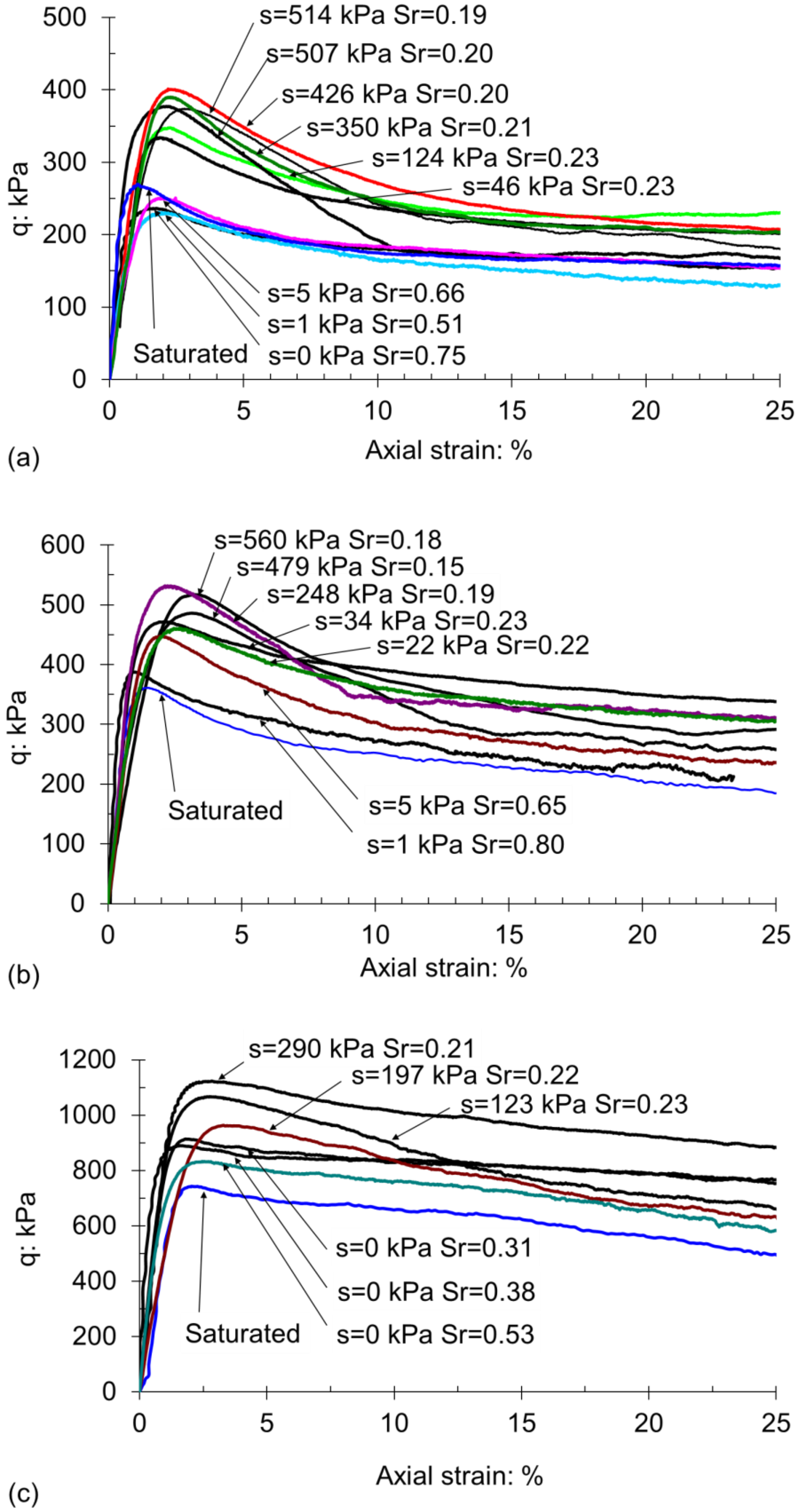
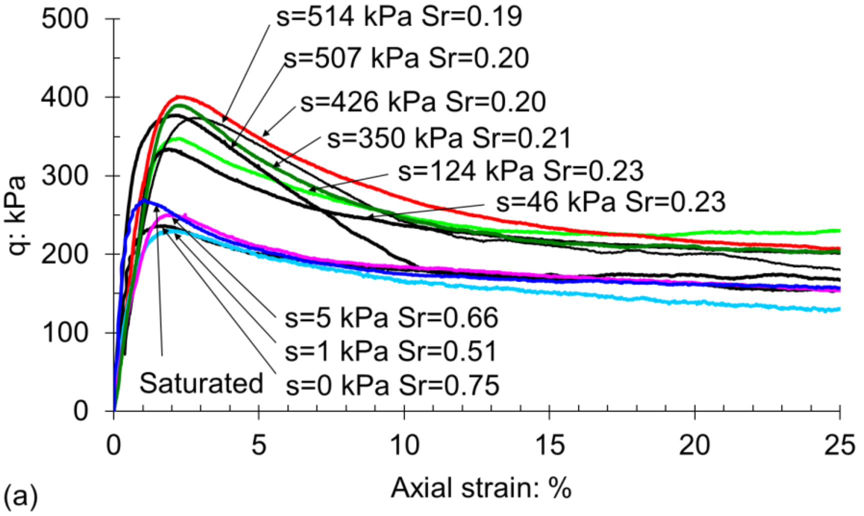
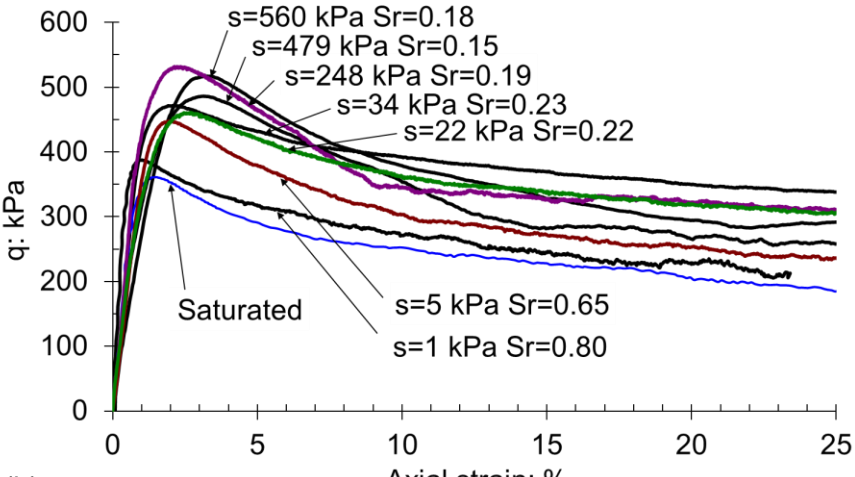


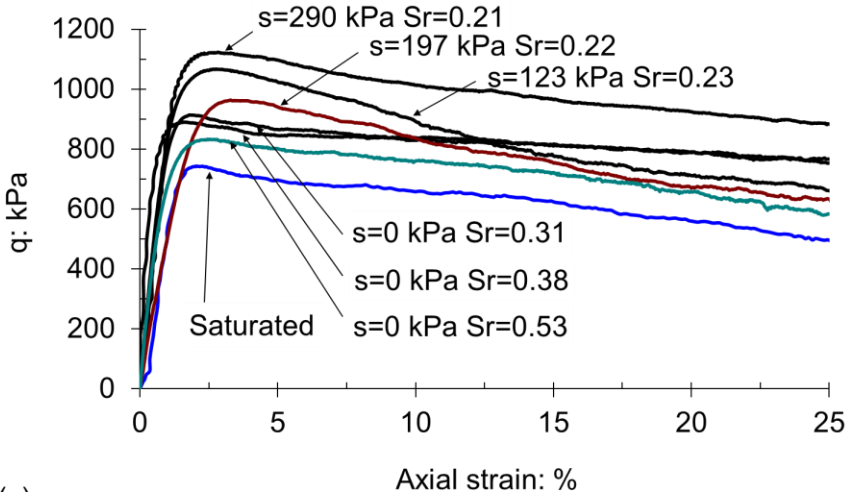
Figure 4(a)). However, at larger suctions there is a significant increase in strength. For the low



net stress case ((a)

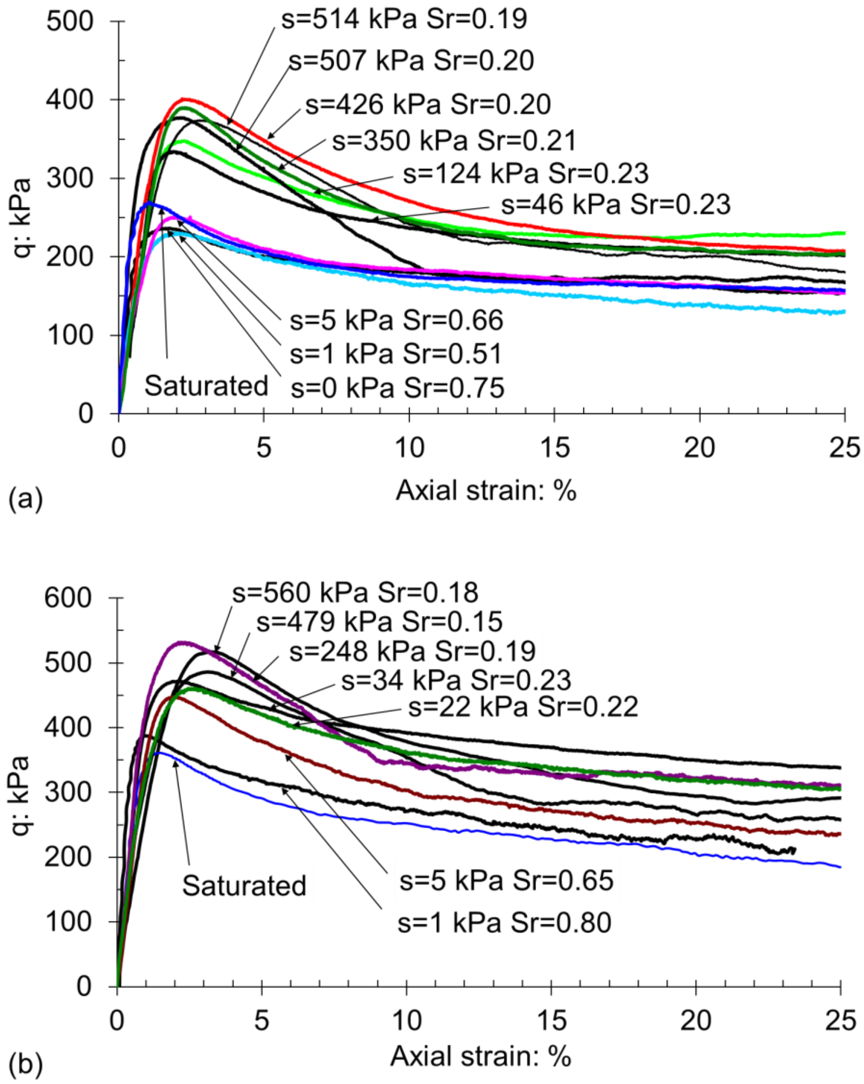


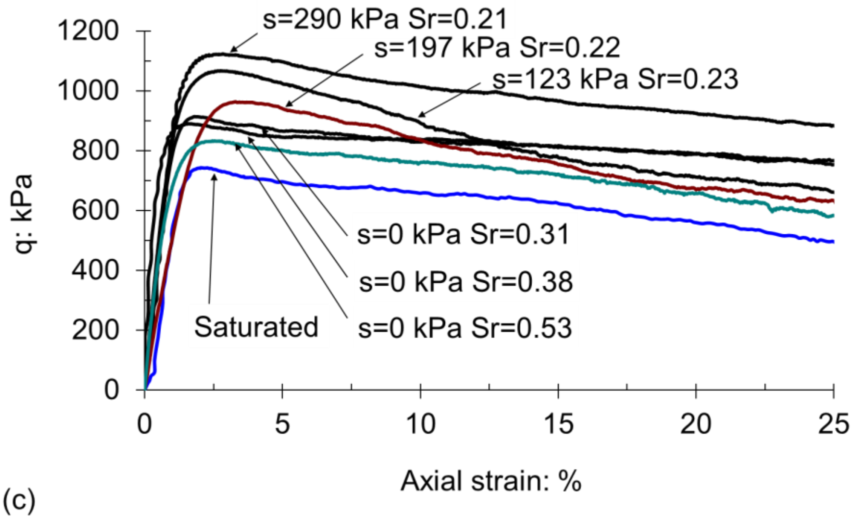
(b)



(c)

Figure 4(a)) it can be seen that the strengths increase markedly when the degree of saturation drops below 50%. Such changes in strength due to suction are more significant at low net stress, as the suction contribution will be relatively greater when the net stress is small.

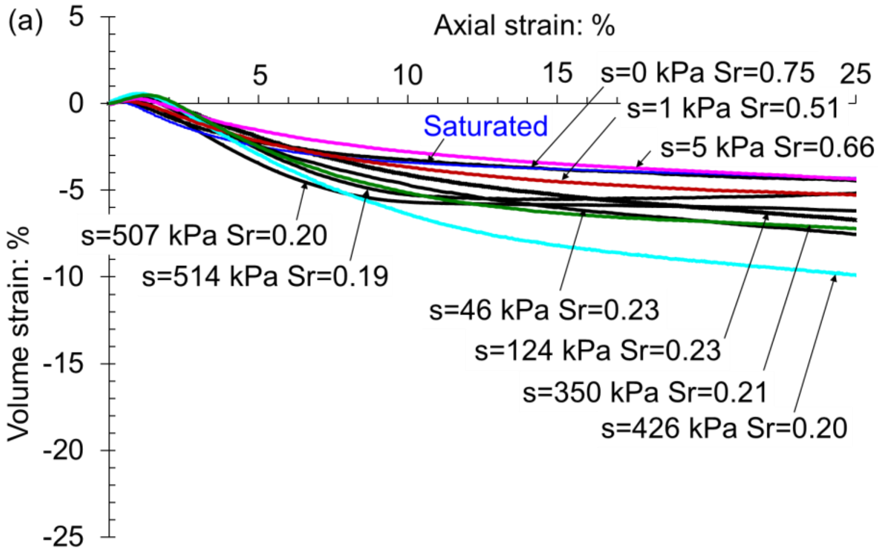




(c)

Figure 4(a)-(c) show samples achieving a peak state at around 2-3% axial strain then dropping to an ultimate state at around 20% axial strain. It can be seen from Figures 4(a)-(c) that the deviator stress generally approached a constant value when the axial strain exceeded 20% suggesting that the critical state was being achieved.

The volumetric behaviour under shear is shown in



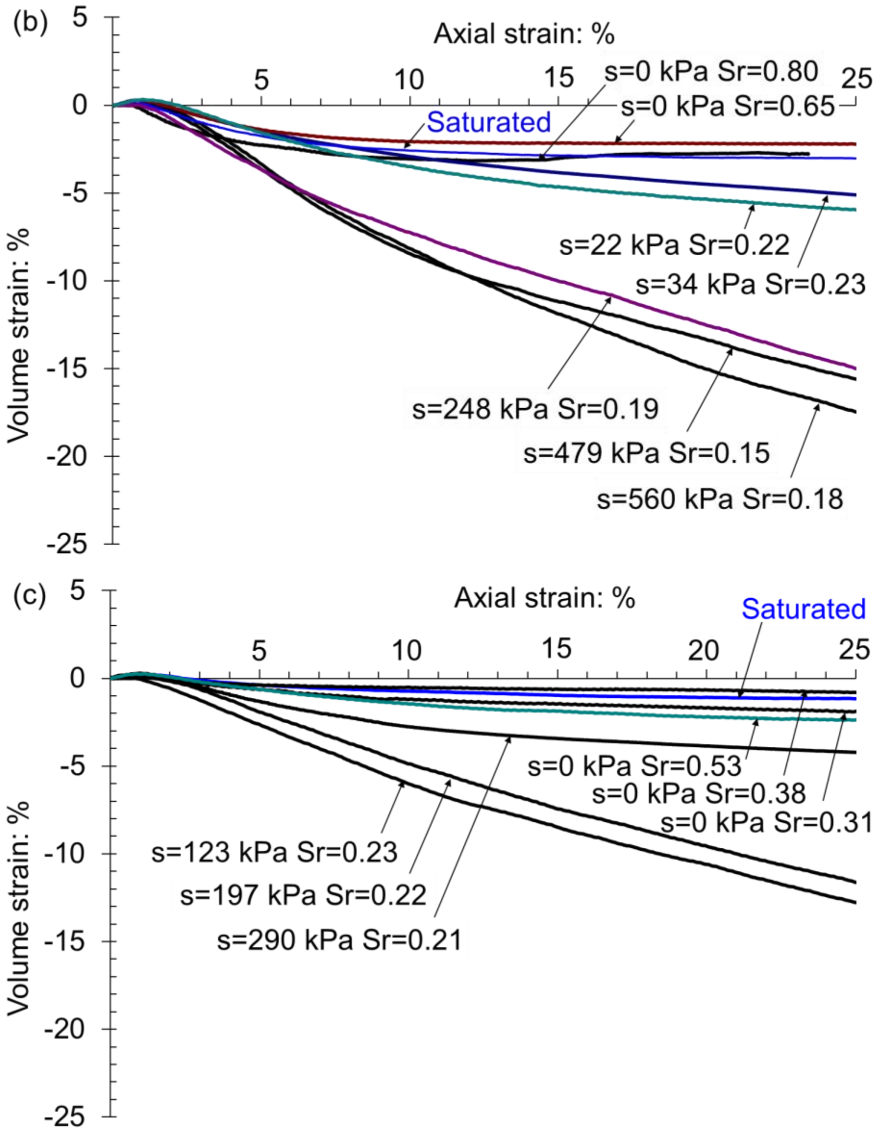


Figure 5(a)-(c) for the three net stress levels of 50, 100 and 300 kPa respectively. The effect of suction can be seen in each series of tests. In

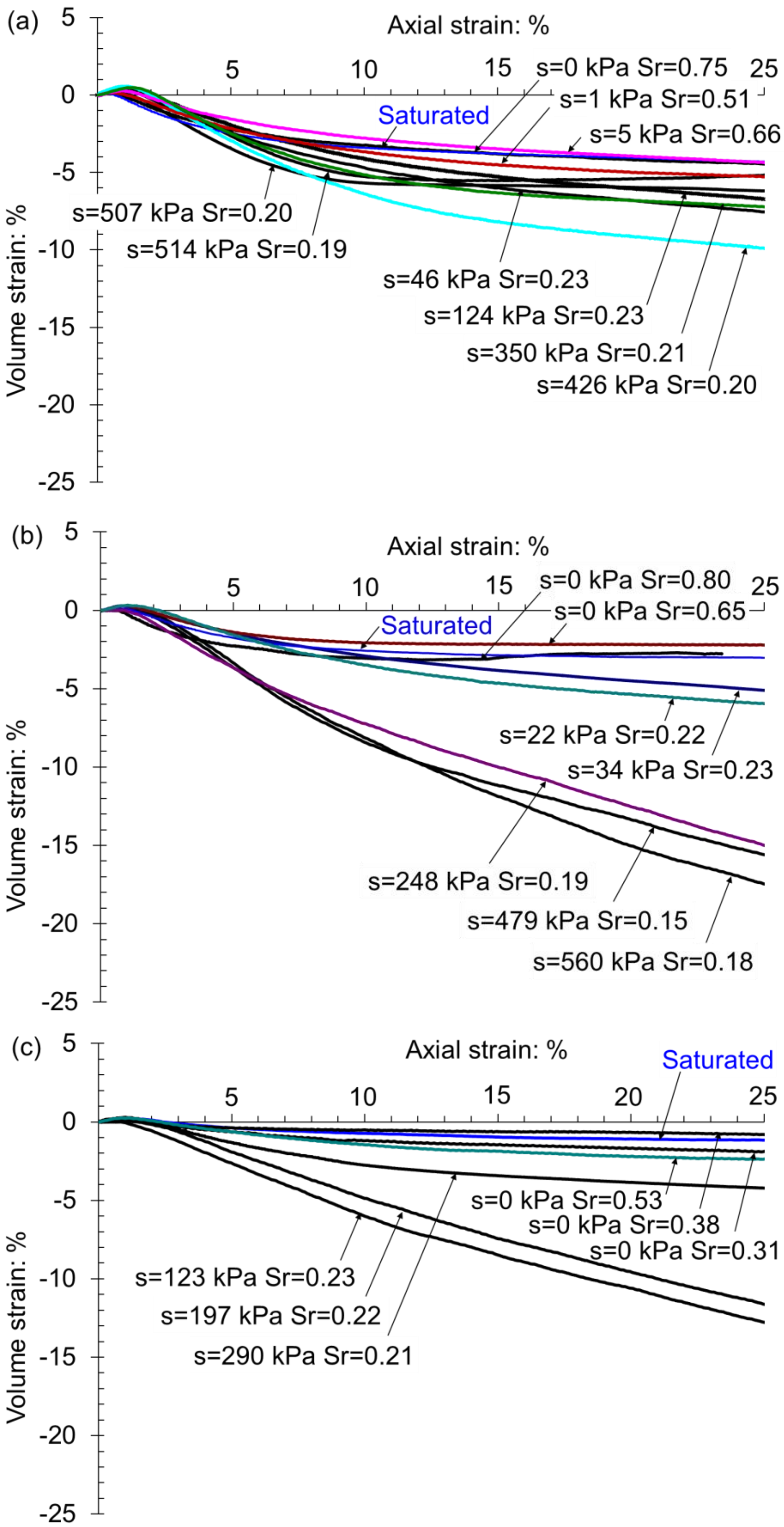


Figure 5(a), for tests at p-u_a = 50kPa, samples with high initial matric suctions (350kPa, 426kPa, 507kPa and 514kPa) show larger rates of dilation than the tests with zero and low suctions. A similar trend can also be seen in

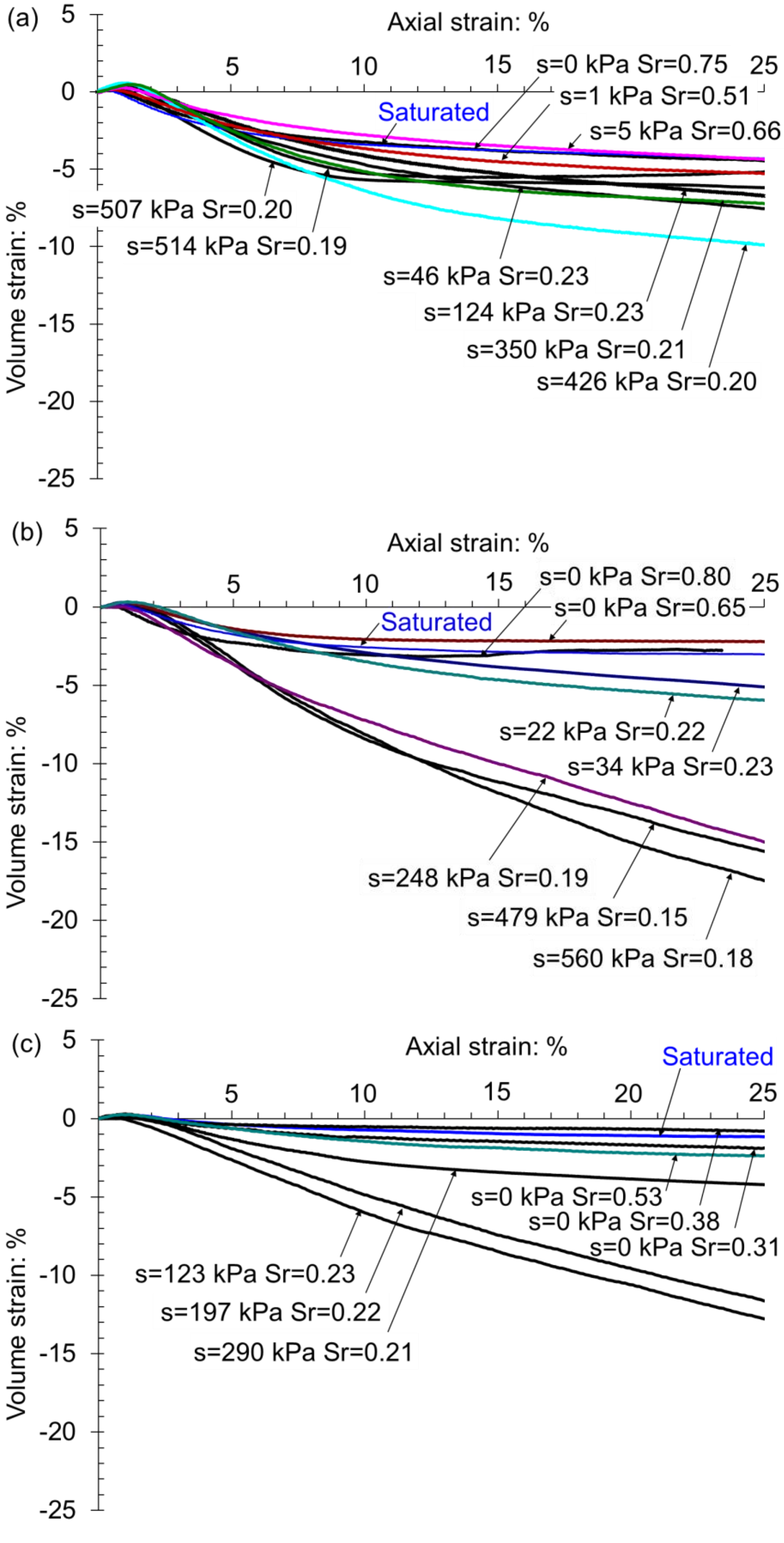


Figure 5(b) & (c) for tests sheared under p-u_a = 100kPa and 300kPa. In

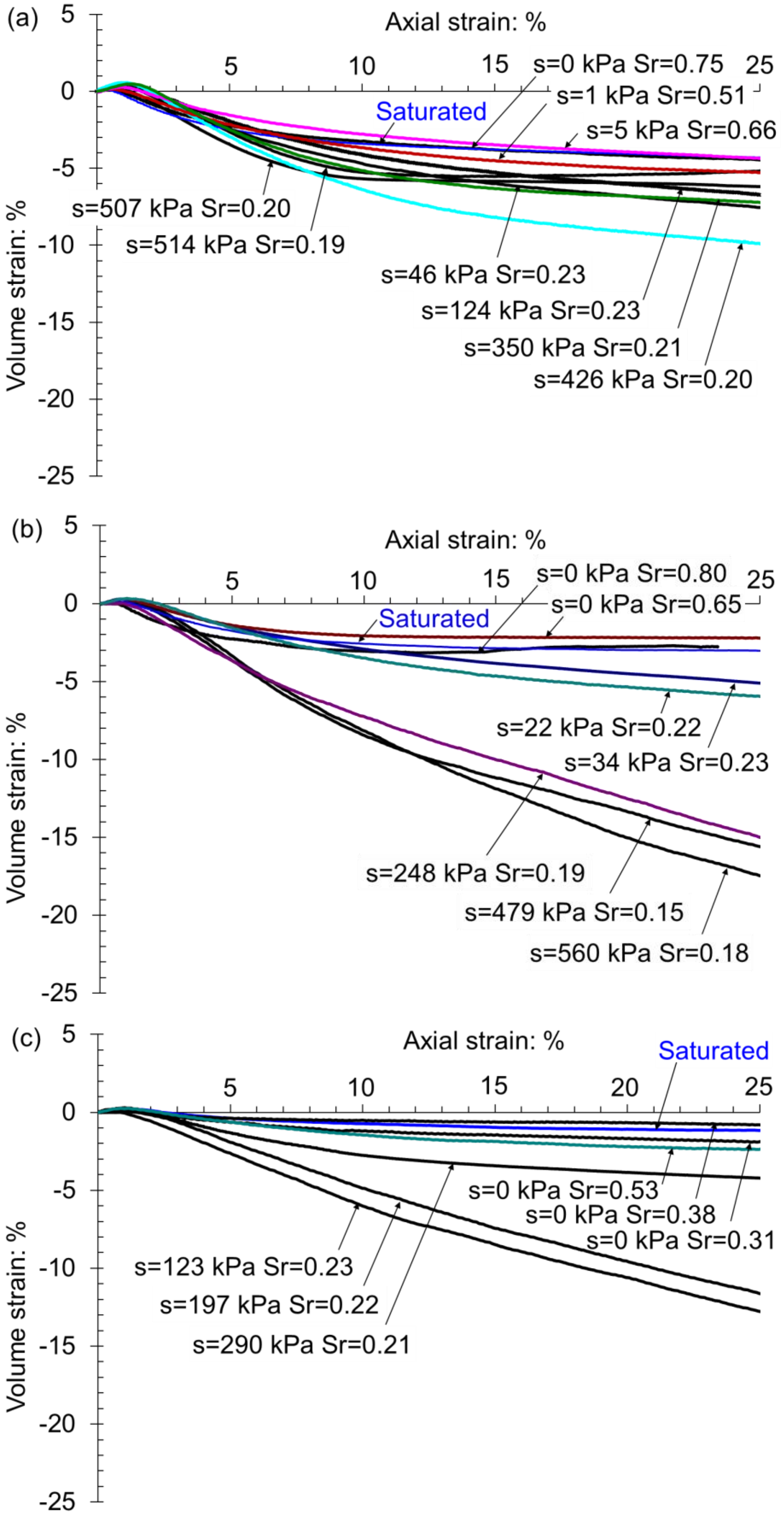
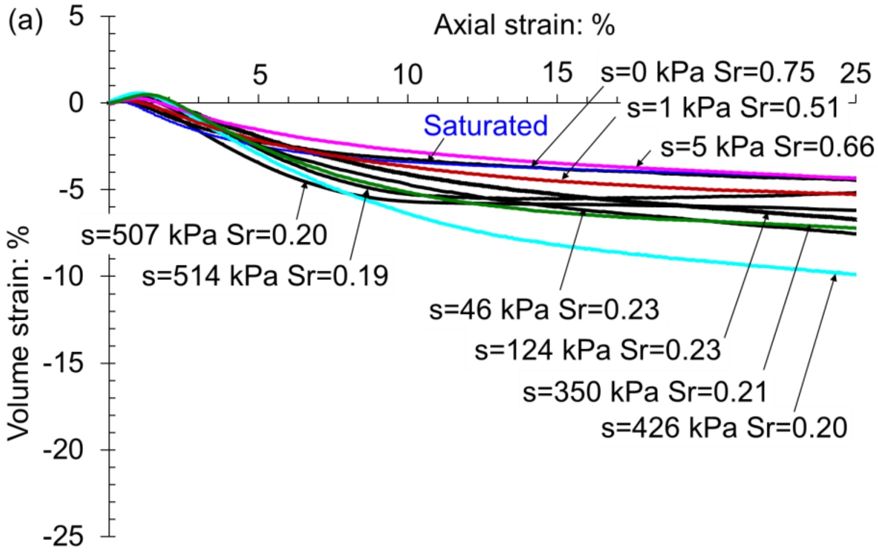


Figure 5(b), samples with initial matric suctions of 248, 479 and 560kPa exhibit a greater degree of dilatancy compared to samples having zero and low suctions. This can also be seen for tests at initial matric suctions of 123, 197 and 290kPa in



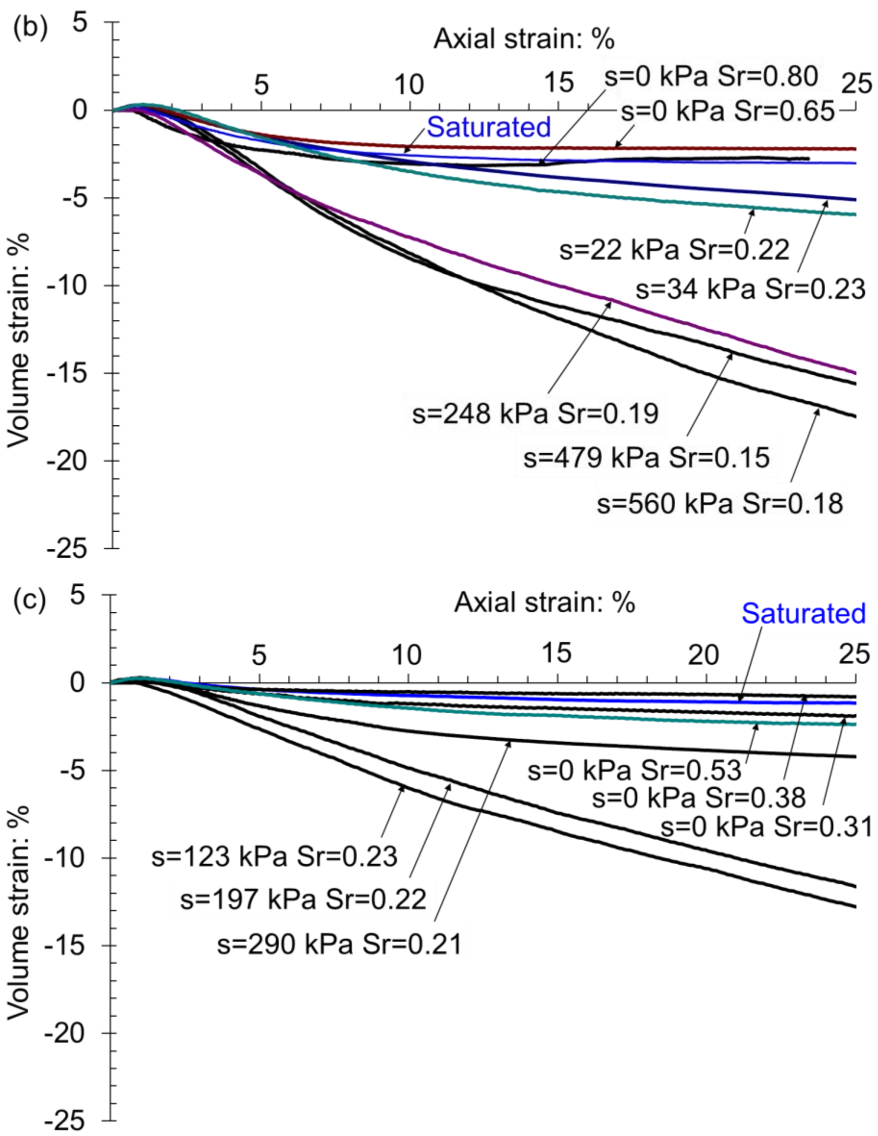
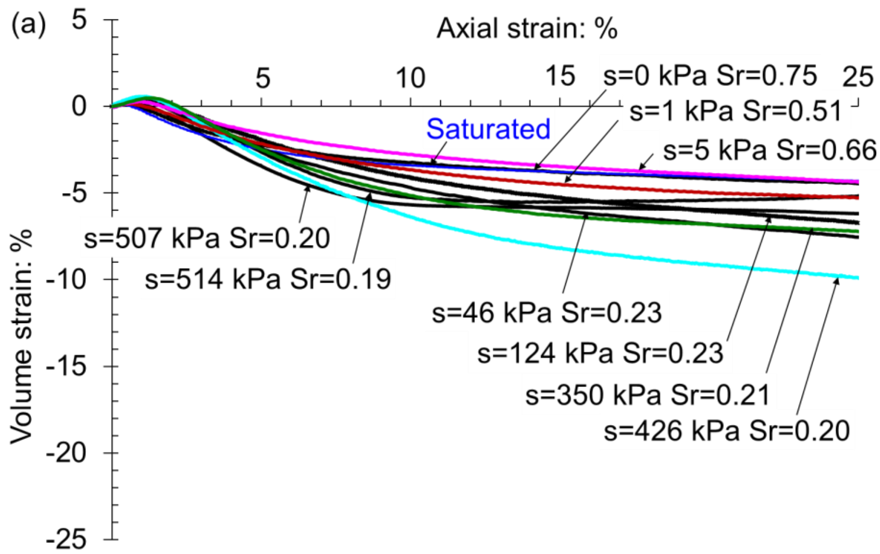


Figure 5(c). This behaviour was recognised by Toll (1988). He noted that this was the opposite of what would be expected if the suction was to be considered as equivalent to effective stress in a saturated condition. In a saturated condition, samples consolidated at higher effective stress, p', tend to show less dilatancy. Toll (1998) noted that this apparently opposite effect could be explained by considering the fabric of an unsaturated soil i.e. suction can act to hold groups of particles together, making the soil behave as if has a coarser grading, and hence is more dilatant.



It can be seen from

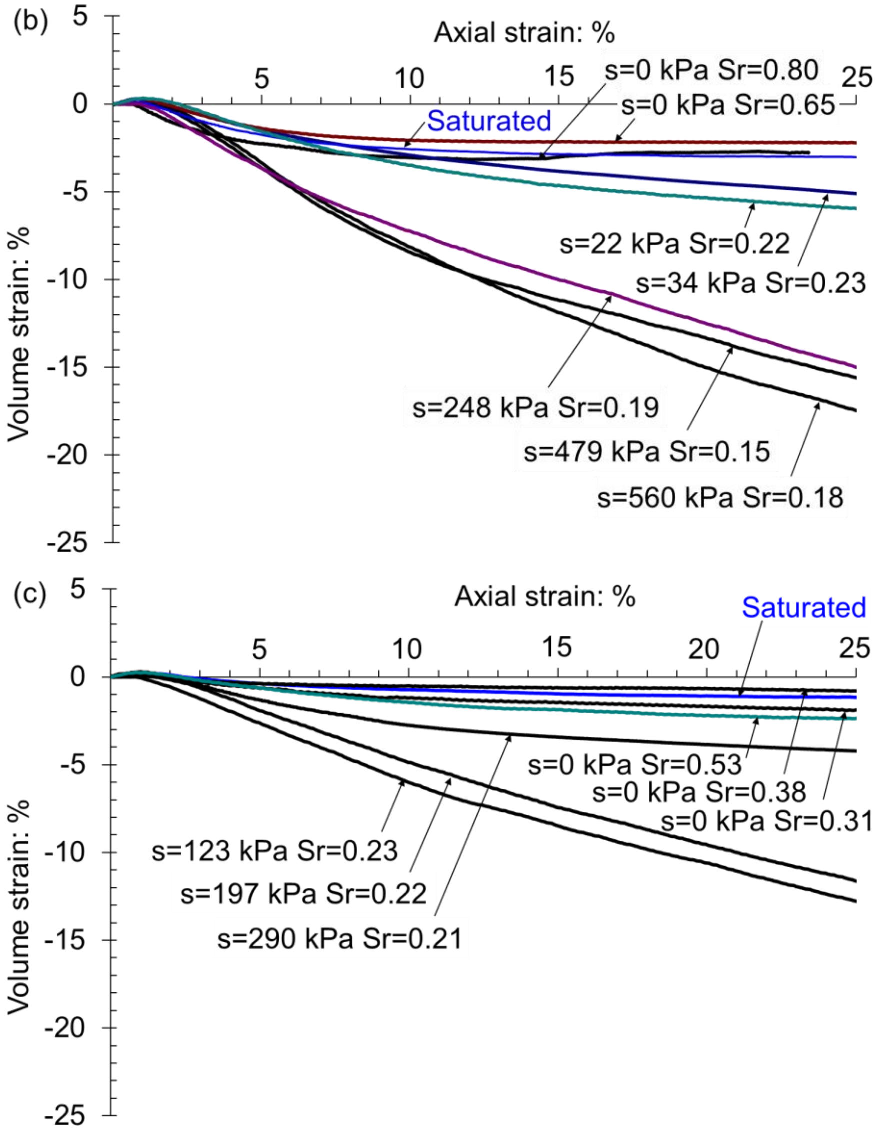
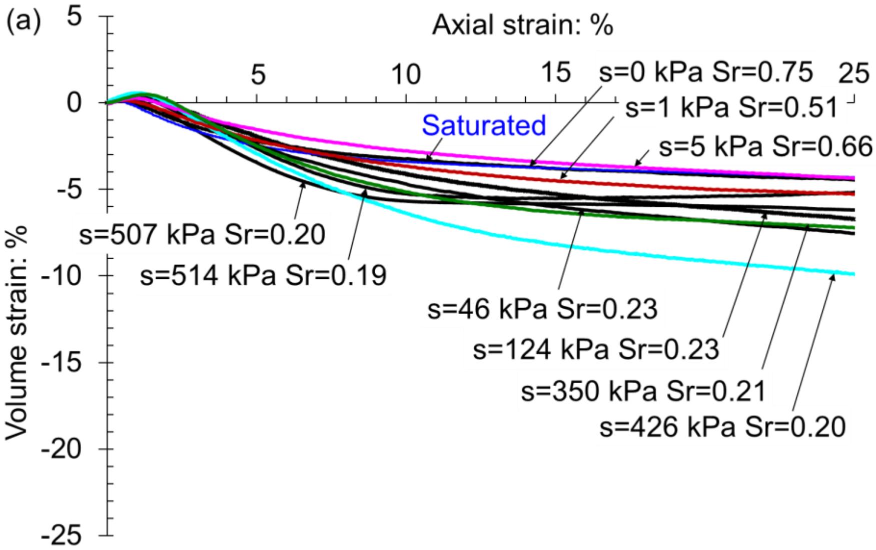


Figure 5(a) that for the tests at p-u_a =50 kPa, the volume strain levelled off at strains greater than 20%, confirming that the Critical State was being approached. For the tests at p-u_a =100



kPa, shown in

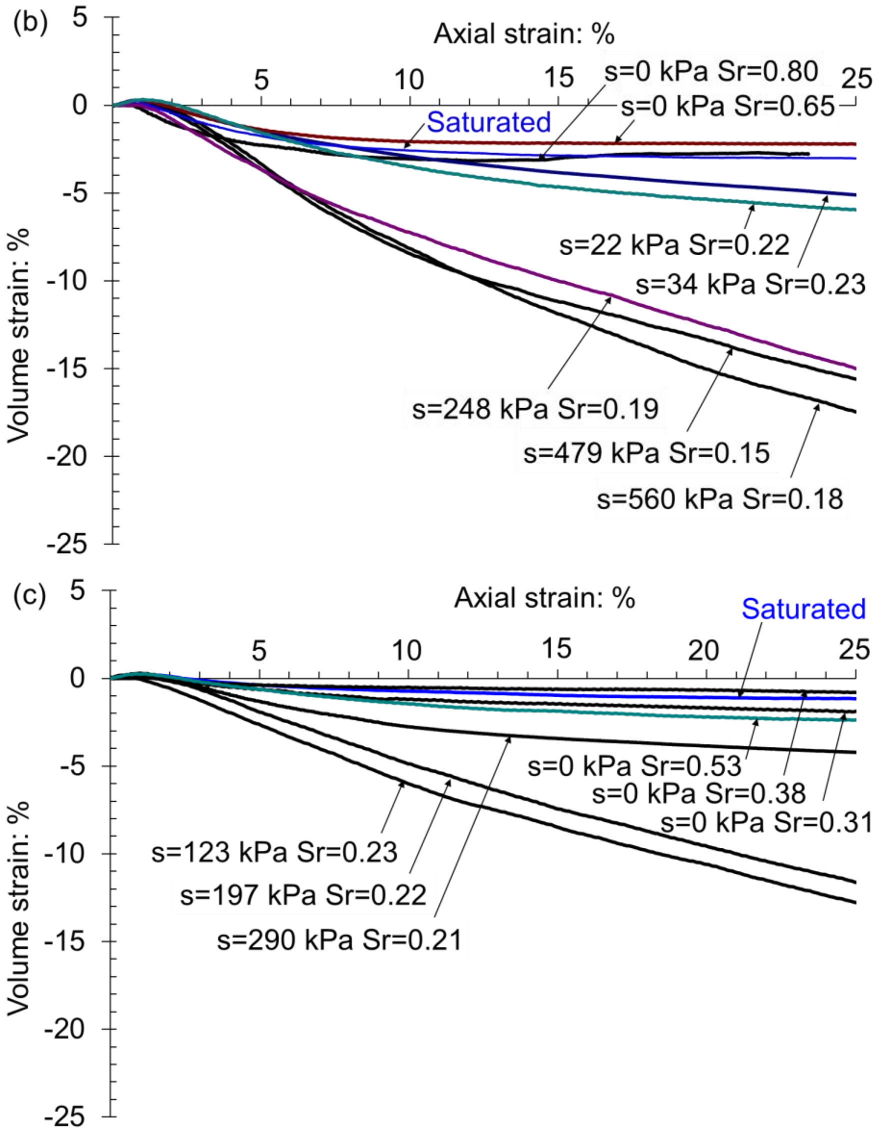


Figure 5(b), the same can be seen for tests where the initial suction was less than 200kPa. However, the three tests at initial suctions of 248, 479 and 560 kPa show volume change continuing even at 25% axial strain. It is clear that a true Critical State has not been achieved for these three tests. Similarly, in

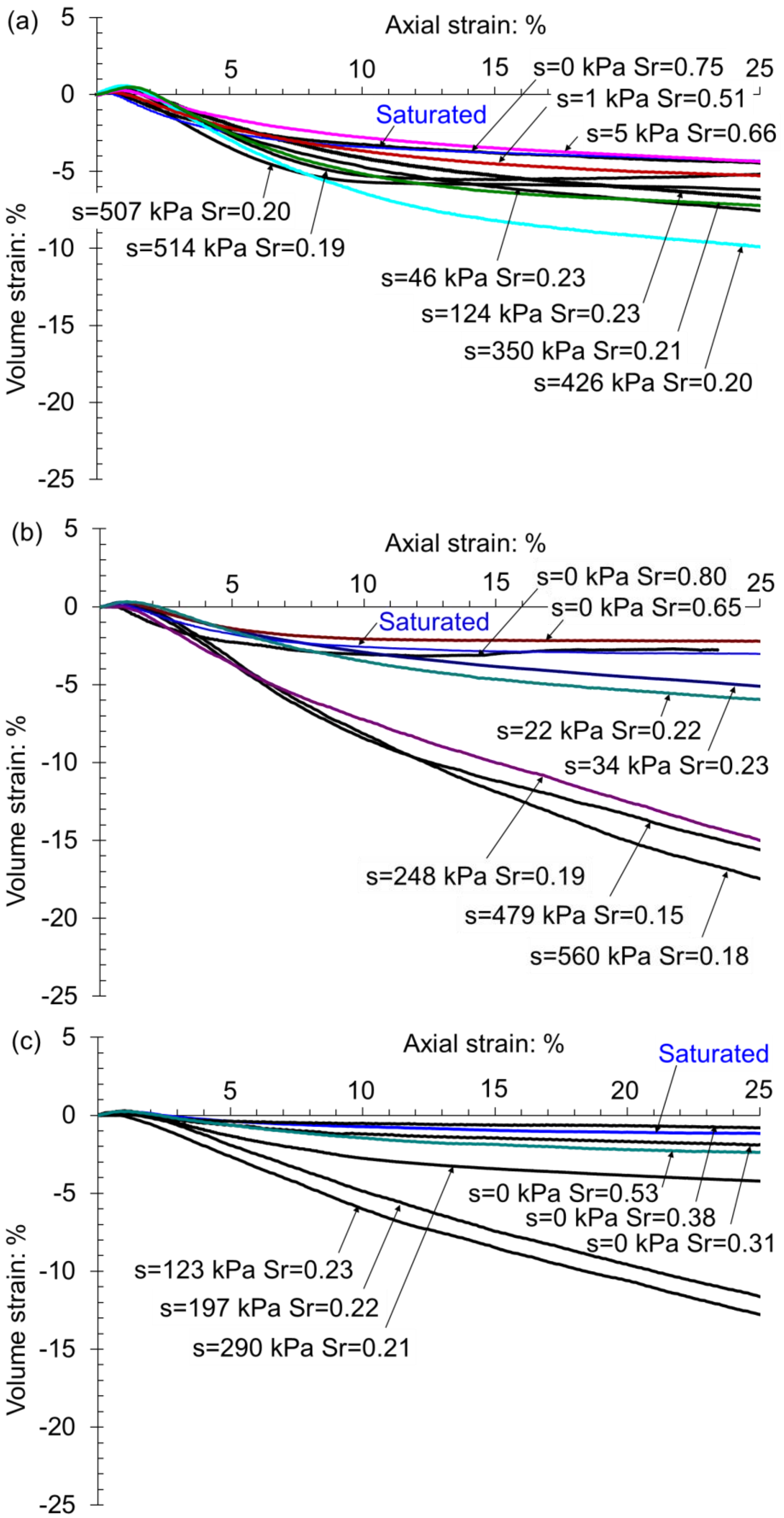


Figure 5(c) the same observation can be made for tests at initial suctions of 123 and 197 kPa.

5 CRITICAL STATE STRESS RATIOS

To examine the Critical State for this material, Critical State values have been interpreted from the triaxial test results based on end of test conditions. For the five tests where the volume strain continued to change at the end of test (as discussed in Section 4), data points shown on the figures will be indicated by a different symbol to indicate uncertainty about the true Critical State point, and the implications will be discussed later.

A method of analysis is needed to separate the two components of net stress and suction that influence the deviator stress at the Critical State, in order to calculate the two stress ratios M_a and M_b. This has been done in a number of ways as explained below.

5.1 *Net stress Component, M_a*

To estimate the stress ratio due to net stress, M_a, tests at low suctions have initially been considered. Eight tests were carried out where the suction at the critical state was 6 kPa or less. For these tests, the $M_b\ (u_a\text{-}u_w)$ term in equation [5] becomes small and the controlling component will be the $M_a\ (p\text{-}u_a)$ term. If the suction is zero then:

$$M_a = \frac{q}{(p-u_a)} \qquad [13]$$

However, since the suction values are non-zero, the values could have a small effect (particularly at lower values of net stress).

A second assumption could be that the values of M_a and M_b would be equal at high degrees of saturation (Toll, 1990). This assumption would lead to:

$$M_a = M_b = \frac{q}{(p-u_a)+(u_a-u_w)} = \frac{q}{(p-u_w)} \qquad [14]$$

However, some of the degrees of saturation are less than 50% even for the tests at low suction. Therefore, this assumption may not be valid.

A third possible assumption is to assume that the values of M_b would be equal to M_s (the saturated critical state ratio). This assumption would lead to:

$$M_a = \frac{q - M_s(u_a - u_w)}{(p - u_a)} \qquad [15]$$

All three assumptions have been used to calculate M_a in Table 1. The range of average values for M_a is 1.38 to 1.41. These values equate to an equivalent angle of shearing resistance of $(\phi^a)_c$ = 34-35°.

The values of deviator stress, q, at the Critical State from saturated tests have been plotted against corresponding values of mean effective stress, p', in

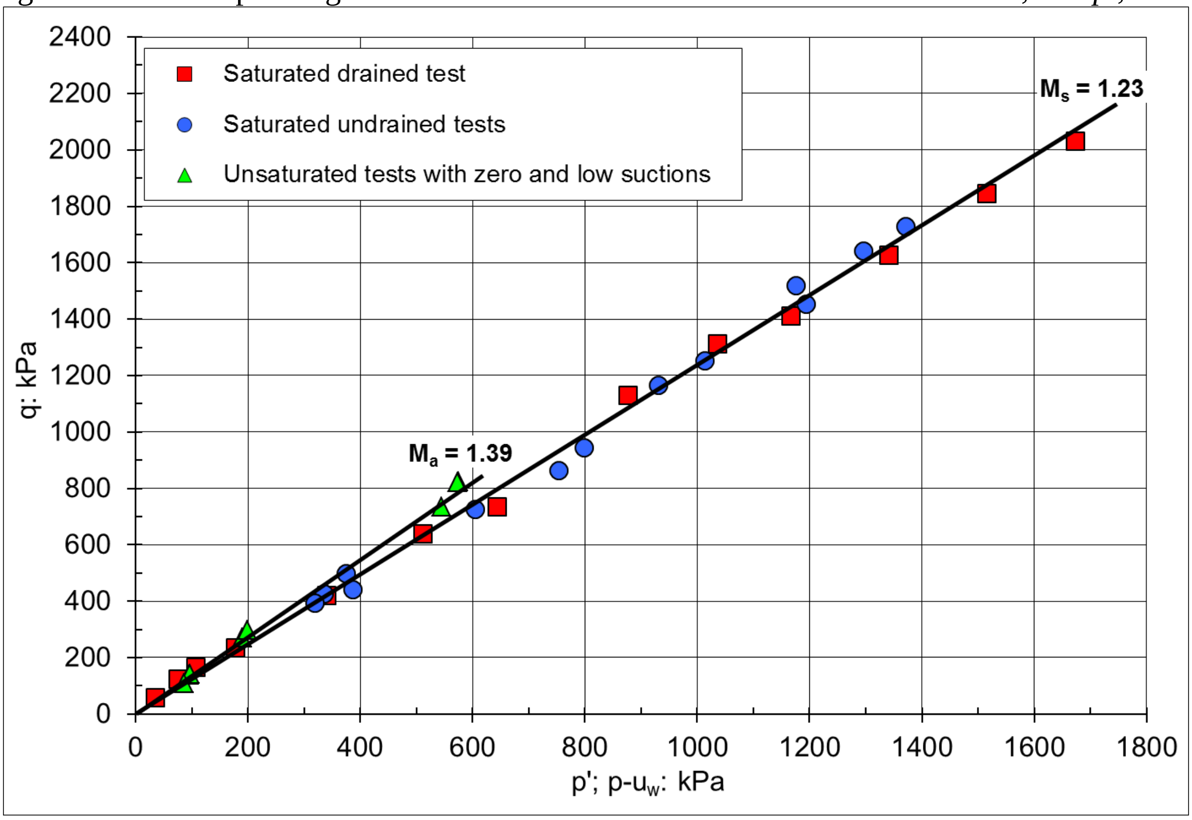


Figure 6. The values for the unsaturated (low suction) tests have been superimposed on the plot. It can be seen that the value of M_a =1.39 (consistent with values in Table 1) is higher than the saturated value of M_s = 1.23, which is equivalent to $\phi'_c = 31^o$.

Values of M_a can also be determined for the entire data set using the multiple regression technique described by Toll (1990) by grouping together tests with similar degrees of saturation (5 tests at a time), then creating a smoothed function. This technique gave very stable values of M_a, ranging from M_a =1.39 at the highest degree of saturation (S_r= 77%) increasing to M_a =1.68 as the degree of saturation reduced to S_r= 18%. The values of M_a are given in Table 2.

This supports the observation by Toll (2000) that we should not always make the assumption that $\phi^a = \phi'$ or $M_a = M_s$. The results for the bonded soil show a significant difference between the two values, with M_a being greater than M_s.

5.2 *Suction Component, M_b*

The values of M_b have been calculated by re-arranging Eq [5] so that M_b is given by:

$$M_b = \frac{q - M_a(p - u_a)}{(u_a - u_w)} \qquad [16]$$

M_b values were calculated using Eq[16] for the M_a values determined from the multiple regression and smoothing technique described by Toll (1990). The values of M_b calculated in this way are given in Table 2. The values in Table 2 are plotted against degree of saturation in

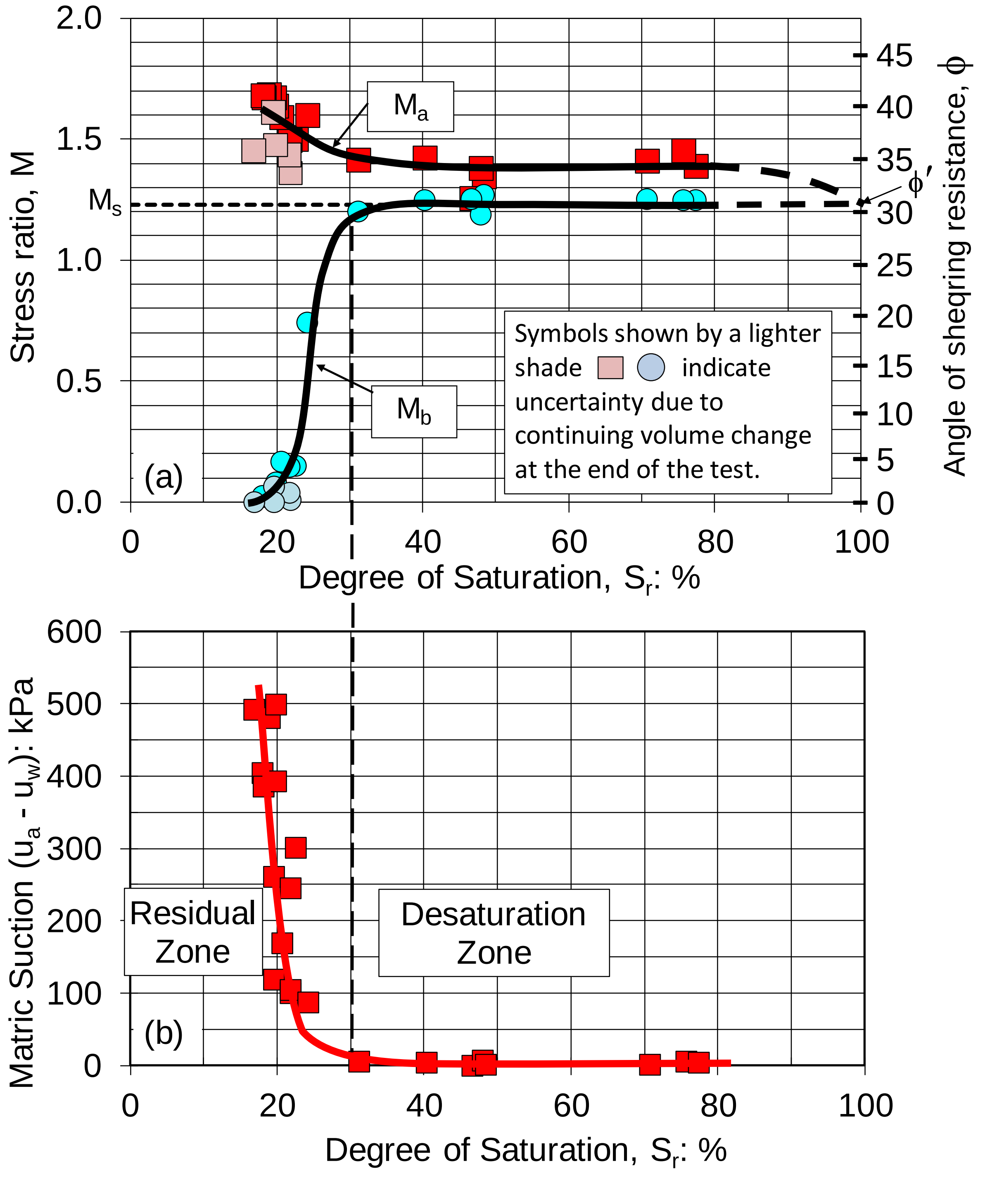


Figure 7(a) and against suction in

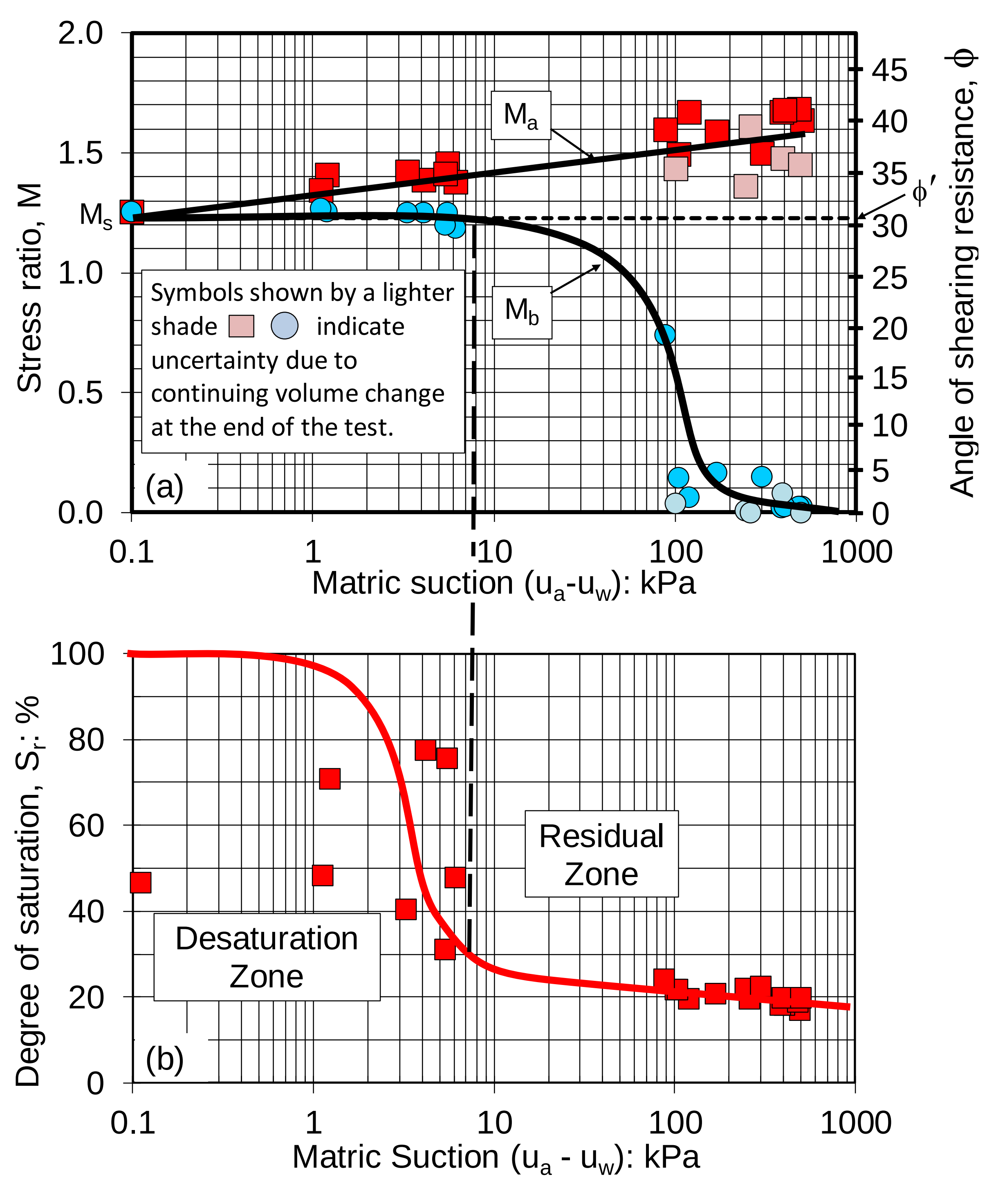


Figure 8(a). Both show a trend of M_b reducing as degree of saturation reduces (or as suction increases).

5.3 *Implications for Critical State stress ratios*

The five data points where there is uncertainty about the Critical State values (as the volume strain continues to change at the end of the test, as identified in Section 4) are shown in lighter symbols in

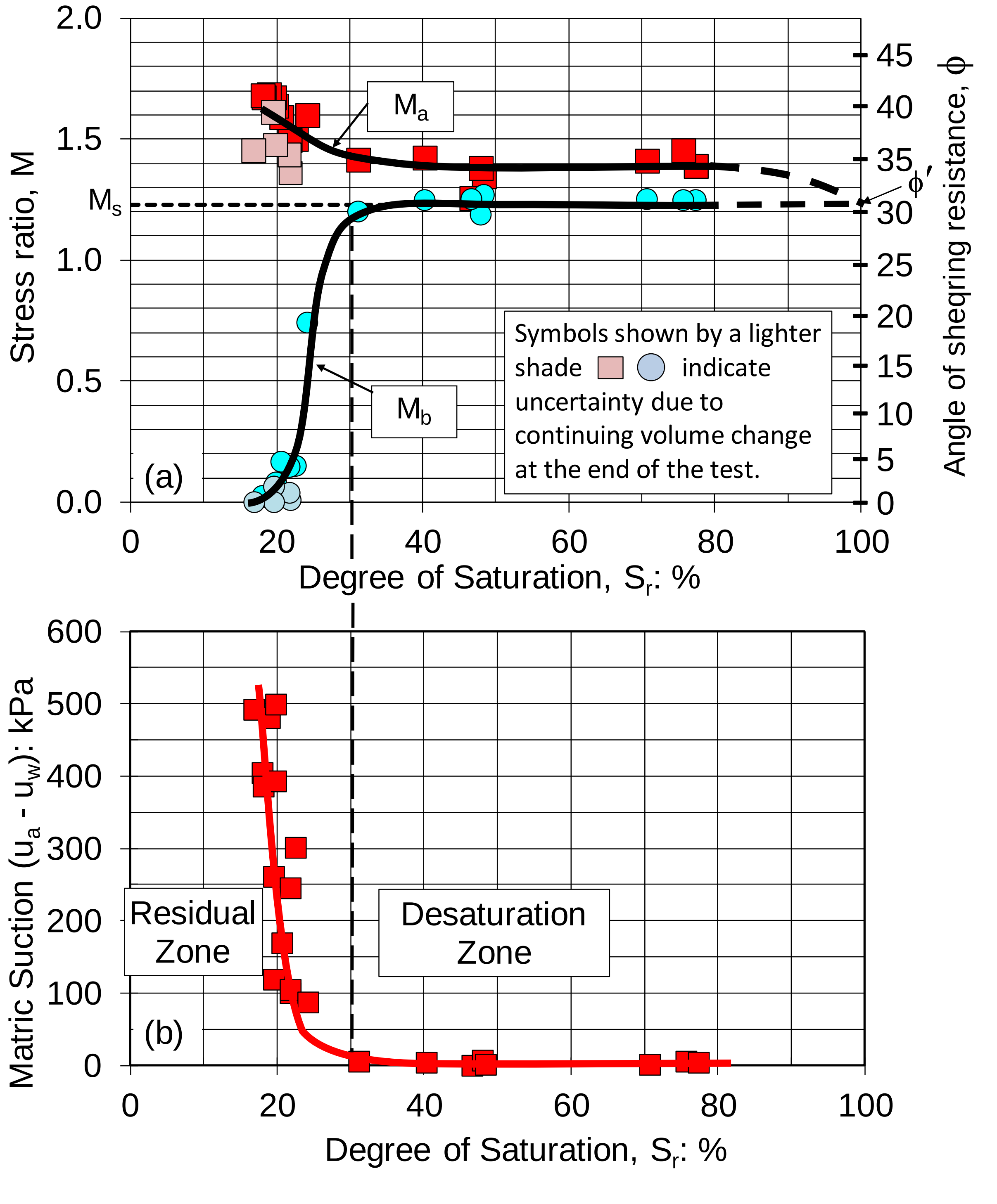


Figure 7(a) and

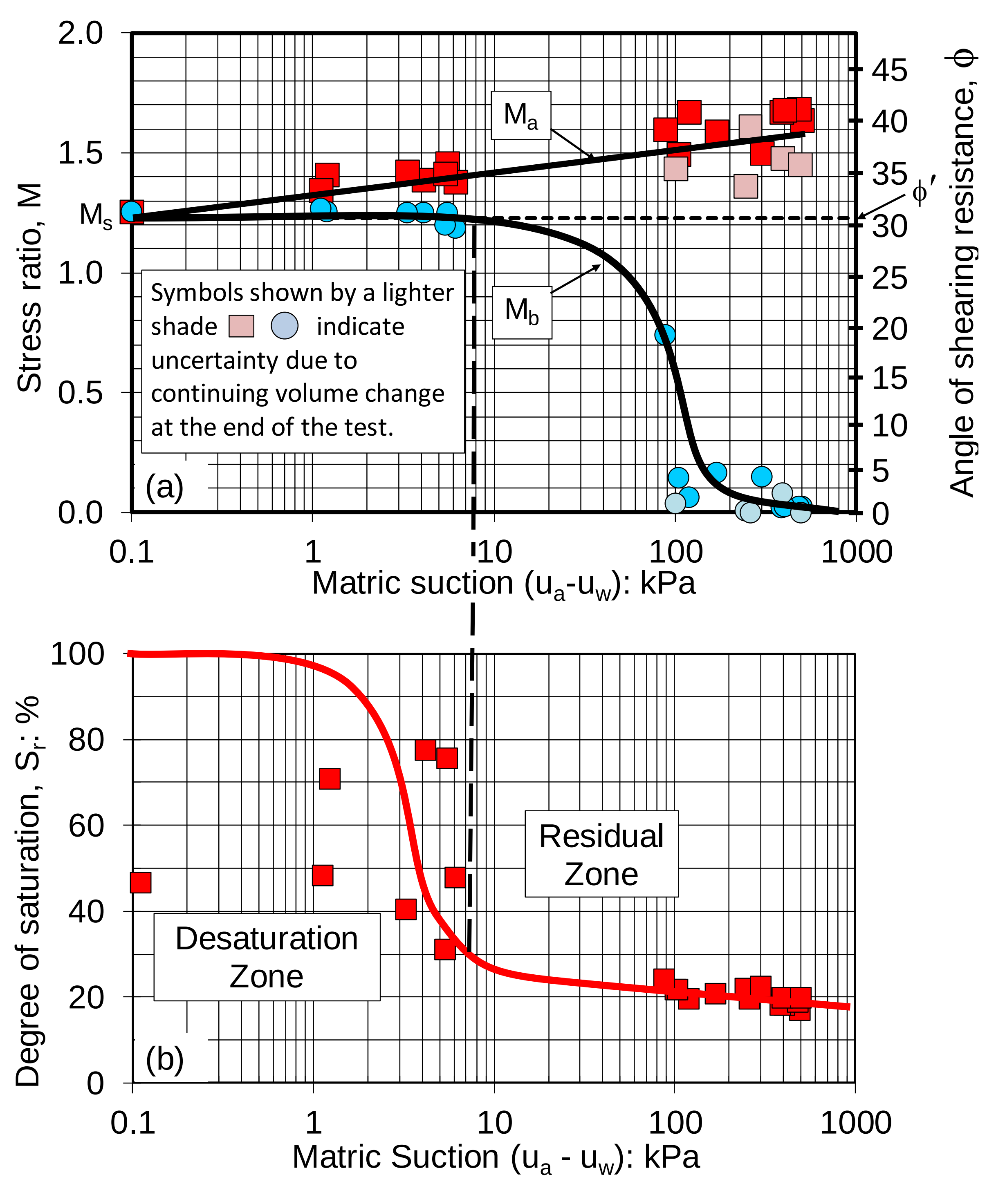


Figure 8(a). It can be seen that the values for both M_a and M_b tend to fall on the lower side, compared to values from tests where a true Critical State was achieved during the test. However, the values are not hugely out of line with other values.

The first observation from

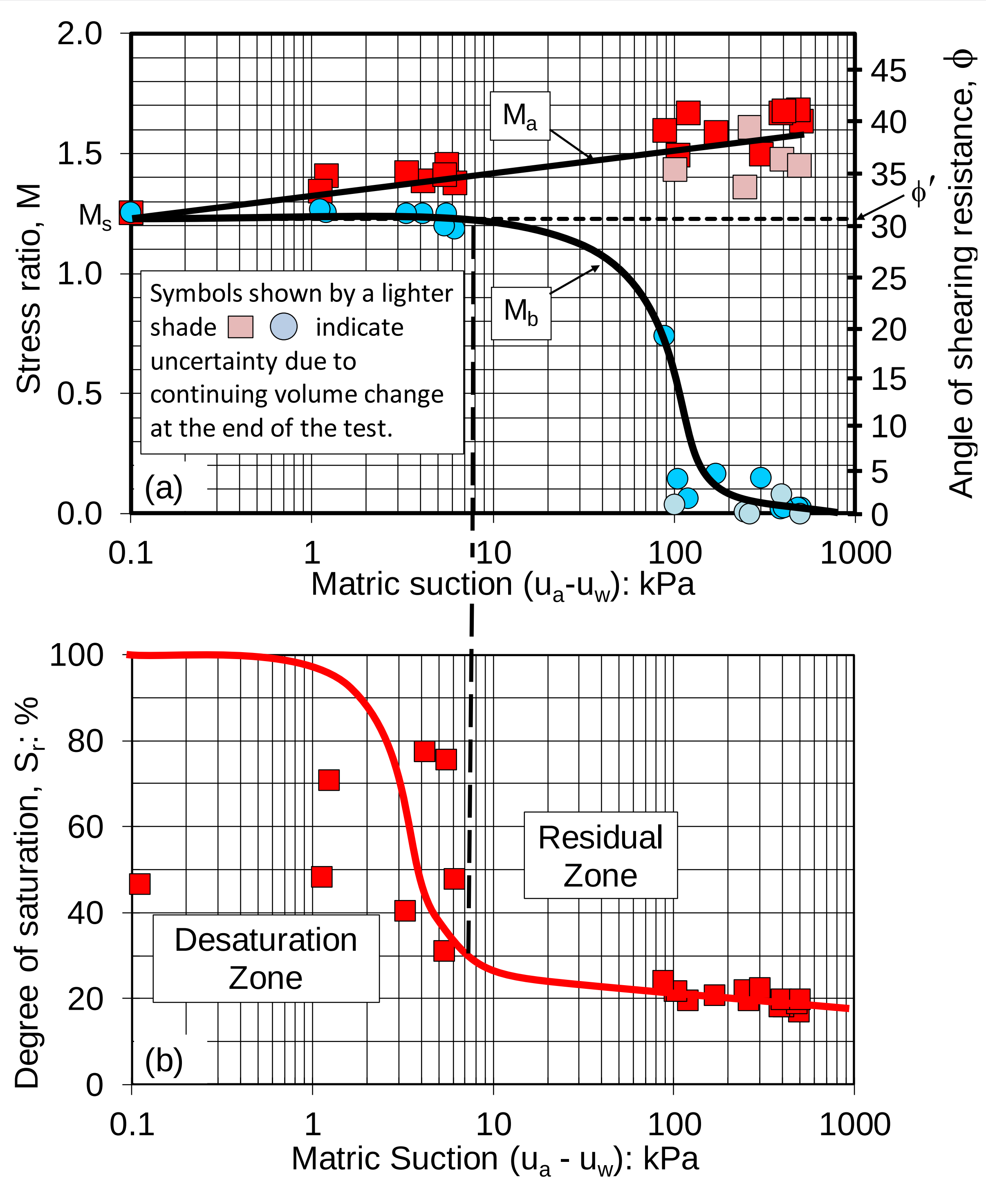


Figure 8(a) is that M_a = M_s at zero suction, although there is only one data point to corroborate this. However, as the suction increases, M_a rises to values greater than M_s. For this soil it would seem that suction has the effect of holding together groups of particles (through the formation of menisci) to produce a material that has a greater shearing resistance from net stress than a saturated material. This is consistent with the argument made by Toll (1990) and Toll and Ong (2003) that the aggregated fabric of a clayey soil could be supported by suction, effectively responding as if it were a “coarser” material, as finer particles are held together to behave as larger particles. It is interesting to see that this can also occur in a cemented sandy soil.

It can be seen from

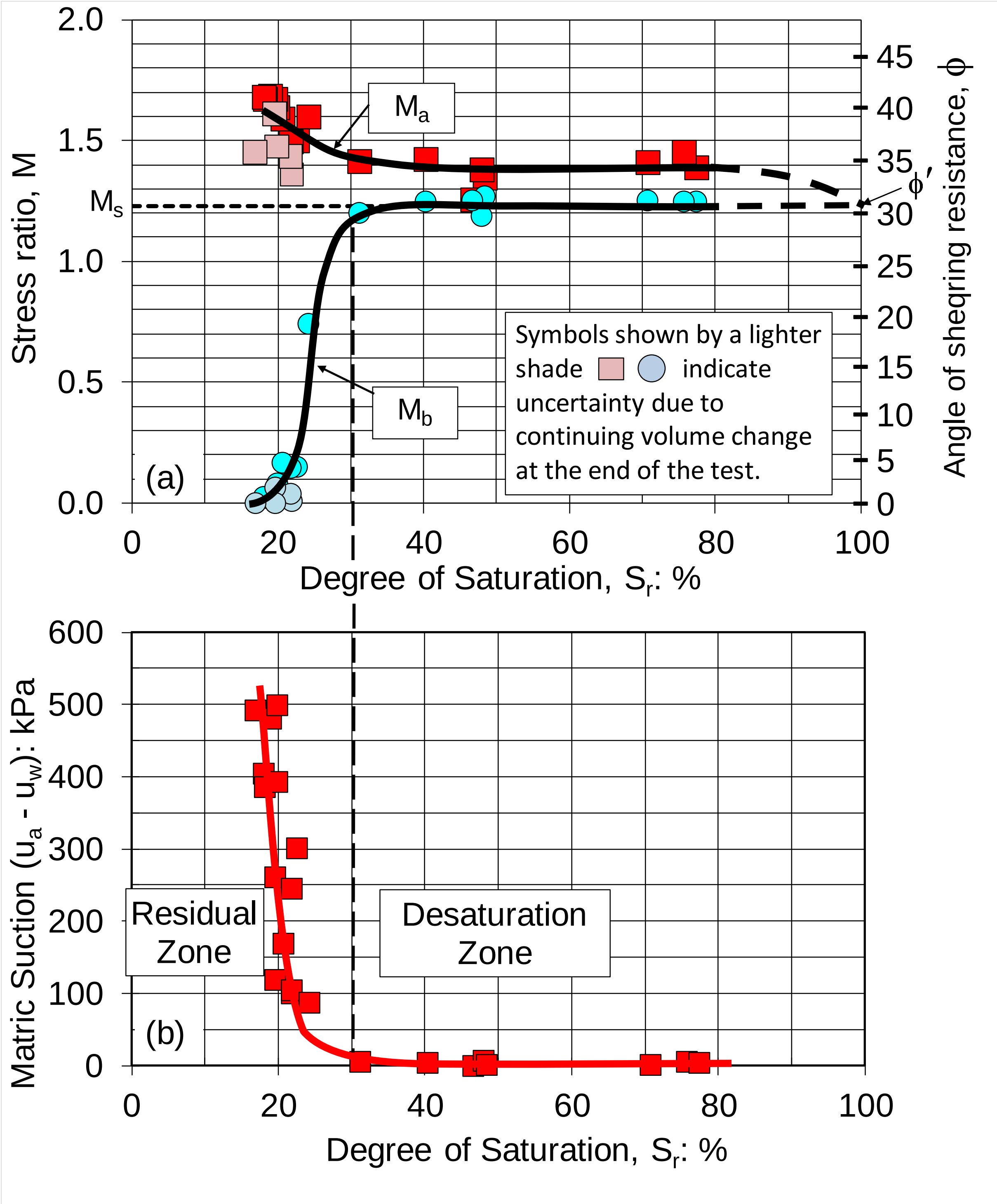


Figure 7(a) that M_b drops sharply at degrees of saturation below 30%. For comparison the soil water retention curve at Critical State conditions is plotted with S_r on the horizontal axis in

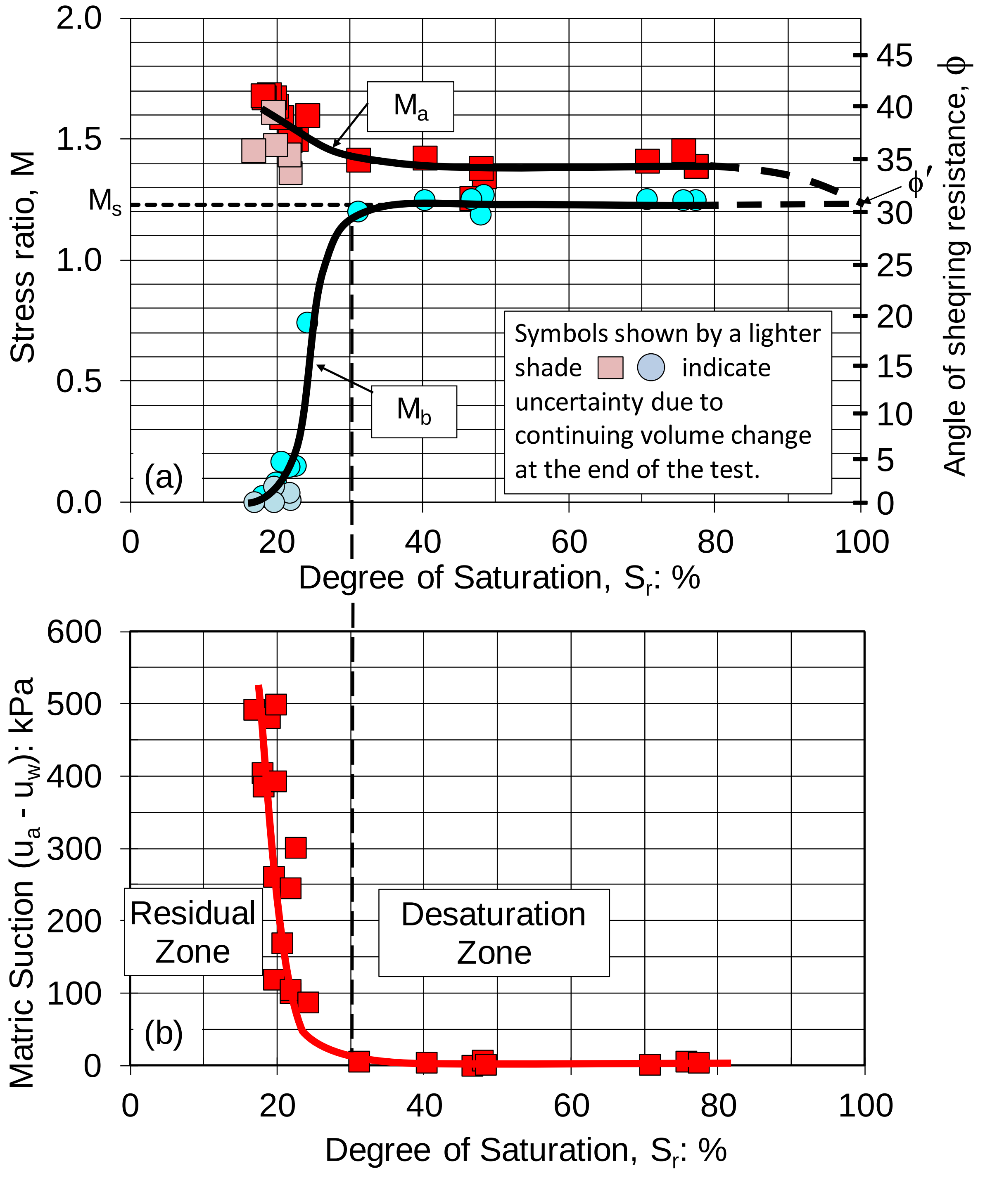


Figure 7(b). The drop in M_b coincides with a change in the water retention behaviour (the end of the desaturation zone). However, it is interesting to note that the value of M_b does not drop when the degree of saturation is changing most significantly, within the desaturation zone, as might be expected. The results suggest that the contribution of suction to strength changes markedly only within the residual zone.

The changes in M_a and M_b with suction (as shown in

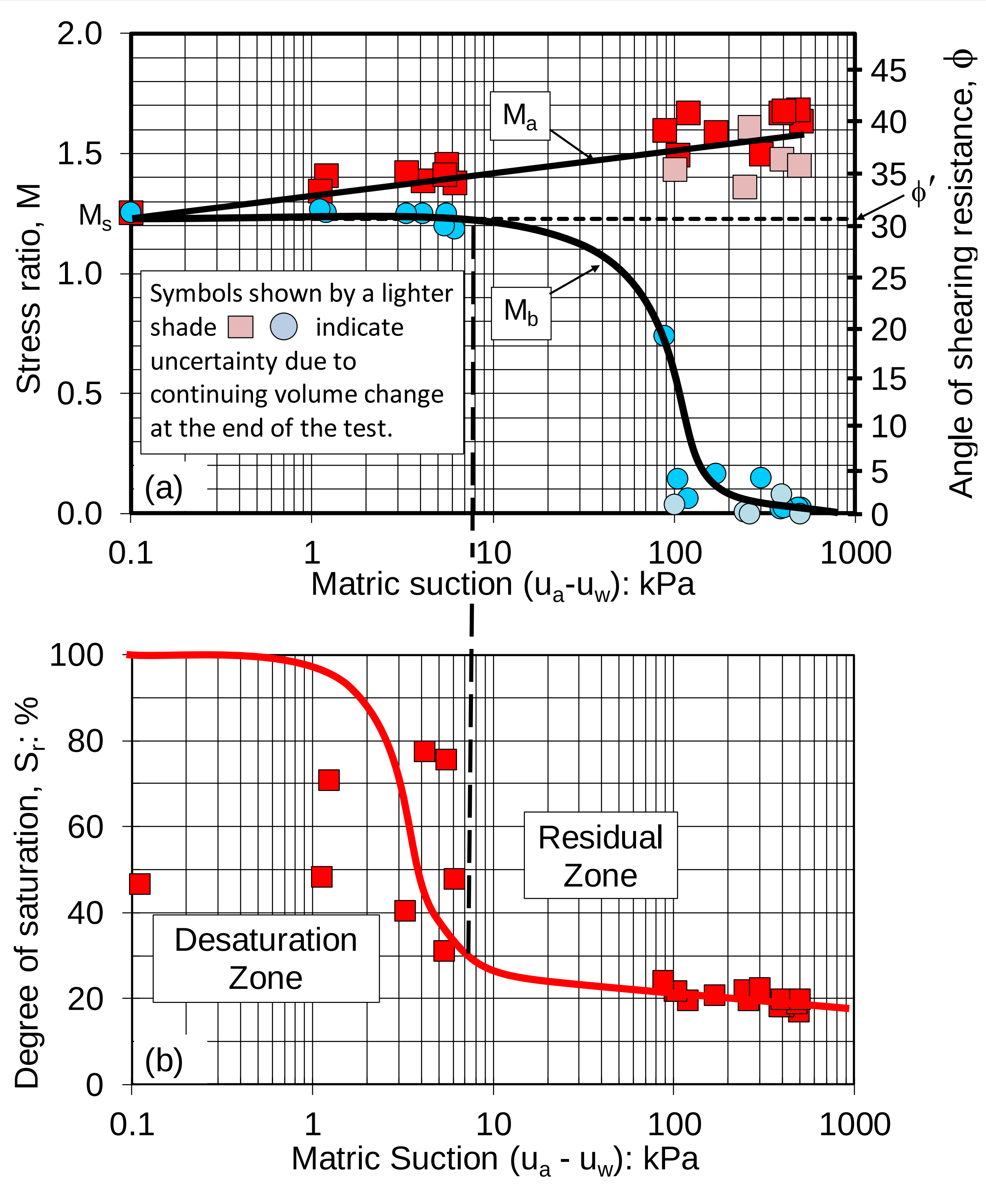


Figure 8(a)) can be related to the different phases of water retention behaviour in

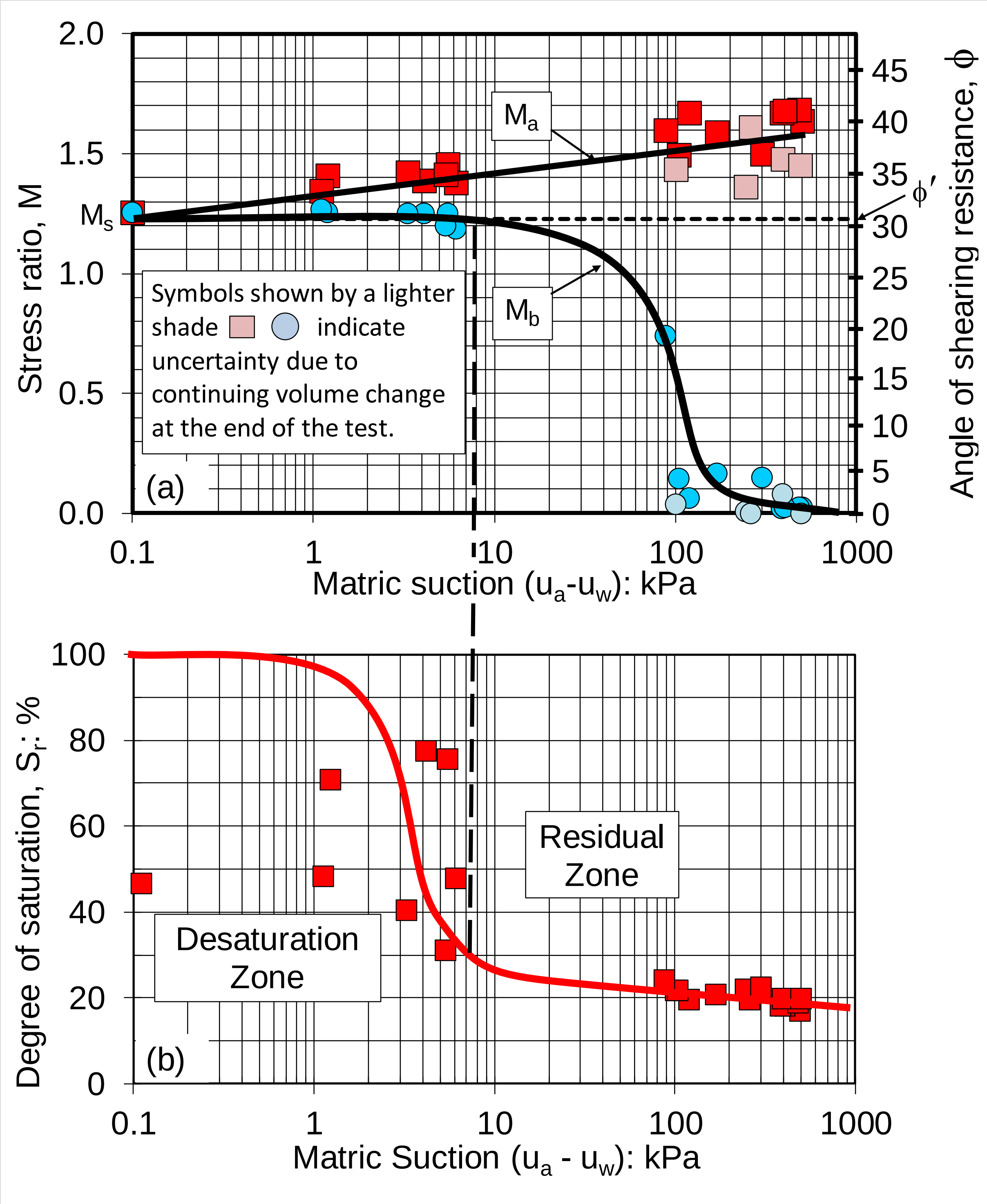


Figure 8(b). The regions of behaviour seem to be:

(i) before the air entry value $M_a = M_b = M_s$

(ii) in the desaturation stage M_a rises above M_s but $M_b = M_s$

(iii) in the residual stage M_a continues to rise while M_b starts to reduce.

It is particularly interesting that for this bonded material, the value of M_b seems to remain close to M_s even when the degree of saturation is significantly reducing. It might be expected that M_b would change within the desaturation zone, as suggested by data from Vanipalli et al. (1996) who showed that ϕ^b reduced in direct relation to the degree of saturation. This was also observed by Toll and Ong (2003) where M_b dropped significantly when the degree of saturation reduced below 90% (when the air phase becomes continuous rather than being present as occluded bubbles).This difference in behaviour in the cemented sand is probably due to the uniform nature of the bonded sand. The desaturation process in this material probably represents a removal of “bulk” water and the development of “meniscus” water (Karube and Kawai, 2001). In more widely graded materials, the desaturation process will be more complex (involving a wider range of pore sizes) and it might be expected that M_b would change within the desaturation zone.

An explanation for a change in M_b within the residual zone lies in the structure of the soil. In a bonded soil, the shear strength will be dominated by the strength of the cemented bonds between soil particles. In an unsaturated soil, the cemented fabric may not be broken down entirely even at large strains, as the suction can support and sustain groups of particles held together by the cementing agent. The desaturation of the macro-voids between the sand particles, at relatively low suctions (<10 kPa) is unlikely to produce a significant increase in strength. However, a continued increase in suction will start to affect the micro-voids within the cemented bond (the fired kaolin). This takes place within the residual zone. Because of the small size of the pores involved, desaturation of the micro-pores within the bonding material can take place with very little change in the global degree of saturation. The fired kaolin would gradually desaturate and the contribution of suction to its strength would become less.

This hypothesis implies that the suction is contributing to the strength of the cementing material itself, as was suggested by Alonso and Gens (1994).

For this material, it seems that M_a and M_b can be related either to degree of saturation (as implied by Bishop's χ factor or Toll's (1990) assumption) or to suction (as suggested by Khalili and Khabbaz, 1998). The consistent trends in both

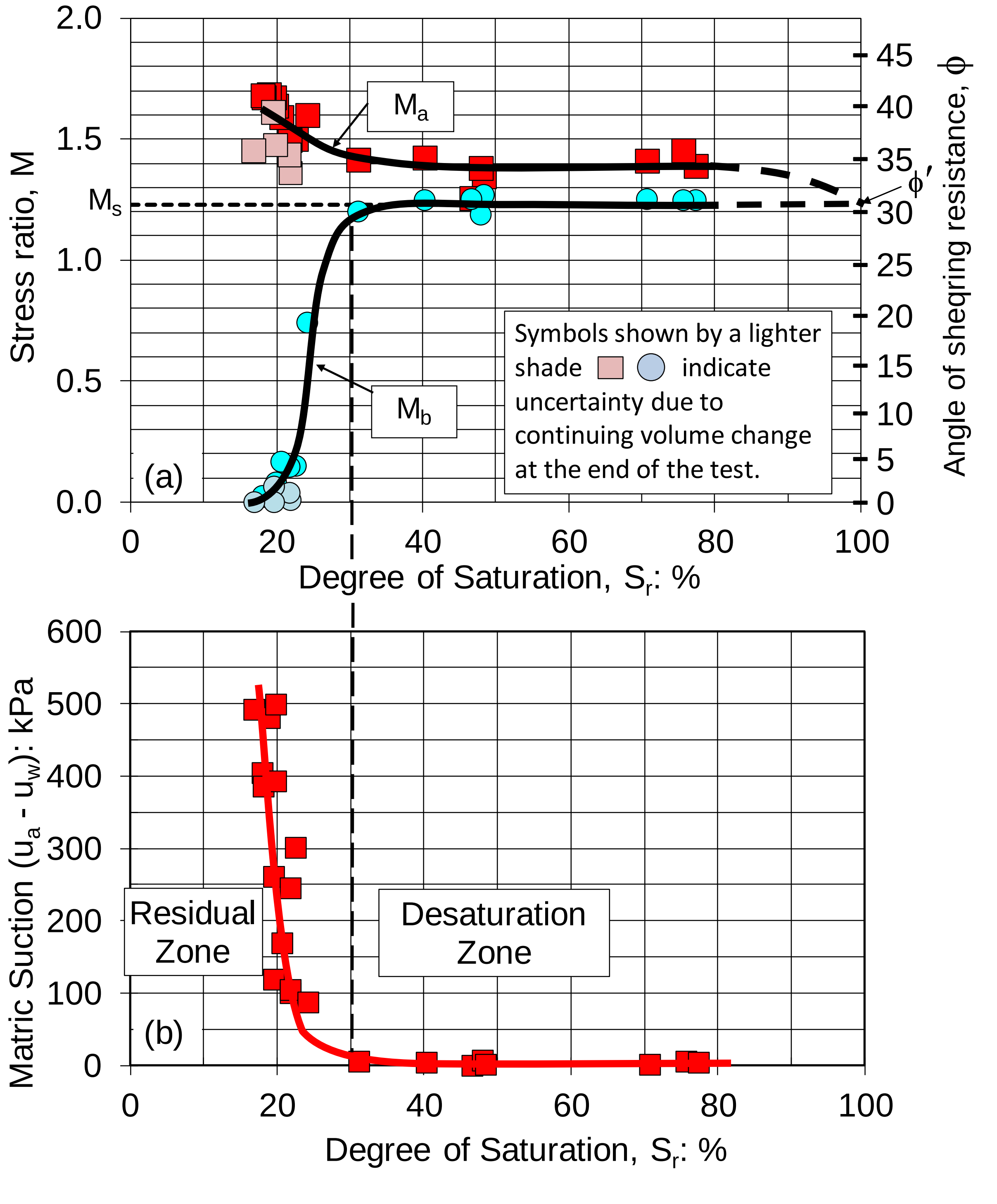


Figure 7(a) and

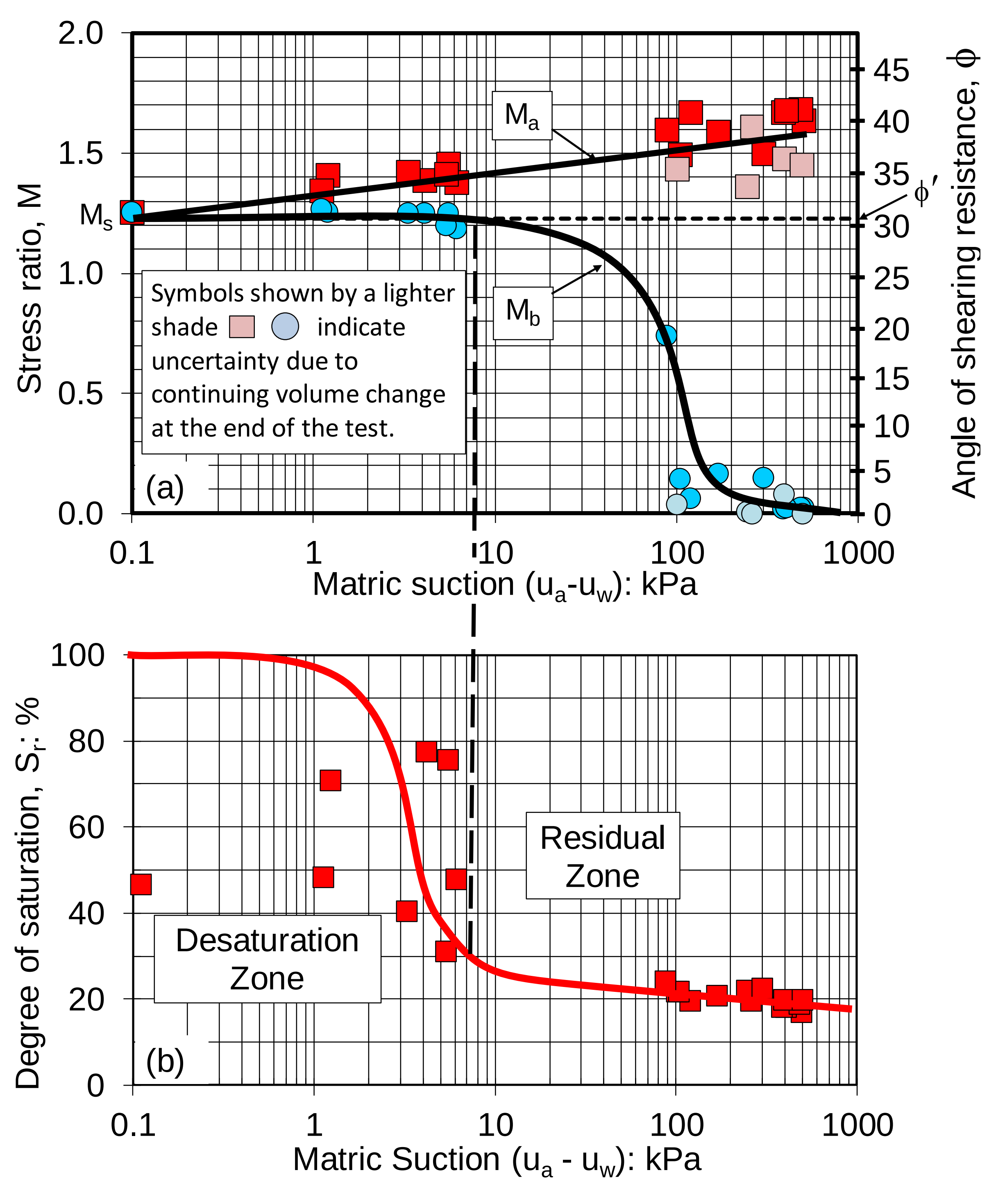


Figure 8(a) show that either variable could be used to explain the behaviour. This is likely to be due to relatively simple fabric of the cemented sand. Toll (2000) showed that for a compacted clayey soil that it was degree of saturation (as a proxy for compacted fabric) that best explained the behaviour and suction could not be used. However, for this cemented single-sized sand, the initial fabric would be the same in all samples, and hence it is the effect of suction changes that dominates the behaviour.

6 CONCLUSIONS

The results from a series of triaxial tests on unsaturated specimens of an artificially cemented sand show that to represent the Critical State of unsaturated soils an approach based on separate stress state variables is needed. This can be achieved by including a stress ratio in terms of net stress (M_a) and a stress ratio in term of suction (M_b). It was found that M_a for the bonded sand was larger than the saturated critical state stress ratio (M_s). The stress ratio in terms of suction (M_b) was found to reduce as suction increased (and degree of saturation reduced). Interestingly though, during the initial desaturation phase when the degree of saturation reduces considerably, the stress ratio M_b was largely unaffected by desaturation. It is suggested that only when the suction increased sufficiently that the micro-voids within the cementing material started to desaturate that a reduction in M_b was seen. This occurred at suctions in excess of the residual suction when the global degree of saturation was changing by very little. This implies that the suction is contributing to the strength of the cementing material itself, even at Critical State, and the change in overall shear strength is due to desaturation of the micro-voids within the cemented bonding material.

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Table 1. Critical State values of the state variables for low suction tests

q (kPa)	p-u_a (kPa)	u_a-u_w (kPa)	S_r (%)	M_a (Eq. 13)	M_a (Eq. 14)	M_a (Eq. 15)
269	190	4.1	77.4	1.42	1.39	1.39
298	200	5.5	75.7	1.50	1.45	1.46
137	96	1.2	70.7	1.44	1.42	1.42
735	545	1.1	48.4	1.35	1.34	1.34
141	97	6.1	47.9	1.46	1.37	1.38
108	86	0.1	46.6	1.25	1.25	1.25
825	577	3.3	40.3	1.43	1.42	1.42
821	575	5.4	31.1	1.43	1.41	1.42
			Average:	1.41	1.38	1.39

Table 2. Critical State values of the state variables for all tests

q (kPa)	p-u_a (kPa)	u_a-u_w (kPa)	S_r (%)	M_a	M_b
285	196	492.0	16.9	1.45	0.00
213	121	403.6	18.1	1.68	0.02
210	121	385.9	18.2	1.67	0.02
217	122	479.7	18.9	1.69	0.02
205	118	119.0	19.6	1.67	0.06
348	216	260.5	19.6	1.61	0.00
355	219	392.1	19.9	1.48	0.08
211	121	498.7	19.9	1.64	0.03
230	127	168.8	20.7	1.59	0.17
282	194	100.7	21.8	1.43	0.04
328	209	104.0	21.8	1.50	0.15
753	552	245.4	21.9	1.36	0.01
991	631	300.4	22.6	1.50	0.15
313	155	87.7	24.2	1.60	0.74
821	575	5.4	31.1	1.42	1.20
825	577	3.3	40.3	1.42	1.25
108	86	0.1	46.6	1.25	1.25
141	97	6.1	47.9	1.38	1.19
735	545	1.1	48.4	1.35	1.27
137	96	1.2	70.7	1.41	1.25
298	200	5.5	75.7	1.46	1.25
269	190	4.1	77.4	1.39	1.25

Figures

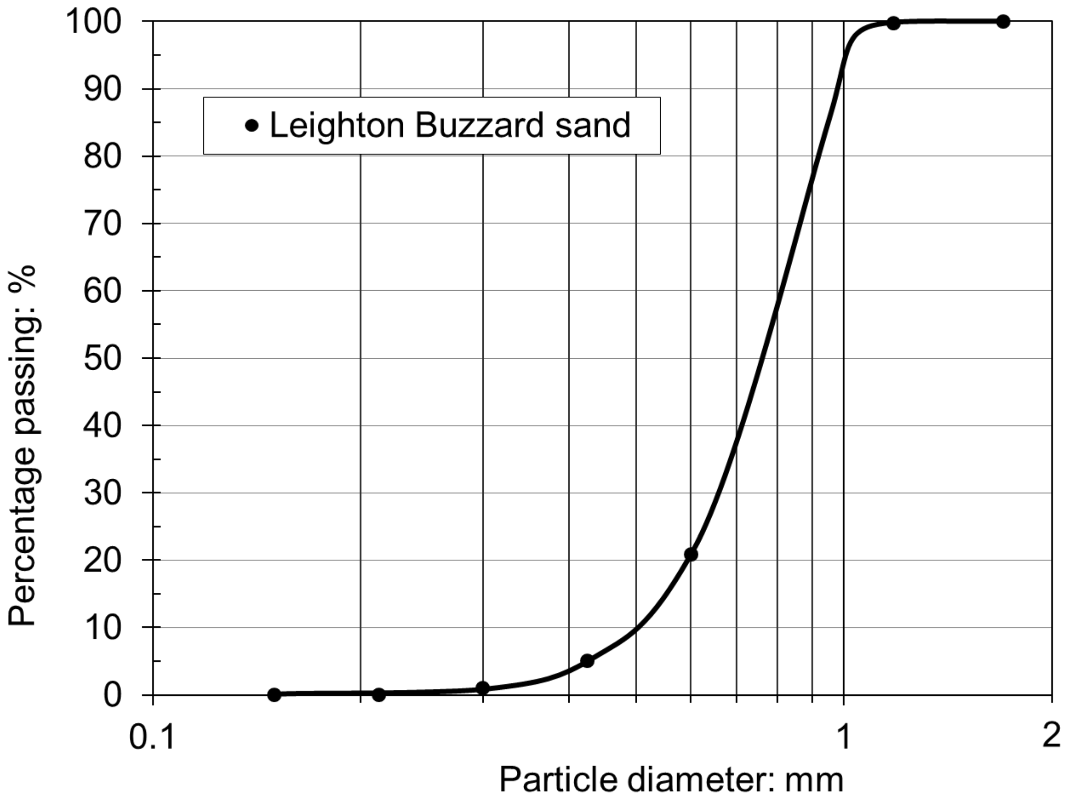
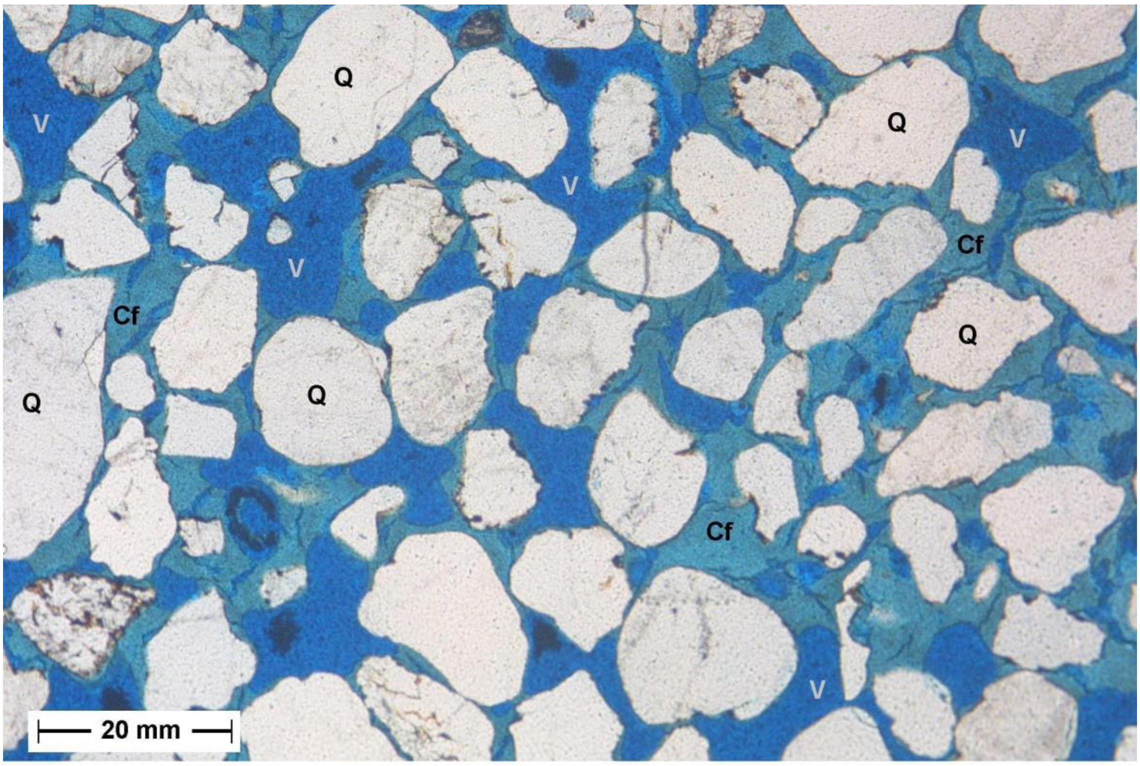


Figure 1. Particle size distribution for the Leighton Buzzard sand used to make the bonded soil



Notes:
Q - quartz Cf - fired kaolin V - void

Figure 2. Microscopic photographs of artificially cemented sand (e= 0.6, magnification × 2.5)

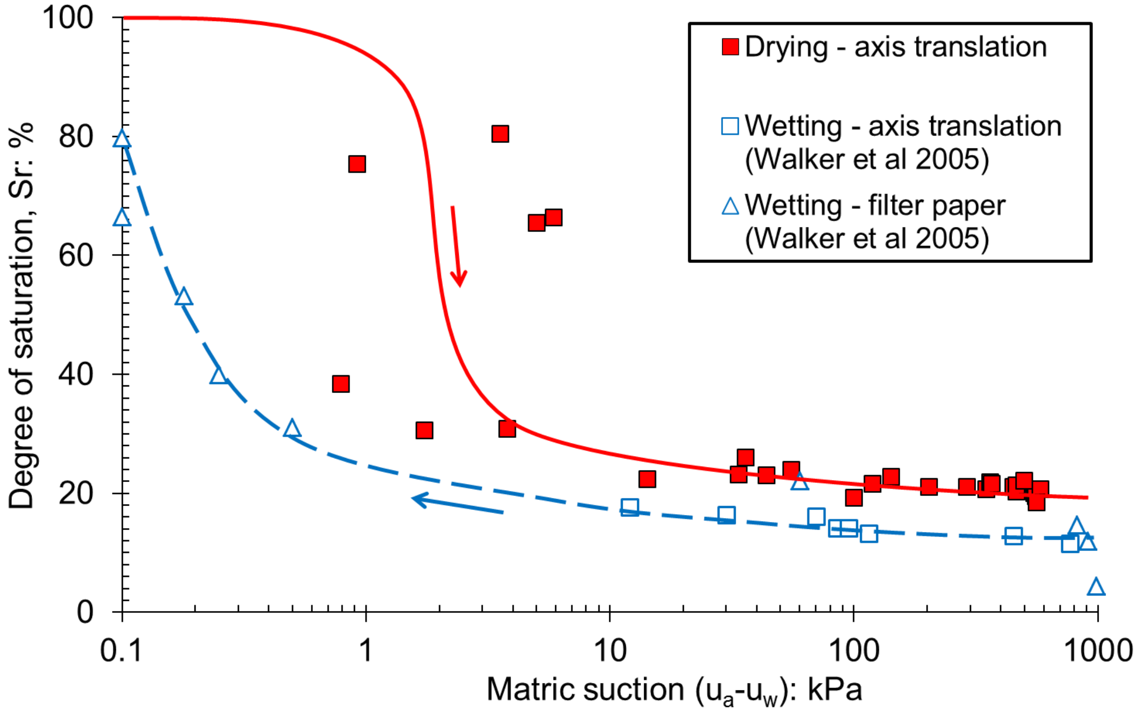


Figure 3. Water retention curve for the cemented sand. Wetting curve data from Walker et al. (2005).

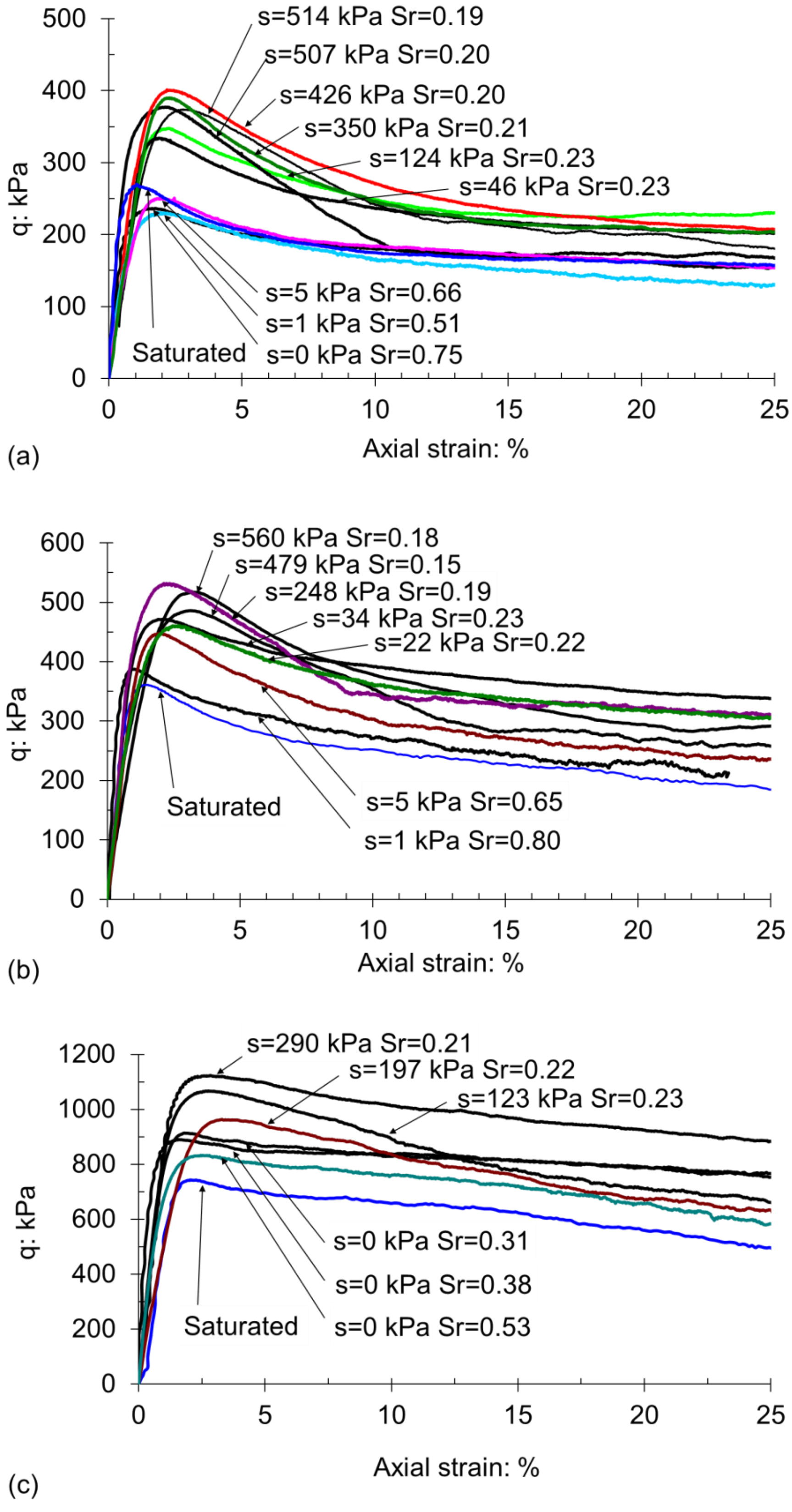


Figure 4. Deviator stress vs. axial strain curves for constant water content tests at net stress of (a) 50 kPa, (b) 100 kPa, (c) 300 kPa.

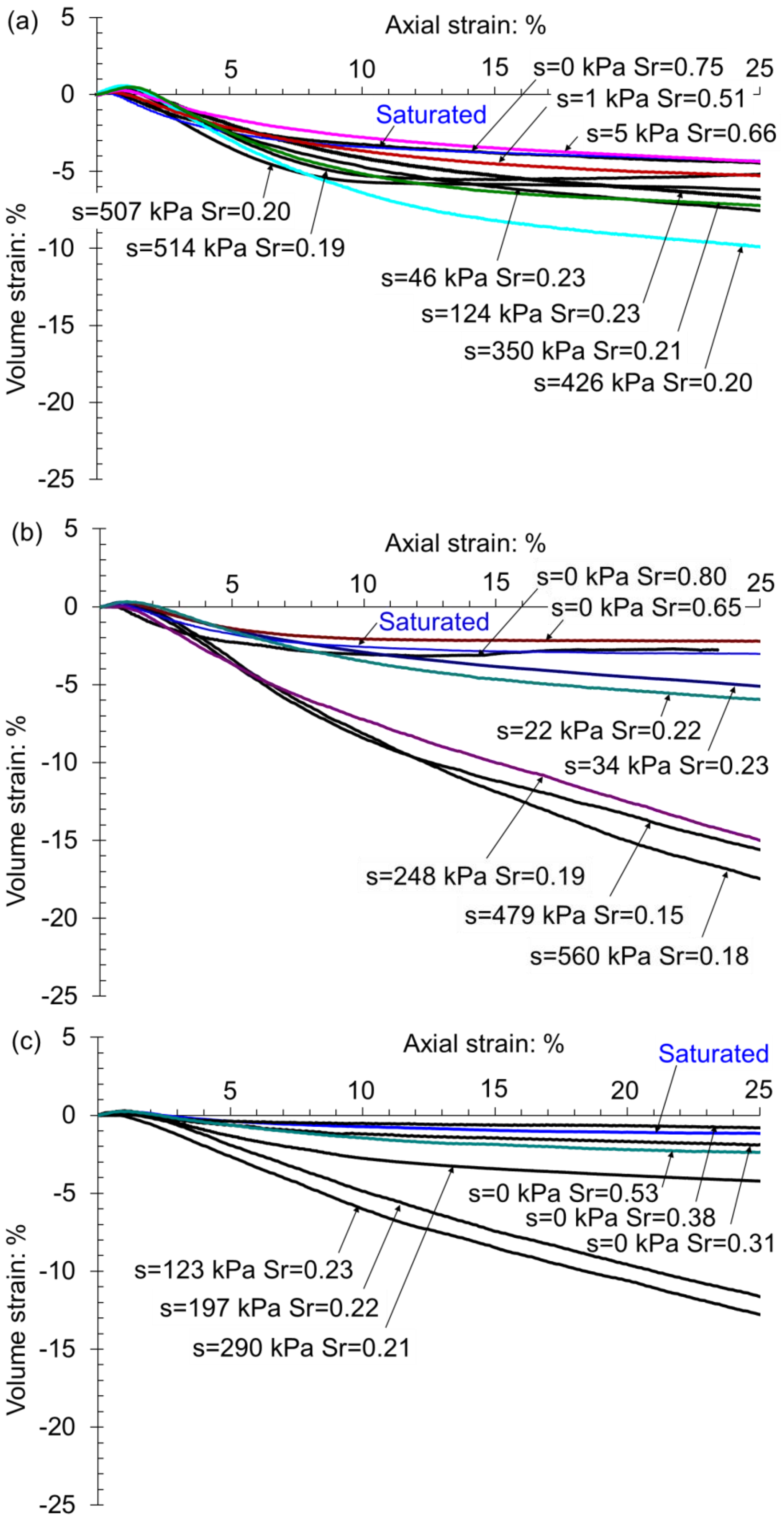


Figure 5. Volume strain vs. axial strain curves for constant water content tests at net stress of (a) 50 kPa, (b) 100 kPa, (c) 300 kPa.

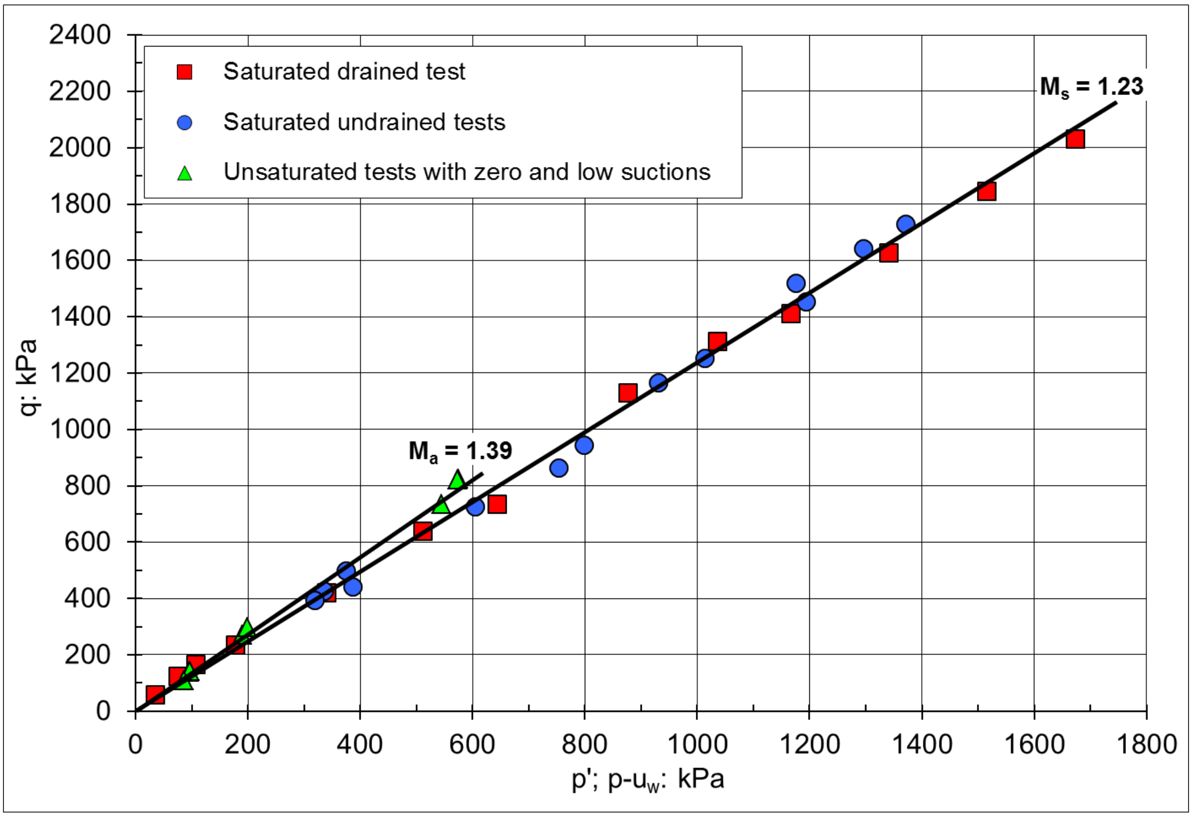


Figure 6. Comparison of Critical State stress ratios for saturated and unsaturated tests

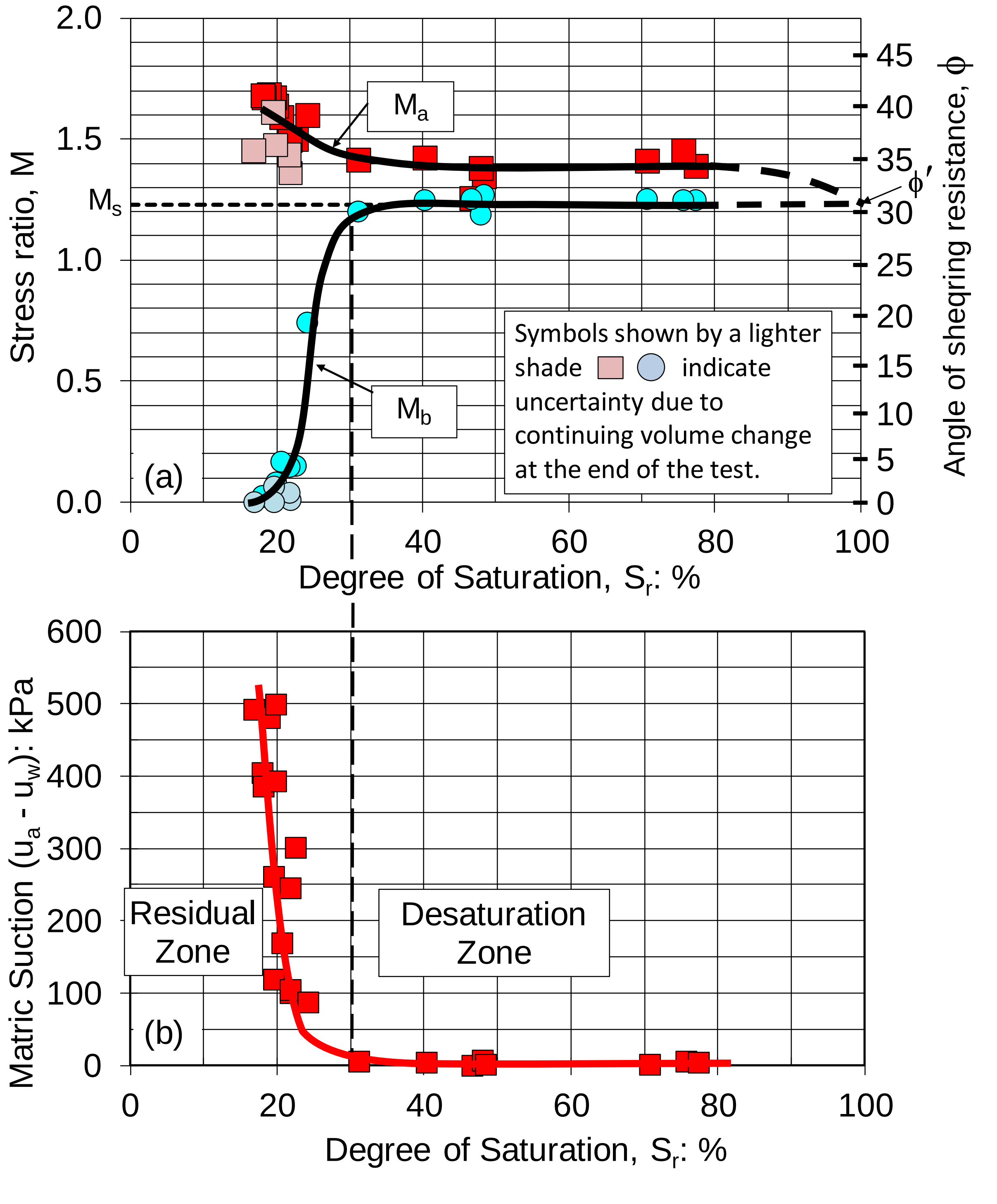


Figure 7. (a) Variation in Critical State stress ratios with degree of saturation (b) Suction vs degree of saturation at Critical State

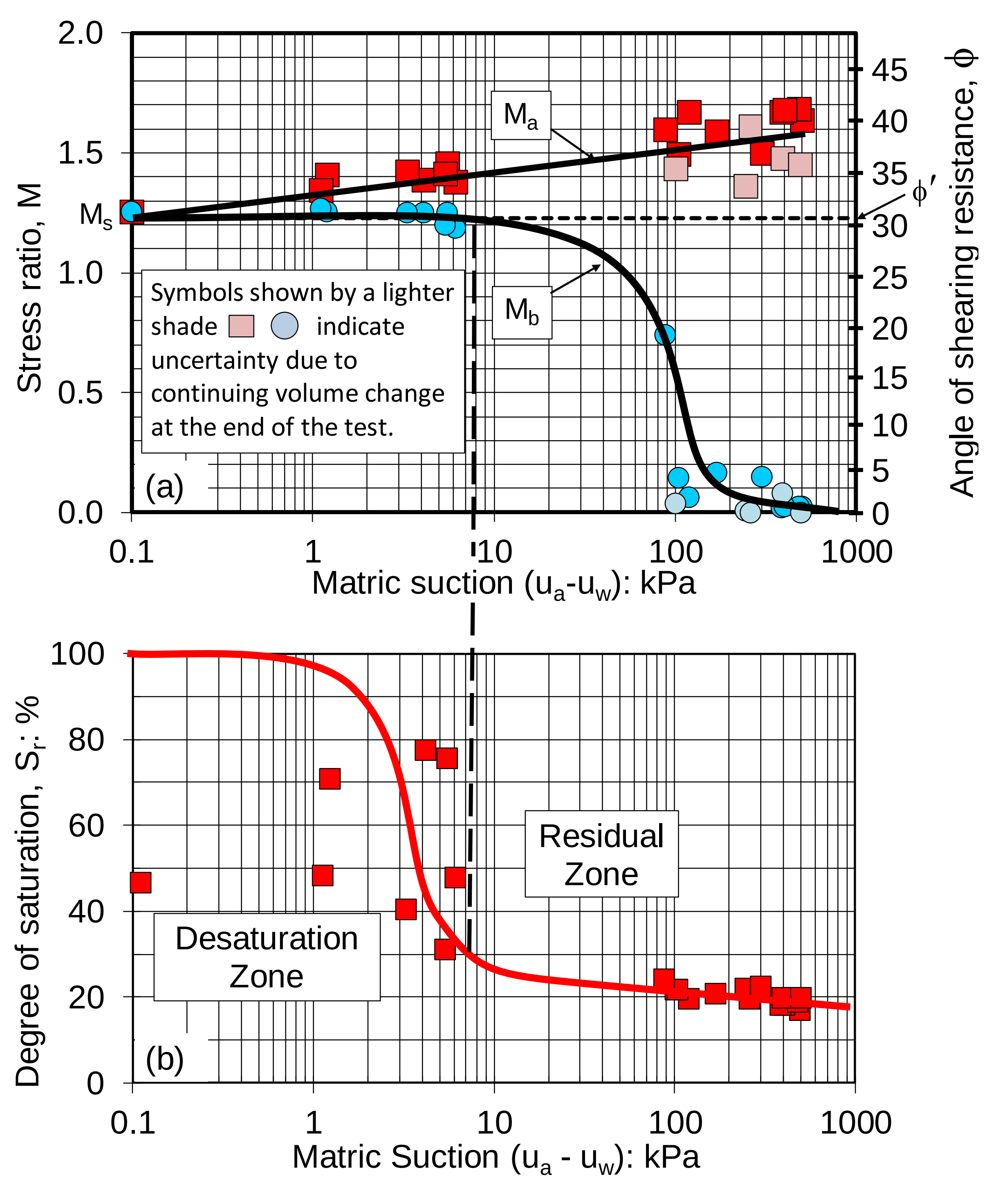


Figure 8. (a) Variation in Critical State stress ratios with suction (b) Degree of saturation vs suction at Critical State