



<b>Title</b>	Assessment of Skempton's pore water pressure parameters B and A using a high-capacity tensiometer
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<b>Publication date</b>	2019-10-14
<b>Publication information</b>	Sanlon, Philip, Vinayagamoothy Sivakumar, Brian Solan, Shane Donohue, and et al. "Assessment of Skempton's Pore Water Pressure Parameters B and A Using a High-Capacity Tensiometer." ICE Publishing, October 14, 2019. <a href="https://doi.org/10.1680/jgeot.18.p.191">https://doi.org/10.1680/jgeot.18.p.191</a> .
<b>Publisher</b>	ICE Publishing
<b>Item record/more information</b>	<a href="http://hdl.handle.net/10197/11293">http://hdl.handle.net/10197/11293</a>
<b>Publisher's version (DOI)</b>	<a href="https://doi.org/10.1680/jgeot.18.p.191">10.1680/jgeot.18.p.191</a>

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# Accepted manuscript doi: 10.1680/jgeot.18.p.191

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**Submitted:** 31 July 2018

**Published online in ‘accepted manuscript’ format:** 10 October 2019

**Manuscript title:** Assessment of Skempton’s pore water pressure parameters  $B$  and  $A$  using a high-capacity tensiometer

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## **Abstract**

Saturation of soils is a prerequisite in many laboratory tests involving consolidation, permeability and stress-strain behaviour. The saturation process is usually time consuming, particularly in clay-rich soils, and this can incur substantial cost and potential delays in reporting findings. The saturation of samples is assessed using the well-established Skempton's pore water pressure parameter  $B$ . In a situation where the soil is fully saturated the  $B$ -value is approximately one. It is often the case that fine soil samples extracted from the ground, particularly those from below the water table, remain saturated. However, current testing protocols require evidence to verify a complete saturation prior to subsequent laboratory investigations. This paper reports experimental results exploring the hypothesis that, if the sample is 'perceived' to be saturated, then further saturation procedures may not be necessary to obtain reliable geotechnical parameters. Laboratory investigations were conducted on three different clays (Kaolin Clay, Belfast Clay and Oxford Clay) in a testing chamber instrumented with a high capacity tensiometer. The confining pressures were applied in a ramped fashion under undrained conditions. The response of the tensiometer confirmed that the samples were saturated from the very beginning of the loading process, as implied by the  $B$ -value being close to one. Further supplementary investigations were carried out to assess the Skempton's pore water pressure parameter  $A$  and the stress-strain behaviour of the soils. The combined finding provides further evidence to suggest that the saturation process as suggested in standards may not be necessary for fine grained soils to establish reliable geotechnical design parameters.

**Keywords:** Clays; Laboratory tests; Mineralogy; Pore pressures; Sampling; Suction

## INTRODUCTION

The extraction of soil samples from the ground under “perfect conditions” (Class 4 sampling) generates negative pore water pressure (Carrubba, 2000; Hight *et al.*, 2003; Donohue *et al.*, 2009; Donohue *et al.*, 2010; Delage *et al.*, 2016). Whether or not the negative pore water pressure makes air get into the voids and causes the sample to become de-saturated (i.e. the degree of saturation  $S_r < 1$ ) depends on various factors: material type (particle size distribution), stress history, clay mineralogy and soil density (Young *et al.*, 1983; Graham *et al.*, 1988; Doran *et al.*, 2000; Long, 2003; Sivakumar *et al.*, 2009). Lynch *et al.* (2019) suggested that, for clay-rich soils such as kaolin, the suction (i.e. the negative pore water pressure) required to initiate the desaturation process is greater than 1000kPa, but for other clay-rich soils, for example Belfast Clay and Oxford Clay, this value may be much higher due to their mineralogical compositions. However, in many cases the suction in a sample recovered from moderate depths (say up to 50m below the ground level) is much less than the suction required for the air to enter the void spaces. In such cases, although the soil possesses suction, the sample remains saturated. Nevertheless, the current testing standards provide procedures to ensure full saturation of soil samples. The main limitation of the existing procedure is that it does not have provision to measure the negative pore water pressure in the sample before applying any external stresses.

Skempton (1954) proposed a hypothesis for the development of pore water pressure upon external loading using two parameters, namely  $B$  and  $A$ . Since then the hypothesis has been widely accepted among geotechnical engineering practitioners and academics (Clayton *et al.*, 1995; BSEN ISO17892-9, 2018). The excess pore water pressure in soils during changes in stress regimes can be determined, for axial symmetry stress conditions, using the following equation involving the pore water pressure parameters  $B$  and  $A$  (Skempton, 1954).

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)] \quad (\text{Eq 1})$$

where  $\Delta u$  is the change in pore water pressure and  $\Delta\sigma_1$  and  $\Delta\sigma_3$  are the changes in major and minor principal stresses respectively. There are two aspects that need attention:

- (a) the  $B$ -value which symbolises the degree of saturation of the sample and
- (b) the  $A$ -value which symbolises the changes in pore water pressure under shear loading.

**B-value:** The primary purpose of this investigation was to assess the  $B$ -value of clay-rich soils subjected to the removal of external loading under undrained conditions (symbolising the extraction of samples from the ground under perfect sampling conditions). The removal of overburden stresses during sampling generates negative pore water pressure within the soil. If that negative pore water pressure is sufficient to allow air to enter into the pore spaces (for example in the case of silt and sand), then the saturation process is essential and that can only be confirmed by assessing the  $B$ -value. In clay-rich soils such negative pore water pressure should be extremely high for the air to get into the void spaces, but this is not usually the case. This paper therefore reports a vast amount of data to confirm that the samples of clay-rich soils remain saturated even when high external stresses are removed.

**A-value:** If the material is linearly elastic, isotropic and subjected to undrained loading or unloading,  $A = 1/3$ . In normally consolidated soils, the  $A$ -value varies non-linearly with the shear strain. Skempton *et al.* (1963) applied Equation 1 to understand the impact of stress changes during sampling which was considered to be an elastic process. They pointed out that, in general, there is no reason to suppose that  $A = 1/3$  in natural soils which are often anisotropic. Doran *et al.* (2000) revised the Skempton's pore water pressure parameter  $A_s$ , by taking account of anisotropy and derived the following relationship:

$$\Delta u = B[\Delta\sigma_3 + (A - J/3G^*)(\Delta\sigma_1 - \Delta\sigma_3)] \text{ where } A_s = 1/3 - J/3G^* \quad (\text{Eq 2})$$

where  $J$  is the coupling parameter and  $G^*$  is the modified shear modulus (Graham *et al.*, 1983). For most natural soils the value of  $J/3G^*$  (which is the slope of the undrained stress path in  $p'$ - $q$  plane at low strain, Doran *et al.*, 2000) is negative ( $p'$  = mean effective stress and  $q$  = deviator stress). This makes the overall pore water pressure parameter  $A$  larger than  $1/3$ . This aspect was also investigated in this paper.

## EXPERIMENTAL WORK

Three different soils were tested namely: Kaolin, Belfast and Oxford Clays. All these three soils are clay-rich materials and the relevant physical properties are listed in Table 1.

### Sampling Procedure

Four different methods were adopted to generate samples.

**Sampling Method A:** Dry Kaolin Clay was mixed with de-aired water to achieve a water content of  $1.5 \times$  liquid limit using a Cope 2000 mixer. This was consolidated one-dimensionally to a vertical pressure of 600kPa or 800kPa depending upon the testing requirements (Table 2) in a chamber (100mm diameter and 300mm height). The consolidation of the slurry lasted 3 days. Upon completion, the external loading was removed and the sample was extruded and trimmed to required height for further testing.

**Sampling Method B:** The correct assessment of the  $B$ -value requires the testing of materials that have known initial suction values to confirm the reliability of the high capacity tensiometer used in this study. In Sampling Method A, although the samples were subjected to known vertical stresses, the stress conditions within the samples at the end of the consolidation were not isotropic. In addition, the side friction between the consolidation chamber and the clay impacted the distribution of vertical stresses along the sample. It would, therefore, not be possible to judge the expected suction in the sample upon removal of the consolidation pressure. Therefore, the following procedure was adopted for preparing samples with known suctions upon removal of consolidation pressures.

A 150mm diameter membrane was sealed against the pedestal of a triaxial cell. A slurry of Kaolin or Belfast Clay was then carefully filled into the membrane to a height of 300mm. In order to avoid bulging of the membrane under the weight of the slurry, the membrane was supported with a thin wire mesh. The top of the membrane was then sealed against a top cap. The triaxial cell was assembled and filled with water and the slurry was subjected to the required confining pressures (isotropic stress conditions) listed in Table 2. Drainage of the water was allowed from the base. The consolidation of the samples lasted ~4 days for the Kaolin Clay and 21 days for the Belfast Clay. Following the consolidation process a small amount of vacuum was applied on the drainage line to remove the water in the porous disc placed at the bottom of the sample. This procedure was carried out to ensure that the sample would not have access to water upon removal of the cell pressure. The sample was then removed from the system and a thin-wall sample tube with an internal diameter of 100mm

was used to extract a cylindrical sample. Some of the samples have had further treatment under Sampling Method C.

**Sampling Method C.** It was anticipated that the suction in the sample prepared using Sampling Method B would be equivalent to the magnitude of the applied external confining pressure in the triaxial cell. However, a concern was that the subsequent extraction of samples using a thin-walled sample tube could have generated sample disturbances (Clayton *et al.*, 1995; Lunne *et al.*, 1997; Donohue *et al.*, 2009) which might lead to reduced suction and stiffness. Therefore, as a modification to Sampling Method B, two Kaolin Clay samples (Tests 9 and 10 in Table 2) and a sample of Belfast Clay (Test 13) were initially prepared using Sampling Method B. The extracted samples of 100mm diameter and known height were then recompressed to the same consolidation pressures (under isotropic stress conditions) applied during initial formation in a standard triaxial cell, with a 100mm diameter pedestal. The intention here was to remove the excess pore water pressure that may have been generated during the sampling process. Once again, before removing the consolidation pressure, the porous disc located at the base of the sample was dried by applying a small amount of vacuum. No further trimming was carried out as the sample changed its volume only marginally.

**Sampling Method D:** Natural samples of Oxford and Belfast Clays were delivered to the laboratory in sampling tubes. They were extruded from the tubes (103mm in diameter core) and carefully trimmed to 100mm diameter using a standard soil lathe.

### **Testing Chamber**

Figure 1 shows the testing chamber used in the research. The chamber contained a high capacity tensiometer, located at the pedestal, which was capable of measuring pore water pressure of -1500 kPa (Lynch *et al.*, 2019). More details on the tensiometer can be found in Lourenco *et al.* (2007). No drainage provisions were provided either in the pedestal or in the top cap as the investigations required loading under undrained conditions.

The tensiometer required a careful saturation procedure. Initially the chamber was filled with de-aired water and pressurised to 1000 kPa for several days. The tensiometer was calibrated in the positive pressure range, assuming the calibration parameters remain the same in the

negative pressure range (Ridley *et al.*, 2003; Take *et al.*, 2003; Lourenco *et al.*, 2007). The tensiometer was calibrated before each test as suggested by Lourenco (2008). Figure 2 shows the calibration factors (intercept and the slope) of the tensiometer for several calibration events and it appears the slope remains generally unchanged during the course of the investigation.

Although tests were conducted under constant water mass conditions, it is possible that the external loading may have resulted in a minor volume change of the sample. A flow of water into the annulus between the sample and chamber would therefore give an indication of a sample volume change. The chamber was calibrated for the apparent volume change (i.e. without the sample in the chamber) by pressurising it to 1500kPa, at a rate of 1kPa/minute and the volume of water flowing into the chamber was measured using the volume change unit shown in Figure 1.

### **Testing Procedure**

In total 16 tests were carried out as listed in Table 2. Upon completion of the sampling (Methods A-D), the samples were trimmed to 140mm height (the maximum height that the testing chamber can accommodate). In a commercial situation the height of the sample should be at least 160mm to meet the requirement of height to diameter ratio of at least 1.6; this is only required if the samples are taken to the shearing stage. Alteration of the height to diameter ratio below 1.6 can influence the performance of soils particularly at a large strain, however given that the study focused on the relative stress-strain behaviour of soils (as a part of the *A*-value assessment), this size was deemed acceptable for the current research.

Samples were assembled in the testing chamber and sealed with a rubber membrane. The chamber was filled with de-aired water and pressurised to a nominal cell pressure of 50kPa and allowed to stabilise for 24 hours. Figure 3 shows the response of the tensiometer. The response time was slightly longer in the case of Belfast and Oxford Clays. Both of these clays have considerably lower permeability than Kaolin Clay. On the following day, the cell pressure was increased to 800kPa at a rate of 0.8kPa per minute. Upon reaching the target cell pressure and allowing a further 6 hours of stabilization time (during this resting period there were no significant changes in the pore water pressure), the samples were subjected to shear

loading at a rate of 0.08kPa/minute. The test condition in this case is referred to as “positive loading” (Table 2). In another set of tests, the samples were subjected to shear loading at a cell pressure of 50kPa without ramping the cell pressure to 800 kPa. In these cases, the pore water pressure in the samples remained negative. The testing under this condition is referred to as “negative loading” (Table 2). Samples of Oxford and Belfast Clays were also tested in a standard triaxial cell following the procedure recommended in BS1377, Part 8 (1990).

## RESULTS AND DISCUSSION

### Reliability of suction measurements

To ensure the tensiometer was reading the correct suction values an evaluation was carried out using the data obtained on a Belfast Clay sample (Sampling Method C, Test Number 13). Kaolin Clay was not considered for this purpose as it tends to undergo significant stress relief upon unloading which will be further discussed later in the article. The sample was originally consolidated to 600kPa of confining pressure in a large triaxial cell (150mm pedestal) and after trimming to the required height it was reconsolidated 610kPa in a standard triaxial cell (100 mm pedestal). Upon the removal of this pressure, the expected negative pore water pressure in the sample was around -610kPa.

Figure 4 shows the changes in cell pressure ( $\sigma_3$ ) and pore water pressure ( $u$ ) in the sample plotted against time during the course of ramping the confining pressure from 50kPa to 800kPa. During this period, a 0.1% volume change took place and the pore water pressure in the sample increased from -578kPa (at a cell pressure of 50kPa) to 190kPa. The effective stress in the sample was therefore would have been approximately 628kPa as indicated in Figure 4. This value was reasonably close to the effective stress in the sample (617kPa) at the end of the application of cell pressure of 800kPa. There is only a 11kPa difference between the start and end of the calibration process, which is not significant. These observations gave confidence in the reliability of the tensiometer measurements.

### Assessment of $B$ -value

Figure 5 shows the pore water pressure, plotted against cell pressure, where the cell pressure was ramped from 50kPa to 800kPa over a period 48 hours for samples of one-dimensionally compressed Kaolin Clay (Tests 1 and 3), isotropically compressed Kaolin Clay (Tests 4, 5

and 6), isotropically compressed Belfast Clay (Test 13) and a natural sample of Oxford Clay (Test 12). The relevant values of initial negative pore water pressure at an external cell pressure of 50kPa are also included. The reader will notice some small anomalies in the response of the tensiometer in the pressure range between -100 and 100 kPa, particularly in the case of Belfast and Oxford Clays. The supplier of the tensiometer confirmed that the support systems for the diaphragm containing the strain gauges for the tensiometer were not perfectly matched on either side. This may lead to a situation where the diaphragm responded slightly differently to negative and positive pressures. Overall the tensiometer yielded very useful information for the purpose of this investigation. In all cases, the volumetric strains of the samples when the cell pressures were applied were  $< 0.15\%$ , which is insignificant.

Figure 5 allows a direct evaluation of the  $B$ -value indicated by the slopes of the lines (solid-lines) which highlights how the  $B$ -values changed with cell pressure. A comparison of the theoretical  $B$ -value lines for saturated soils (shown with dashed lines) with the tested soils confirms that all the samples tested appeared to be saturated from the beginning of the application of the confining pressures. This was indicated by  $B$ -values ( $\Delta u/\Delta\sigma_3$ ) value of  $\sim 1$  (varies between 0.95-1.0). Interestingly the Belfast Clay sample exhibited the same response even with a significant negative pore water pressure of -578kPa at the beginning of the application of confining pressure. The slopes of the solid lines seem to suggest that the samples were saturated at the time of extraction, sampling and trimming, therefore prompting a question regarding the need for saturating the samples of clay-rich soils before further testing.

Some additional pieces of information collected from this research allowed the assessment of stress relief (i.e. the expected effective stress in the sample based on the initial consolidation pressure) under unloading conditions. This aspect has been examined by many researchers (Graham *et al.*, 1988). The Kaolin Clay samples have shown significant stress relief upon unloading under undrained conditions. The samples prepared under one-dimensional consolidation during the initial preparation cannot legitimately be brought into this discussion, as the stress regime, particularly the lateral stresses, is unknown (Tests 1, 2 and 3). This could have been one of the possible reasons for significantly lower effective stresses in the samples upon unloading (Table 3). The discussion will therefore focus on the samples

prepared using isotropic compression (Sampling Methods B and C; Tests 4-10 and 13). Table 3 lists the consolidation pressures before sampling, and the effective stresses in the samples, when the specimens were subjected to 50kPa of confining pressure. The following assessments were made based on the investigations:

- (a) Stress relief in all the Kaolin Clay samples was significant when Sampling Method B was adopted (Tests 4-8).
- (b) Stress relief in the sample of Belfast Clay was not significant when the sample was prepared using Sampling Method C (Test 13).
- (c) Stress relief in the case of the Kaolin Clay sample prepared using Sampling Method C was significant, but less than that observed in the case of samples prepared using Sampling Method B (Tests 9 and 10).

In all of the cases, the water in the filter disc was depleted before the removal of external confining pressures and therefore water influx from the filter disc to the samples cannot be the prime reason for the reduced effective stresses. Therefore, the possible contributing factors for the reduction in effective stresses under undrained loading (also referred to as stress relief) include: sample disturbances during the extraction process, cavitation of water, particle size distribution, and clay mineralogy (Graham *et al.*, 1988).

The influence of sample disturbances (i.e. induced excess pore water pressure) was eliminated by reconsolidating the samples to the same stress history in a standard triaxial cell (Sampling Method C). This was done in Tests 9 and 10 in the case of Kaolin Clay and Test 13 in the case of Belfast Clay. Where this procedure was not exercised (Tests 4-8) the average stress relief was about 30%. When the reconsolidation process was undertaken, as described in Sampling Method C, the stress relief was ~ 10% in the case of Kaolin Clay (Tests 9 and 10, Table 3). Table 3 demonstrates that the stress relief was not significant in the case of the Belfast Clay. On the aspect of cavitation, the negative pore water pressure required for the air to come out of solution in the case of Kaolin Clay is more than 1000 kPa (Lynch *et al.*, 2019). In addition, information presented in the early part of this article suggests that the *B*-value was close to 1 and any reasons to postulate any air within the pore spaces are limited. Therefore, the particle size distribution and the mineralogy of the soils tested probably have played a more significant role in relation to the stress relief in Kaolin Clay.

The Kaolin Clay used in the present investigation is made of 100% kaolinite mineral and the kaolinite particles were generally uniform in size. Also, the attractive force is more predominant in the kaolinite-water system. Under external loading, the kaolin particle arrangement tends to be more dispersed (Baille *et al.*, 2014), and in the event of an unloading process, structural rearrangement could take place. In such cases the dispersed structure can become more flocculated leading to a potential micro-remoulding process. This remoulding resulted in the reduced effective stresses in the samples. In the case of Belfast Clay, the clay mineralogy was predominantly illite and therefore the repulsive forces are more predominant (Sridharan *et al.*, 1982). Under external loading, the tendency of the particles to become flocculated is limited, particularly in the presence of silt and sand particles, which are not present in Kaolin Clay. In the event of removal of external loading, potential structural arrangement is limited, and therefore little or no micro-remoulding takes place, hence leading to little or no stress relief.

### **Stress-strain behaviour, stress paths and assessment of pore water pressure parameter**

#### **A**

Figure 6 shows the stress-strain responses of the samples of Kaolin Clay prepared using Sampling Methods B and C (Tests 4-9). The solid lines indicate the samples tested by bringing the pore water pressure to the positive range. The broken lines indicate when the samples were tested while the pore water pressures were still in the negative range. Both types of loading conditions resulted in similar axial strains at failure (6-9%). Given that the loading conditions in the present research were “stress controlled”, it was not possible to take the samples to a true critical state. The pore water pressure parameter  $A$  ( $\Delta u/\Delta q$ ), varied with axial strain as shown in Figure 6b. The value of  $A$  gradually increased with axial strain and reached a value approximating to 1 for all the tests, except Test 9 (Sampling Method C). For the latter, the  $A$ -value was over 1, which was expected for near-normally consolidated Kaolin Clay (Sivakumar *et al.*, 2018). An  $A$ -value below 1, in the other cases, implies that the samples may have been slightly overconsolidated before the undrained compression. This agrees well with the earlier comment, that samples of Kaolin Clay prepared using Sampling Method B have undergone about 30% stress relief, making them slightly overconsolidated.

Further interesting information was obtained when the responses of the samples during the undrained compression were plotted in terms of  $p'$  and  $q$ . To allow a suitable comparison, the stress paths for an overconsolidated sample (Test 10, Table 2) and a consolidated undrained

compression test carried out by adopting the procedure recommended in the BS1377, Part 8, 1990 are shown in Figures 7a and 7b respectively. The information for the latter was obtained from the authors' database. Figure 7a shows the stress path of the sample tested in Test 10. In this case the overconsolidation ratio was 2 and therefore the sample exhibited classical elastic behaviour at the early stage of compression. The sample inherited isotropic elastic properties during its initial formation, given the previous stress conditions were isotropic. On this basis, the value of the cross-anisotropic parameter  $J/3G^*$  is equal to 0, implying that the slope of the stress path would be vertical in the  $q:p'$  plane. Figure 7a shows exactly this, confirming the pore water pressure parameter  $A = 1/3$ . Figure 7b shows the stress path for a sample which was consolidated to 800kPa of effective consolidation pressure prior to undrained shearing. The shape of the stress path is typical of normally consolidated clays, where the stress path leans toward the  $q$  axis from the very beginning of the undrained compressions. These two observations aid the discussion on the stress paths for Tests 4-9 (Table 3).

The direction of the stress path at the early stage of the shearing (Test 4-9) is approximately vertical irrespective of the loading condition (negative or positive) (see Figure 7c). This indicates that  $J/3G^*$  is equal to 0, suggesting  $A = 1/3$ , and the behaviour of samples was near elastic (i.e. the samples were slightly overconsolidated). This agrees favourably with the earlier comment that stress reliefs in these samples were considerable, mounting up to 30% in Tests 4-8 and about 10% in Test 9, which would have made the samples slightly overconsolidated. The stress paths soon lean towards to the left and right, for positive and negative loading conditions respectively, and they approached the near critical state failure condition. The slope of the critical state line, represented by  $M$ , has a value of approximately 0.7 for positive loading and 0.68 for the negative loading conditions. This equates to an angle of internal friction of  $19^\circ$ . This is considered strong evidence to suggest that the responses of the samples are not significantly affected by the loading conditions (i.e. pore water pressure being positive or negative).

Figure 8 shows the stress paths obtained on samples of Oxford Clay, which was in its natural state before testing. One of the samples was tested under negative pore water pressure conditions (Test 11) and other sample was tested using the BS1377, Part 8, 1990 procedure (Test 12B). The slope of the stress paths at the early stage of shearing was represented by

$J/3G^*$ . The cross-anisotropic parameter value was approximately 0.29 under positive loading and 0.27 under negative loading conditions. The agreement between these two values is reasonably good. The slopes of the critical state line in positive and negative loading conditions are 1.44 and 1.35 respectively. The differences are small contributing to a friction angle difference of just over  $1^\circ$ . For normal design situations this difference in friction angle would not adversely impact the design process. This difference may have been due to possible variations in the natural materials as they were extracted from different sample tubes. In addition, except the sample tested using BS1377, Part 8, 1990 procedure, the deviator stress was applied using stress control loading and achieving true critical state is not possible. Also note that as both samples have had significantly different initial effective stresses, consequently they attained different deviator stresses at the critical state. Similar information is echoed in the case of Belfast Clay and the relevant information is shown in Figure 9. The slope of the initial stress path is approximately 0.18 in the case of positive loading (Test 15) and 0.15 in the case of negative loading (Test 14). The slope of the critical state line is 1.04 and 1.0 respectively for positive and negative loading, confirming the earlier postulation.

### **Evaluation**

The information presented in this article suggests that the  $B$ -value was 1 for reconstituted and natural samples of predominantly clay-rich materials. According to the BS1377, Part 8 (1990) procedure, a  $B$ -value approximating to 1 is ensured by increasing the back pore water pressure and cell pressure in steps over a set period of time. In the present investigation,  $B$ -values  $\sim 1$  (after the sampling process) were confirmed by using a high capacity tensiometer to measure the initial negative pore water pressure in the samples. However, the authors concede the fact that only a few commercial/research laboratories have the provision to use high capacity tensiometers, therefore implementing the procedure reported herein would be costly. An alternative procedure may be considered, in which the clay-rich samples can be consolidated to the required effective stresses, without the saturation process prior to shearing. Such a procedure was evaluated and proven acceptable for measuring the permeability of compacted clays prepared at the wet of optimum water content (Murray, 2002; Sivakumar *et al.*, 2015). This research presents evidence to suggest that the strength,

anisotropic parameters and the pore water pressure parameter  $A$  are unaffected by the alternative testing conditions.

## CONCLUSIONS

The saturation of samples for laboratory investigations is assessed by determining the pore water pressure parameter,  $B$ . This paper examined whether the saturation process is necessary in clay-rich soils using a chamber equipped with a high capacity tensiometer. As a part of the investigation, the Skempton's pore water pressure parameter  $A$  was also studied.

The  $B$ -value was examined on Kaolin, Belfast and Oxford clays. The observations have shown that the  $B$ -value approximated to 1 from the very beginning of the application of confining pressure in a ramped fashion under undrained conditions. This has highlighted the possibility of avoiding the need for saturating samples by elevating back and the confining pressures. Further observations collected by shearing the samples to critical state confirmed that the  $A$ -values were not significantly affected by pore water pressure being in positive or negative range. The undrained stress paths obtained through two types of loading, as stated above, resulted in a similar trend, including capturing the degree of anisotropy. Stress relief is a significant issue in the sampling process. The investigation reported in this paper suggests that, while the sampling method being the primary cause of stress relief, other factors include particle size and clay mineralogy may contribute to further stress relief.

## NOTATIONS

$J$	is the coupling parameter
$G^*$	is the modified shear modulus (Graham <i>et al.</i> , 1983)
$p'$	mean effective stress
$q$	deviator stress
$\Delta u$	is the change in pore water pressure and $\Delta\sigma_1$ and $\Delta\sigma_3$ are the changes in major and minor principal stresses respectively

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Table 1 Physical properties

<b>Soil type</b>	<b>Liquid Limit%</b>	<b>Plastic Limit %</b>	<b>Clay fraction%</b>	<b>Main mineral</b>
Kaolin Clay	70	30	90	Kaolinite
Belfast Clay	60	26	60	Illite
Oxford Clay	71	33	75	illite

Table 2 Testing schedules

Test No	Soil Type	Loading conditions	Sampling Method	Consolidation pressure kPa	Removal of water in filter	Reconsolidation	Testing condition	Void ratio	Degree of saturation $\neq$	Water content
1	Kaolin Clay	One-dimensional	A	600	NO	No	Positive	1.160	99	0.43
2	Kaolin Clay	One-dimensional	A	600	NO	No	Negative	NA		
3	Kaolin Clay	One-dimensional	A	800	NO	No	Positive	1.114	98	0.41
4	Kaolin Clay	Isotropic	B	400	Yes	No	Positive	1.154	99	0.43
5	Kaolin Clay	Isotropic	B	600	Yes	No	Positive	1.081	100	0.41
6	Kaolin Clay	Isotropic	B	800	Yes	No	Positive	1.031	100	0.39
7	Kaolin Clay	Isotropic	B	400	Yes	No	Negative	1.164	99	0.43
8	Kaolin Clay	Isotropic	B	600	Yes	No	Negative	1.085	98	0.40
9	Kaolin Clay	Isotropic	C	800	Yes	Yes	Negative	1.041	99	0.39
10	Kaolin Clay	Isotropic	C	800 $\rightarrow$ 400 (OCR =2)	Yes	Yes	Negative	1.069	98	0.40
11	Oxford Clay	Natural	D	Depth 3.5m	Not applicable	Not applicable	Negative	0.675	99	0.25
12	Oxford Clay	Natural**	D	Depth 3.5m	Not applicable	Not applicable	Positive	0.692	100	0.26
12B	Oxford Clay	Natural*	D	Depth 3.5m	Not applicable	Not applicable	Positive	0.690	100	0.26
13	Belfast Clay	Isotropic	C	600	Yes	Yes (610 kPa)	Not sheared	0.814	100	0.30
14	Belfast Clay	Natural	D	Depth 18.0m	Not applicable	Not applicable	Negative	0.786	99	0.29
15	Belfast Clay	Natural*	D	Depth 18.0m	Not applicable	Not applicable	Positive	0.771	98	0.28

\* Tested in a standard triaxial cell (BS1377, Part 8 (1990))

\*\* Not sheared

$\neq$  Calculated based on initial sample dimensions, where it was more than 100%, it was taken as 100%.

Table 3 Stress relief in kaolin and Belfast clay

Test No	Soil Type	Stress conditions	Sampling Method	Consolidation pressure kPa	Negative pore water pressure at 50 kPa of cell pressure	Effective stress kPa	Stress relief kPa
1	Kaolin Clay	One-dimensional	A	600	258	308	Not applicable
2	Kaolin Clay	One-dimensional	A	600	267	317	Not applicable
3	Kaolin Clay	One-dimensional	A	800	410	460	Not applicable
4	Kaolin Clay	Isotropic	B	400	205	255	145
5	Kaolin Clay	Isotropic	B	600	356	406	194
6	Kaolin Clay	Isotropic	B	800	551	601	199
7	Kaolin Clay	Isotropic	B	400	233	283	117
8	Kaolin Clay	Isotropic	B	600	403	453	147
9	Kaolin Clay	Isotropic	C	800	655	705	99
10	Kaolin Clay	Isotropic	C	800 → 400 (OCR =2)	298	348	52
13	Belfast Clay	Isotropic	C	610	578	628	-18

**Figure captions**

Figure 1 Testing chamber facilitated with high capacity tensiometer

Figure 2 Calibration factors for the tensiometer. (a) Intercept, (b) Slope

Figure 3 Response time of the tensiometer to negative pore water pressures in kaolin, Belfast and Oxford Clays at 50 kPa of confining pressure

Figure 4 Decay of negative pore water pressure in Belfast clay during a gradual increase of cell pressure (tensiometer verification, Test 13)

Figure 5 Assessment of B value of various samples. (a) One-dimensionally consolidated kaolin (600 kPa, Test 1), (b) One-dimensionally consolidated kaolin (800 kPa, Test 3), (c) Isotropically consolidated kaolin (400 kPa, Test 4), (d) Isotropically consolidated kaolin (600 kPa, Test 5), (e) Isotropically consolidated kaolin (800 kPa, Test 6), (f) Isotropically consolidated Belfast clay (600 kPa, Test 13), (g) Natural Oxford clay, Test 12

Figure 6 Stress-strain and pore water pressure responses of kaolin samples during undrained shearing. (a) Deviator stress vs axial strain (Tests 4-9), (b) Change in pore water pressure vs change in deviator stress (Test 4-9)

Figure 7 Stress paths of normally consolidated and overconsolidated kaolin samples. (a) Overconsolidated sample ( $OCR = 2$ , Test 10), (b) Normally consolidated (data from authors data base), (c) Stress paths obtained on samples of kaolin with initial pore water pressures in negative and positive values

Figure 8 Stress paths of Oxford Clay loaded under undrained conditions (Test 11 and 12B)

Figure 9 Stress paths of Belfast Clay loaded under undrained conditions (Test 14 and 15)

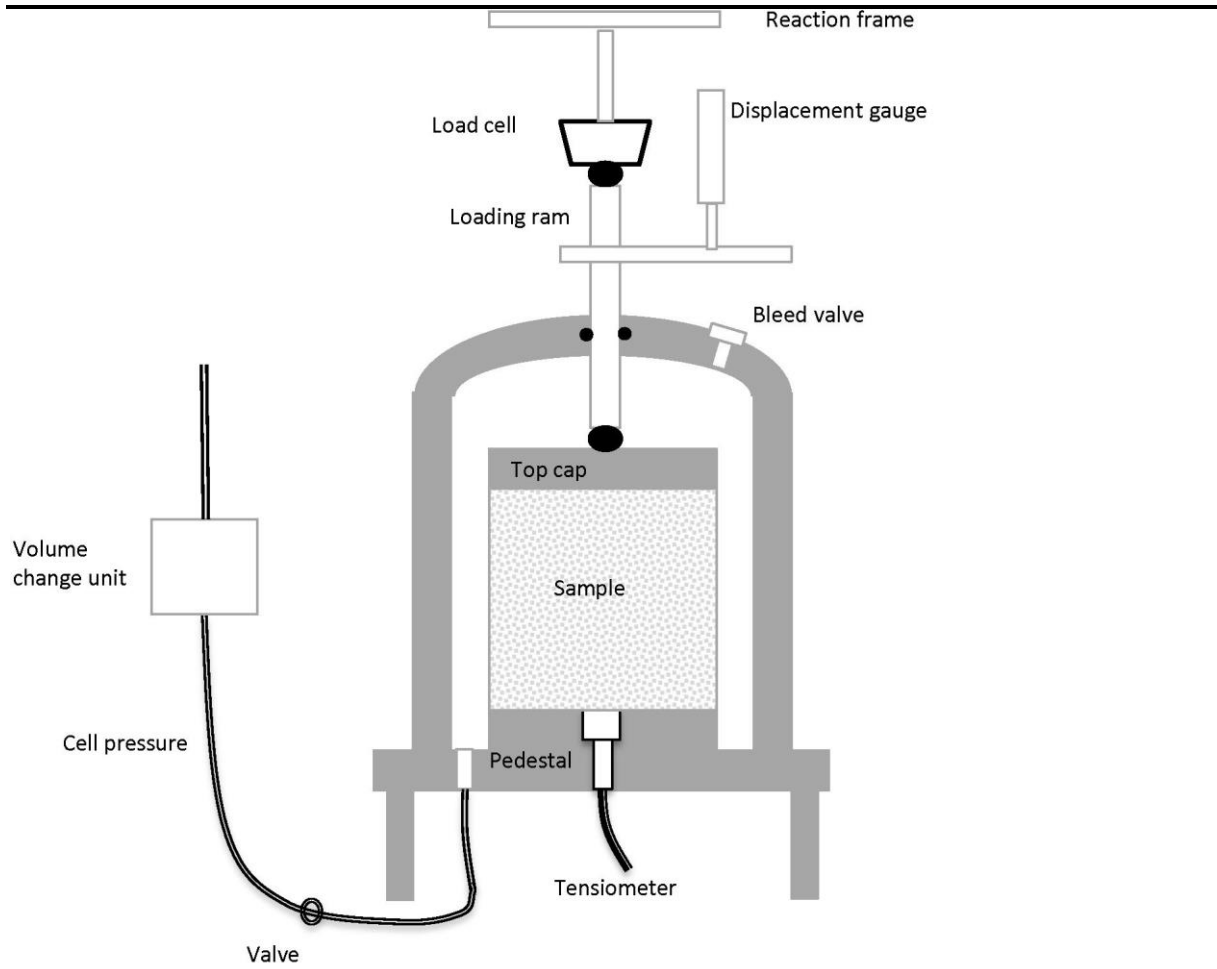


Figure 1

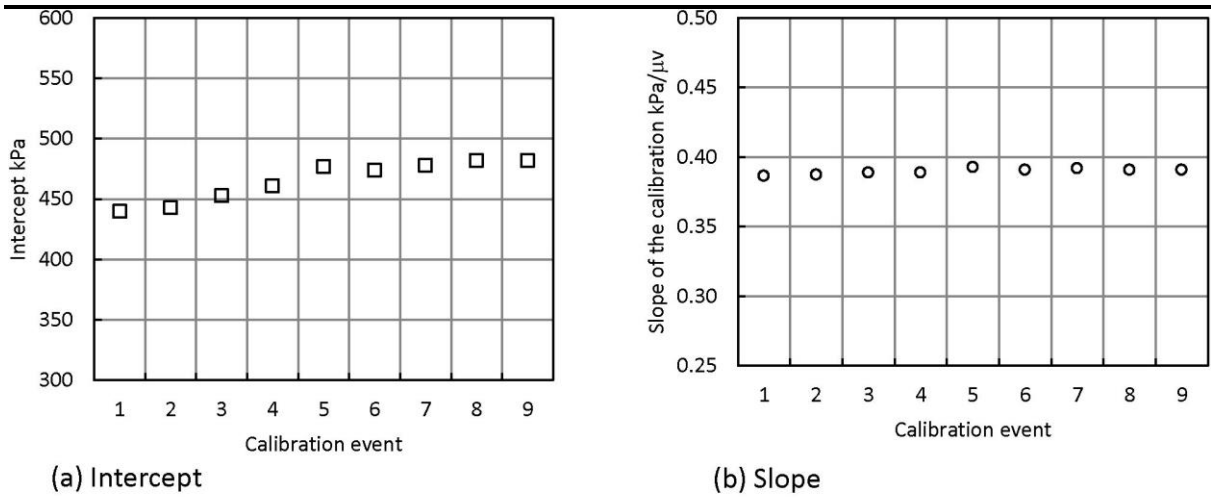


Figure 2

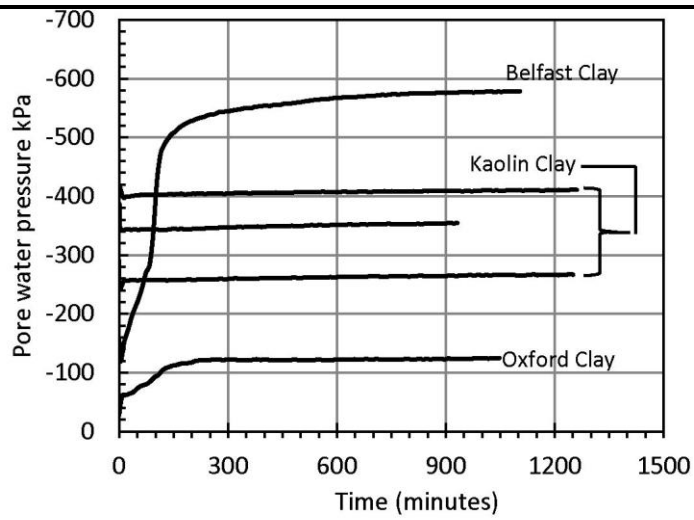


Figure 3

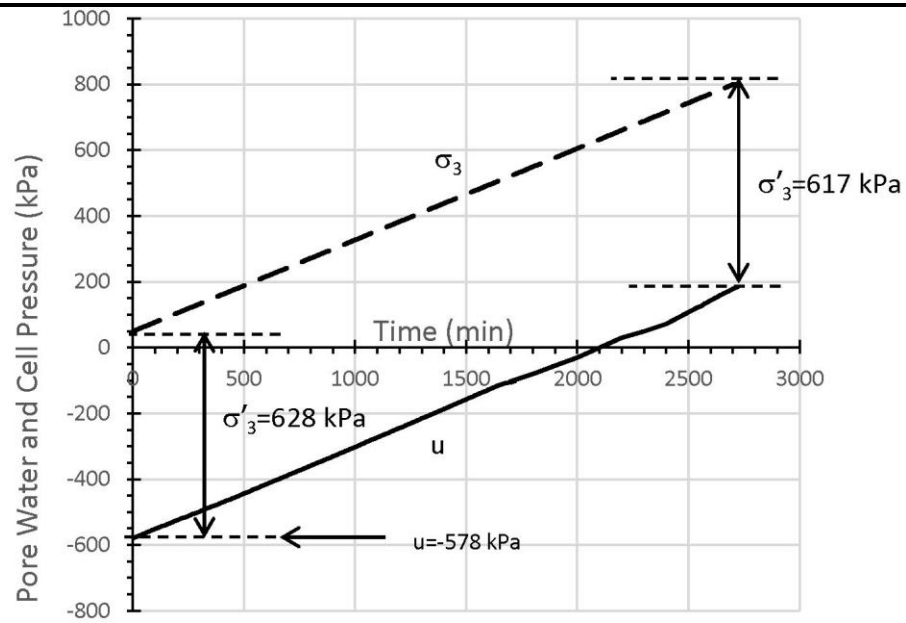


Figure 4

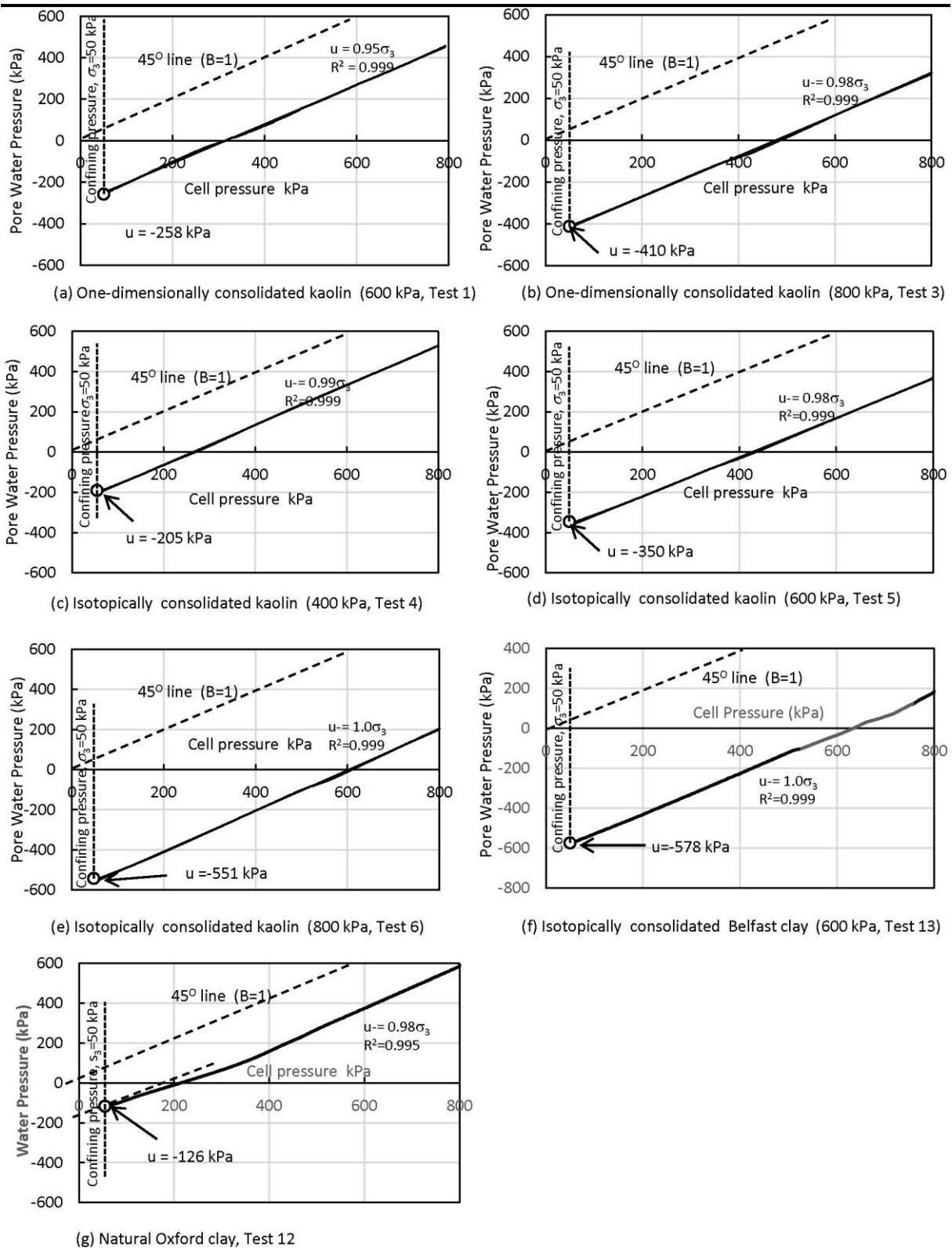
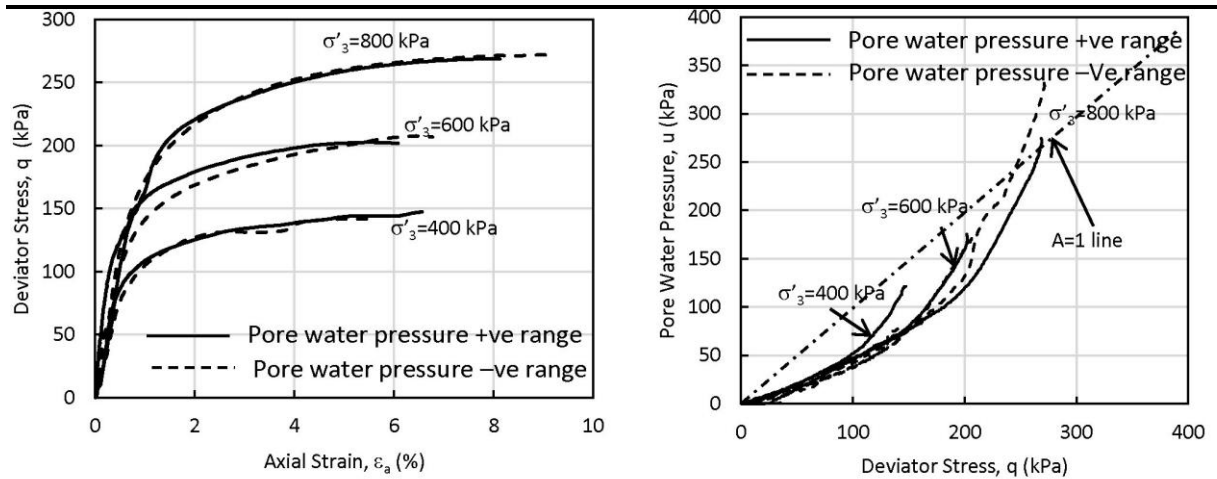


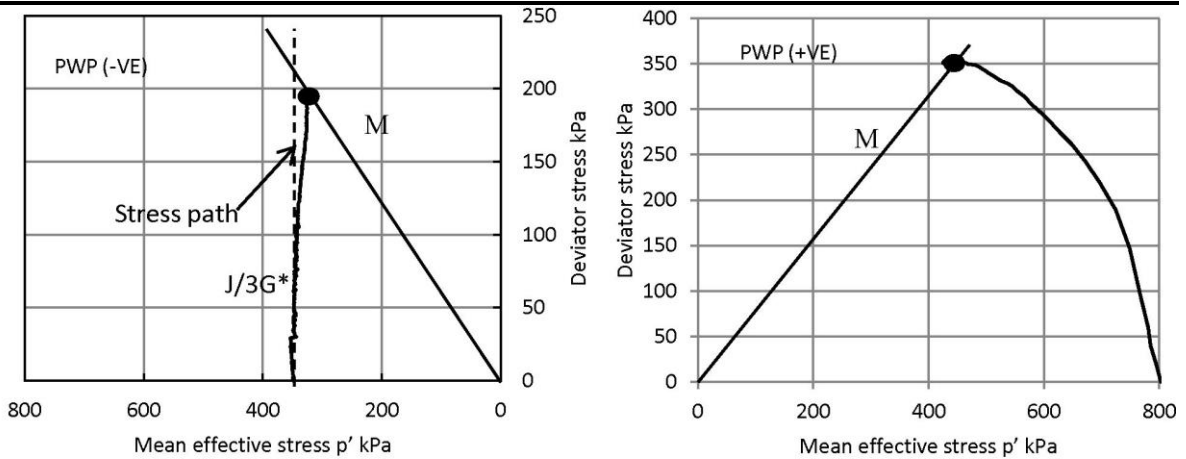
Figure 5



(a) Deviator stress vs axial strain (Tests 4-9)

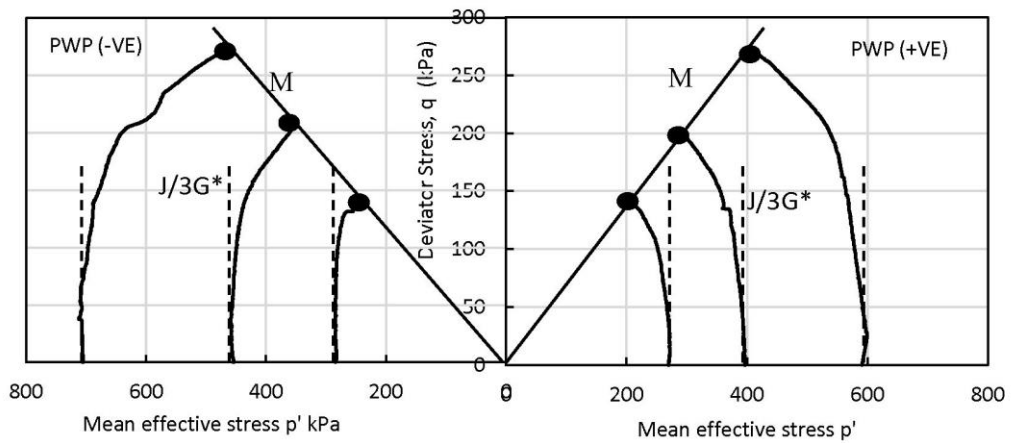
(b) Change in pore water pressure vs change in deviator stress (Test 4-9)

Figure 6



(a) Overconsolidated sample (OCR = 2, Test 10)

(b) Normally consolidated (data from authors data base)



(c) Stress paths obtained on samples of kaolin with initial pore water pressures in negative and positive values

Figure 7

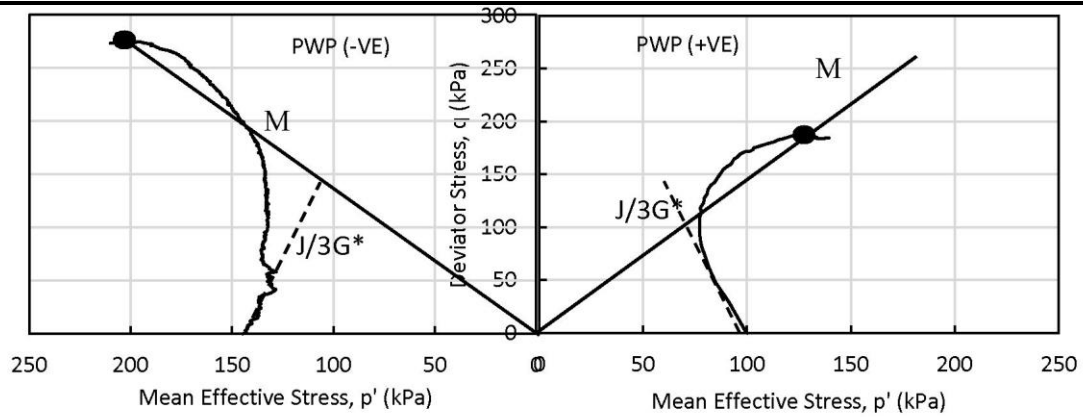


Figure 8

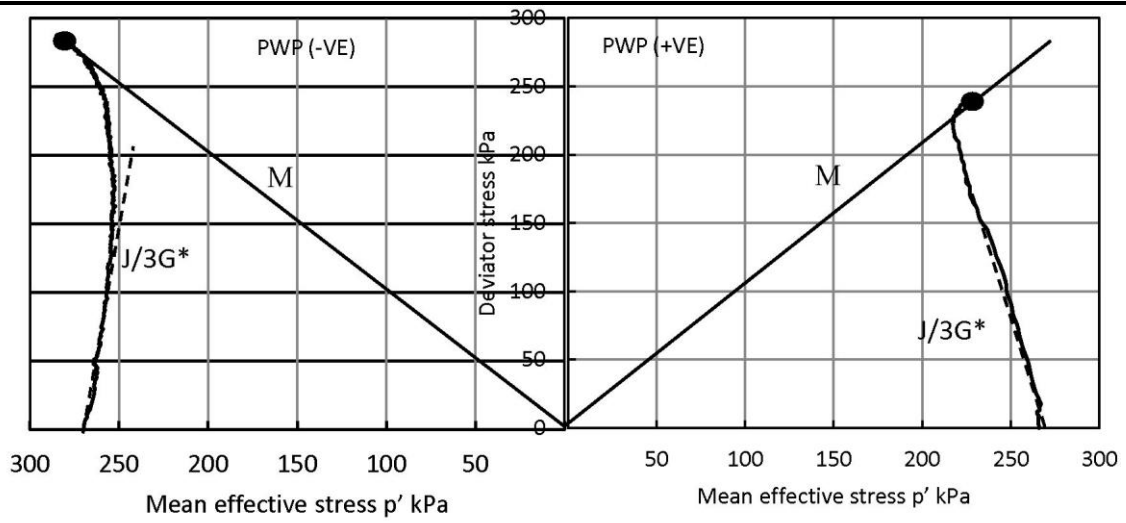


Figure 9