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Cement-Bentonite in Comparison with other Cemented Materials

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Abstract

The deformation behaviour of cement-bentonite (CB) materials used in low permeability cut-off walls is critical to the performance of these barriers *in situ*. Whilst a number of investigation have focused on the deformation behaviour of CB materials, it is suggested that insufficient knowledge has been generated to allow for the determination of the behaviour of a CB wall *in situ* with confidence. This paper reviews the deformation behaviour of other cemented particulate systems commonly encountered in civil engineering: concrete, rock, clays and cemented soils, and compares them with CB response to determine if the greater research effort associated with these materials could be used to improve understanding of CB. It is concluded a direct comparison of physical behaviour between these materials is problematic due to the differences observed. Furthermore, the formation of mircocracks prior to reaching the peak strength in cemented materials (rocks, etc.) is

an area that does not appear to have been studied previously with CB materials; yet microcrack formation could have a significant detrimental impact on the ability of a CB barrier to retard groundwater migration. Therefore, additional research is required into CB behaviour, prior to achievement of the peak strength, to determine if microcracking in CB is a significant hazard.

Keywords: Geoenvironment; Geomaterial Characterization; Waste Containment and Disposal System

1 Introduction

Cement-Bentonite (CB) cut-off walls are low permeability barriers that are used in geotechnical engineering to contain contamination plumes and control groundwater flow in engineering structures (such as dams and levees). Cement-bentonite (CB) slurry cut-off walls were initially developed in the 1970s, arising from the slurry trench cut-off walls (soil-bentonite) used in construction projects since the 1940s (Jefferis, 1997) (with soil-bentonite barriers being installed in earth dams from the mid-1960s, US EPA, 1984). The major perceived advantages of CB are (when compared with most other remedial cut-off walls used in environmental projects; Manassero et al., 1995): the self-supporting nature of the barrier; the relative uniformity of the mixture; the cost effectiveness of the technique; and the low hydraulic conductivity. The thixotropic nature of the dispersed bentonite provides the newly installed slurry wall with the ability to resist ground movements and prevent segregation of the constitutive materials as the initial stages of curing take place; over time the cementitious materials cure to form the hardened barrier.

Garvin and Hayles (1999) state that a typical CB mixture would comprise water, bentonite (30 g to 60 g per litre of water), cement (100 g to 350 g per litre of water), and cement replacement materials (Pulverised Fuel Ash, PFA, or Ground Granulated Blast furnace Slag, GGBS, at replacement levels up to 30 % and 80 % respectively) in order to achieve the low hydraulic conductivities required (ICE, 1999, specifies hydraulic conductivity of $1x10^{-9}$ m/s or lower); although mixtures outside these stated proportions can still result in the desired physical properties.

It could be argued that as the primary function of these barriers is to retard the movement of groundwater, and not to transmit load, then the deformation response of these barriers is not of paramount importance. Whilst the authors accept that these barriers must achieve the hydraulic conductivity performance criteria in order to meet the engineering need, it is difficult to justify that these barriers will not experience changes in loading conditions throughout their engineering lives; such changes could have a detrimental effect upon the performance of the barrier. For example, if a CB barrier is installed on a site that is being remediated and redeveloped then changes in loading conditions acting on the barrier could conceivably take place relatively quickly after installation due to subsequent construction activities associated with redevelopment. Therefore, it is suggested that the deformation response of these barriers must be well understood, and consideration of potential changes in loading on a barrier should be undertaken during the design stages, to ensure that these CB barriers are a sustainable engineering solution.

Whilst there has been a significant research effort into the behaviour of cements and clays, there appears to have been comparatively little research undertaken on combined cement-clay behaviour (Jefferis, 2012), resulting in a comparative lack of understanding of the behaviour of these systems. However, as considerable research has been undertaken in investigating other cemented materials, perhaps knowledge arising from this research could be applied directly to the behaviour of CB. Therefore, this paper reviews and highlights similarities and differences in stress-strain behaviour of

CB materials (particularly pre-peak to peak response to deformation) with various examples of the following materials: concrete, sedimentary rocks, clay soils and cemented soils.

2 Need for Improved Understanding of CB Deformation Behaviour

Previous research illustrates that CB slurry walls have variable physico-chemical and mechanical properties (Deschênes et al., 1995; Manassero et al.,1995; Philip, 2001; Opdyke and Evans, 2005; Joshi, 2009; Williams and Ghataora, 2011; Jefferis, 2012; Royal et al., 2013; and Soga et al., 2013). Furthermore, the combination of small proportions of high-swelling bentonite with cementitious materials results in some 'unusual' behaviour. CBs exhibit various types of failure when deformed: brittle, ductile and strain-hardening deformation response, resulting in failure via shear or tension (Manassero et al., 1995; Joshi, 2009; Jefferis, 2012; Royal et al., 2013; and Soga et al., 2013). Models have been proposed (Manassero et al., 1995; and Joshi, 2009) to describe deformational behaviour of CB (based on observed behaviour for specific CB mixtures investigated by the researchers). However, there would appear to be variations in the behaviour predicted by these models and it is suggested that deformation behaviour of other CB mixtures should be considered to improve understanding and refine the models.

2.1 Uncertainty Regarding the Required Testing Methodologies to Determine Representative Deformation Response

Manassero et al. (1995) state that specifications of mechanical properties of CB material do not often provide detailed introduction about the required tests to be conducted to check whether the requirements of minimum shear strength and maximum allowable strain without cracking are achieved: i.e. drained/undrained loading conditions, magnitude of confining stresses for triaxial tests, and rate of strain for unconfined compression strength (UCS) tests, etc. These parameters fundamentally affect the observed failure mechanisms exhibited by the CB (Section 3) and thus must be chosen carefully to reflect the loading environment that cured barrier is likely to experience *in-situ* if the suitability of the CB mixture is to be validated. Jefferis (2012) states that UCS is typically used as quality control check for CB material, while occasionally confined drained triaxial tests are specified to help in identifying the *in-situ* behaviour of the material. However, UCS is, at best, an indicative test rather than an authoritative means of determining shear strength. Furthermore, as CBs have very low hydraulic conductivities, it is uncertain under what conditions drained deformation behaviour should be considered to be applicable when CBs appear to be weaker in undrained conditions (and therefore more likely to fail before drained conditions are established).

3 Variations in CB Stress-Strain Behaviour

The deformation characteristics of CB are a function of: the constitutive materials and their respective quantities (Jefferis, 1981; and Fratalocchi and Pasqualini, 2007); the curing age (Plee et al., 1990; Deschênes et al., 1995; and Soga et al., 2013); and the environmental conditions, i.e. the nature of surrounding soil and groundwater (Joshi et al., 2010; and Soga et al., 2013), which influence the volume changes of slurry prior to hardening (due to filtration, bleed, and syneresis) (Jefferis, 2012). In addition, the deformation characteristics of CB barriers also depend on: the confining stress acting upon the newly installed and cured barrier (Manassero et al., 1995; Joshi, 2009; and Soga et al., 2013); chemical interactions between barrier and contamination (Garvin and Hayles, 1999; Philip, 2001; Jefferis, 2012; Fratalocchi et al., 2013; and Soga et al., 2013); and loading drainage conditions (Manassero et al., 1995; Joshi, 2009; and Soga et al., 2013).

Stress-strain and shear strength behaviour of CB materials have been investigated in a number of laboratory based studies, including: Deschênes et al. (1995), Manassero et al.(1995), Philip (2001), Opdyke and Evans (2005), Fratalocchi and Pasqualini (2007), Joshi (2009), Williams and Ghataora (2011), Royal et al. (2013), Soga et al.(2013) and Royal et al. (under review). The results, summarised in Figures 1 to 5, revealed broad variation in the stress-strain behaviour and shear strength with constitutive components within the CB mixtures, duration of curing, confinement and loading conditions, although these results appear to broadly support the fundamental aspects of the available models.

3.1 Constitutive Materials and Duration of Curing on the Deformation Response of the CB

The range in physical properties exhibited by the different CB mixtures described within the introduction is considerable. The type and quantities of bentonite, cement or cement-replacement materials used within the CB mixture have a significant impact upon its deformation response. In addition the duration of curing also has a significant impact upon the strength of the CB (over the first 90 days of curing, after this point the rate of change in strength with time diminishes considerably).

Royal et al. (2013) observed stress-strain behaviour for three CB mixtures containing PFA (minimum of 28 % PFA as cement replacement) in UCS and unconsolidated, undrained triaxial (TXUU) tests, and found that the mean UCS were generally lower than the ICE (1999) specification's recommendations for minimum strength (100 kPa), Figure 1 (which presents results for the strongest of the three mixtures investigated). The results in Figure 1 also show unexpected decrease in strength from 60 days to 90 days of curing, which is not considered to be indicative of the material deformation response with curing but more likely due to the natural variation of material batched from slurry. Royal et al. (2013) noted that samples containing PFA at 14 days of curing or less generally failed through development of an inclined shear plane, and rarely through propagation of vertical tension cracks, whereas those cured for 28 days or more failed through shearing of a 'cone’ or ‘wedge' that developed at the base of the loading cap with associated development of longitudinal tension cracks.

In contrast, CB containing GGBS were far stronger than those containing PFA. Soga et al. (2013) conducted UCS testing (unconfined compression strength tests give the ultimate strength at failure under compressive loading) with CB, containing GGBS (80.1% cement replacement) and obtained mean UCS as of approximately 360 kPa and 890 kPa at 28 days and 90 days respectively. Williams and Ghataora (2011) encountered similar findings (CB samples containing 80% GGBS as cement replacement investigated using TXUU, at confining pressure of 60 kPa (Figure 2) and 120 kPa, and UCS) to Soga et al., (2013). Fratalocchi and Pasqualini (2007) investigated a CB material (using a blended cement containing between 66 % to 80 % GGBS) using TXUU and TXCU (consolidated, undrained triaxial) and observed that the mixture exhibited significant increase in shear strength with curing on both types of test and the material was sensitive to the magnitude of the confining pressure in TXCU tests (more so within the first month of curing).

Royal et al. (under review) tested samples containing GGBS (80 % cement replacement, although the amount of bentonite and total cementitious material was the same as those with PFA) and found mean UCS values of approximately 260 kPa and 405 kPa for 28 days and 90 days respectively. The samples tested on UCS predominantly failed via cone and tensile cracking, with shear failure observed in samples cured for seven days (and a minority of samples at 14 days). Beads of water were observed to form on the surface of samples cured for 14 days or less, during deformation of the samples, these would flow down and pool at the base; this was not observed in samples cured for 28 days or longer.

The outcomes of these studies illustrate the differences in CB behaviour in UCS/triaxial tests with respect to variation of mixture design and particularly cement replacement materials; PFA appears to result in low compressive strengths and this supports the statement, by Jefferis (2012), that PFA should be included in addition to the cement (to improve resistance to chemical degradation) rather than act as a replacement material. The effect of mix design variation can be seen through the difference in UCS between mix design adopted by Royal et al (2013) (Figure 1) and Deschênes et al. (1995) (Figure 3). Deschênes et al. (1995) mixture did not contain any cement replacement material, and this has dramatically increased the UCS achieved in 7 days comparative to mixes which contains PFA as a cement replacement. Conversely, CB containing GGBS would appear to result in more rapid strength gain and stronger materials with curing than for mixtures containing a similar proportion of cement or cement-PFA.

3.2 Impact of Confining Pressure on Deformation Behaviour

Manassero et al. (1995) undertook triaxial testing (UU, CU, and Consolidated Drained, CD) on CB samples containing 60% GGBS as cement replacement at curing ages of 5 to 7 months, and observed that the failure mechanism varied with confinement and drainage conditions (Figure 4). Manassero et al. (1995) observed that under CU conditions the samples were brittle and developed tension cracks at low confining pressures (lower than 100 kPa effective confinement) and were brittle but developed shear planes at higher confining pressures (greater than 400 kPa effective confinement). Under CD conditions the samples failed via brittle-hardening (shear failure) at low confining pressures (less than or equal 100 kPa effective confinement) and ductile-hardening (uniform contractive failure) at higher confining pressures (greater than or equal 400 kPa effective confinement).

Deschênes et al. (1995) and Soga et al. (2013) encountered similar behaviour. Deschênes et al. (1995) observed that strain at failure in TXCD tests could exceed 8%, whereas a range of strains, 0.8 % to 1.3 % (depending on duration of curing), was encountered on the UCS (Figure 3 and Figure 4). Soga et al. (2013) noted that samples consolidated at 100 kPa (effective confinement); which was below compression yield stress of the mixture (300 kPa to 320 kPa; Joshi, 2009), exhibited ductile behaviour in drained conditions (drained strength at 660 kPa to 850 kPa, and peak axial strain at 5 % to 10 %); while samples consolidated above the compression yield stress (at 500 kPa effective confinement) exhibited strain-hardening and did not fail. This is shown in stress-strain behaviour of 'mixer cast' samples (containing 80.1% GGBS as cement replacement) in drained triaxial tests at 35 days and 90 days in Figure 5.

Soga et al. (2013) also observed that the failure pattern (UCS) for many samples was via tension cracking and were brittle (occasionally samples had inclined cracks); this is similar to failure patterns observed by Royal et al. (2013). Therefore, as confining pressures approaches zero, the triaxial conditions will be similar to UCS conditions. The undrained strength determined by Soga et al. (2013) varies from 535 kPa to 745 kPa at axial strain ranging from between 0.5 % and 2 %. Opdyke and Evans (2005) investigated a CB containing 15 % cementitious materials (air entraining cement with 75 % GGBS replacement) and noted that the majority of samples tested on the UCS failed by an inclined shear plane. The samples investigated were stronger than those investigated by Royal et al. (under review) but had a significantly lower preconsolidation pressure (100 kPa to 200 kPa as opposed to approximately 800 kPa at 90 days, respectively).

Both Manassero et al. (1995) and Soga et al. (2013) suggest that transformation in failure mechanisms observed in TXUU, TXCU and TXCD tests are a result of restructuring (collapse) of the fabric of CB samples when the compression yield stress (preconsolidation pressure) is exceeded by the effective confining pressure (it is presumed that this explains the early strength behaviour under the greater confining pressures for TXCU testing by Fratalocchi and Pasqualini, 2007). This restructuring is dependent on the effective confining pressure, the preconsolidation pressure (which will increase during the early stages of curing), and type of admixture/cement replacement material used.

Manassero et al., (1995) developed a ‘tentative’ conceptual elasto-plastic-work hardening model (based on the outcome of experimental study) which categorised the stress-strain behaviour of a specific mix CB into four zones: brittle-softening, brittle-hardening, ductile-softening, and ductile-hardening. Subsequent experimental investigations undertaken by additional researchers appear to support this model. However, the model compares deviatoric stress and void ratio with isotropic effective stress and the range of void ratios considered does not appear to represent the materials encountered by Opdyke and Evans (2005) or Royal et al., (under review), suggesting additional research is require to determine if the model is valid for additional other CB mixtures.

3.3 Impact of Undrained and Drained Conditions under Low Effective Confining Pressures (200 kPa or less) on Deformation Behaviour of CB

Results of triaxial testing undertaken by Philip (2001) imply that the stress-strain behaviour for drained (effective stress) triaxial loading is more sensitive to confining stress variation than for undrained (total stress) triaxial loading; due to the eliminated pore water pressure in drained triaxial tests. This is confirmed by Fratalocchi and Pasqualini (2007), Royal et al. (2013), and Soga et al. (2013) who also observed that undrained strength does not vary (generally) with variation of confining pressure for CB material cured for 90 days or more.

Comparison of the range of peak strengths and corresponding strains at failure for drained and undrained conditions indicates that under drained conditions the material is stronger. The practical implication being that short term response to 'rapid' loading, i.e. in undrained conditions, may result in brittle failure with inducement of cracking within the barrier, hence risking the main function of CB slurry wall: a low permeability structure. Philip (2001) states the maximum effective confining pressure is expected in field to be around 200 kPa (for shallow barriers), and from the trends illustrated in Figures 2 to 5, the low confining pressure is likely to be a dominant factor in material response to deformation. Hence, even if drained conditions were to prevail, strain-hardening may not occur. This is supported by Manassero et al.'s (1995) model which suggests at low confining pressures (up to 200 kPa), brittle-softening is the likely deformation response.

The brittle deformation response under low confinement, is likely to be accompanied with the development of cracking in the material. Royal et al. (2013) observed tension cracks develop and widen well before the peak stresses were achieved on the UCS. If cracks develop within the CB fabric before the cemented products shear, then the implications this has on the hydraulic conductivity of the barrier are not clear. It is possible that these barriers could achieve the stated strength parameter (implemented to ensure the barrier is both self-supporting and able to resist ground deformations) yet be compromised due to an increase in hydraulic conductivity with response to loading post hardening. Such an occurrence clearly is to be avoided if the barrier is to be a resilient design solution. Microcracking in other cemented structures has been studied previously and observed to happen in rocks (as discussed in section 5).

4 Comparison of CB with Concrete Deformation Behaviour

Concrete might not be an obvious material to directly compare to CB, due to differences in water-cement ratios, inclusion of well graded aggregates, etc., although there is (albeit) limited commonality in overall stress-strain response and failure mechanisms between the two materials. The general behaviour of concrete depends mainly on the mix design and water/cement ratio, which results in a variation of strength. The mechanical properties of concrete are influenced by: the water-cement ratio, the degree of compaction, properties of cement paste, and the type and grading of the aggregates (Neville, 1995; Khandelwal and Ranjith, 2013).

4.1 Brittle and Ductile Behaviour of Concrete

Concrete is commonly assumed to be brittle and this behaviour has been observed using both UCS and triaxial tests. Neville (1995) proposed that the greater the compressive strength, the lower the strain at failure (Table 1). Concrete can exhibit plastic behaviour through fracturing at relatively low strains (0.1 % to 0.5 %, Neville, 1995), thus the strain at failure of CB is approximately one order of magnitude higher than strain of failure of concrete. In addition, the higher the rate of deformation applied, the higher compressive strength achieved (Figure 6). At zero, or low confining pressures, concrete shows typical brittle failure mode followed by strain-softening behaviour (Dragon and Mróz, 1976; Neville, 1995; Kang et al., 2000; Jafarzadeh and Mousavi, 2012; and Khandelwal and Ranjith, 2013).

The application of increasing confining pressures can result in transformation of the stress-strain behaviour from brittle to ductile. Neville (1995) states that, concrete exhibits two failure mechanisms in unconfined compression: firstly, concretes tend to exhibit tensile failure perpendicular to the direction of acting load, secondly, concretes exhibits shear failure through propagation of inclined shear planes. This is in keeping with observed behaviour to CB, although the concrete's peak strength tends to be significantly greater and strain at failure smaller than CB. Conversely, Neville (1995) observes that concrete tends to fail by crushing in triaxial compression; the authors are not aware of this failure mechanisms being observed with CB. Compression crushing is exhibited by extremely stiff cemented materials such as rocks and concrete. The reported outcomes of research into CB deformation behaviour have not mentioned such a mechanism being observed in UCS or triaxial tests (Manassero et al., 1995; Joshi, 2009; Jefferis, 2012; Royal et al., 2013; and Soga et al., 2013).

4.2 Plastic Concrete

Inclusion of bentonite slurry within concrete mixes creates ‘plastic concrete’, an alternative low permeability cut-off wall material to CB. The addition of the bentonite results in a comparative increase in ductility; peak strength mobilization at strains from 0.4 % to 0.9 % was reported by: Mahboubi and Ajorloo (2005), Hinchberger et al. (2010), and Jafarzadeh and Mousavi (2012), which is considered to be slightly greater than for normal concretes, and the material achieves the low hydraulic conductivities required for cut-off walls.

Mahboubi and Ajorloo (2005), Hinchberger et al. (2010), and Jafarzadeh and Mousavi (2012) conducted UCS and consolidated drained triaxial tests (TXCD) on plastic concrete mixtures (at varying ages, water-cement ratios, and bentonite contents), Figure 7. The stress-strain relationships of plastic concrete show that increasing the age and the effective confining pressure from zero, in unconfined testing, to 800 kPa in drained triaxial testing resulted in increasing compressive strength and corresponding strain at failure. Furthermore, the behaviour changed from brittle strain-softening to become ductile, and the failure modes of plastic concrete change from tensile to shear as confining pressure is increased (Figure 8). This is similar to the pre-peak behaviour of CB materials with GGBS in certain circumstances (i.e. under effective confining pressures less than 500 kPa), but is not an ideal comparison as CB experiences strain-hardening in drained conditions and also appears to experience greater strains at failure (for lower peak strengths).

5 Comparison of CB with Sedimentary Rock Deformation Behaviour

Certain types of sedimentary rocks could be considered analogous to both concrete and CB, for example sedimentary rocks can experience changing deformation characteristics with confinement similar to CB. Jones et al. (1984), and Clayton and Mathews (1987) investigated various chalks, Nygard et al. (2006) investigated different shales and mudrocks, and variable deformation behaviour from brittle to ductile response with loading conditions were observed. Similar responses have been observed with sandstones, etc.: Yang et al., 2011; Yang and Jing, 2011; Wang and Xu, 2013; and Alam et al., 2014. However, the compressive strength of these relatively 'weak' rocks are often significantly greater than those encountered with CB, and it is clear that direct comparison between the strength of rock and hardened CB slurry is unsatisfactory.

Despite this, sedimentary rocks can experience tension and combination (tension-shear) modes of failure at zero or low confining pressures (Figure 9), which is similar to CB (Figure 4) and plastic concrete (Figure 8). Research reveals the development of microcracking within sedimentary rock's fabric prior to achievement of the peak strength (Farmer, 1983; Yang et al., 2011; Yang and Jing, 2011; Wang and Xu, 2013; Jia et al., 2013; and Alam et al., 2014), and this behaviour could also occur in CB materials when deformed.

5.1 Microcracking

A generalised curve of stress-strain behaviour of brittle rock is presented in Figure 10: the model suggests that rock deformation process in UCS test can be divided into six stages which are indicated by letters A-F. Stages B, C, and D are the main three stages at which microcracking events are concentrated. Stage (B) denotes deformation that is largely recoverable but microcrack propagation is argued to onset within this stage at about 35 % to 40 % of the peak stress (Farmer, 1983). Stage (C) still represents recoverable deformation but microcracks exhibit an increase in growth; this stage ends at approximately 80% of the peak strength at which non-recoverable deformation begins (stage D). Stage (D) results in rapid acceleration of microcracking events: clusters of cracks in the zones of highest stress tend to coalesce and start to form tensile fractures or shear planes (this is a function of rock strength and degree of confinement) (Farmer, 1983). Therefore, during stage (D) the rock is suspected to experience changes in hydraulic conductivity due to initiation and propagation of microcracks, yet the rock has not reached its peak strength.

Subsequent research into the brittle behaviour of sedimentary rocks have furthered understanding of microcrack development (Yang et al., 2011; Yang and Jing, 2011; Nicksiar and Martin, 2012; and Jia et al., 2013) and validated the basic principle of the model presented in Figure 10. Whilst rocks are likely to be much stronger than CB, this model (Figure 10) is worthy of note as it depicts the development of microcracking at stresses significantly less than the peak strength. If the primary role CB barriers are to control groundwater flow, then the development of microcracks prior to peak strength may result in significant loss of performance. It is unclear if this model is valid with respect to CB material behaviour, and further research is clearly required to understand the relationship between: loading environment, development of cracking, and hydraulic conductivity of the CB barrier material.

6 Comparison of CB with Clay Deformation Behaviour

6.1 Stiff Clay Deformation Response

CB behaviour has previously been compared to that of clays (Evans, 1993; and Garvin and Hayles, 1999). Evans (1993) refers to the strength of CB being akin to stiff clay soils. Whilst the behaviour in certain instances appears similar, it is not clear if the mechanisms controlling stabilised colloidal suspensions (CB) (Jefferis, 2012) are the same as those for clays. Clay soils experience phenomena such as electrostatic, physio-chemical, or other forces that act to connect the particles (Cotecchia and Chandler, 1997), often labelled as cohesion; these attractive interparticle forces are essentially a function of the clay minerals present, and control the flocculation-defloculation behaviour in suspension (Mitchell and Soga 2005; Atkinson, 2007). These forces are also important in denser soils as they may influence the intergranual stresses and control the strength at interparticle contacts, which in turn controls resistance to compression and strength (Mitchell and Soga, 2005). Therefore, the mechanical response of clays depends on: their consolidation state, their structure (the combination of fabric and bonding), their loading history (Cotecchinaet al., 2007) as well as the loading conditions (undrained/drained). Whilst these conditions may be quantified, or justifiably estimated, this is not the case of CB slurry barriers where the loading history does not exist, and consolidation conditions are not confidently known.

6.2 Overconsolidated Clay Deformation Response

CB has previously been likened to overconsolidated clay as the undrained response of overconsolidated clays would appear to be similar to that of CB (containing GGBS, those with PFA illustrate significant softening post-peak). However, this analogy appears to be less than ideal as the consolidated drained response for overconsolidated clays do not appear to replicate those encountered for CB. Burland (1990) investigated London Clay (Figure 11) and a clay from Todi, Italy (overconsolidated intensely fissured), and found the behaviour of the two clays to be similar. The response of overconsolidated soils to deformation reported by Burland (1990) has also been encountered by others (e.g. Roscoe et al., 1958; Georgiannou and Burland, 2001; and Atkinson, 2007).

Whilst there may be similarities between undrained stress-strain response and consolidation behaviour of CB (Figure 12) with overconsolidated clays (with respect to the presence of preconsolidation pressures in overconsolidated soils and apparent ‘preconsolidation pressures’, or critical stresses, in the case of CBs), the similarities between volumetric responses of these materials seems less clear. Overconsolidated clays are expected to dilate upon shearing, in contrast, Soga et al. (2013) illustrated that CB compresses upon loading (with large amount of volume change in drained triaxial tests, regardless of the magnitude of confining pressure). It is these differences that make comparisons between deformation behaviour of CB and overconsolidated/stiff clays unsatisfactory.

7 Comparison of CB with Cemented Soil Deformation Behaviour

CB deformation behaviour bears, albeit it limited, similarities to concretes and clays, and may experience microcracking with deformation (as identified with deformation of brittle sedimentary rocks), although attempting to make such comparisons between these materials is an effort to better understand CB response appears inadequate. Clearly the behaviour of cemented soils should also be compared to the behaviour of CB mixtures, although many soils stabilised with cement differ from CB as the latter has cemented colloidal structure.

Soils may be naturally cemented due to geological processes (i.e. precipitation of minerals such as calcite, silica, and/or other inorganic or organic components), or 'man-made' with the inclusion of cementitious materials, or stabilisers, such cement, PFA, GGBS, rice husk ash (RHA), lime, gypsum, etc. (Indraratna et al., 1995; Cokca, 2000; Lee and Lee, 2002; Chew et al., 2004; Rao and Shivananda, 2005; and Consoli et al., 2007). Introducing stabilising materials into a soil is undertaken for a number of reasons including the creation of bonds to enhance the mechanical properties of weak soils and reduce compressibility.

7.1 Deformation Behaviour of Cemented Soils

Mitchell and Soga (2005) state that behaviour of cemented soils is a function of the time at which cementation bonds developed. Overburden loading might be applied after cementation in artificially cemented soils, whereas it might be applied during, or shortly after, the development of cementation in natural soils. If a particle contact is cemented, it is possible for some interparticle forces to become negative due to the tensile resistance (or strength) of bonds; thus stiffness and strength properties of a soil are likely to differ according to when, and how, cementation was developed (Mitchell and Soga, 2005). Schnaid et al. (2001) observed that cemented soils exhibit very stiff behaviour prior to a well-defined yield point that is extensively controlled by cementation bonds, followed by plastic deformation upon reaching failure. Furthermore, cemented soils have been observed to dilate upon shearing, much like overconsolidated clays, (e.g. Lade and Overton,1989; Indraratna et al., 1995; Lee and Lee, 2002; Moses et al., 2003; Horpibulsuk et al., 2004; Chew et al., 2004; Chiu et al., 2008; and Kamruzzman et al., 2009) (Figures 13 to 17).

Cementation increases peak strength, initial stiffness and brittleness, and generates tensile strength (Leroueil and Vaughan, 1990). In artificially cemented soils, for a given water content, an increase in cement content (over the range of proportions investigated) results in an increase in peak strength and stiffness, thereby reducing the strain at which failure occurs (Lade and Overton, 1989; Moses et al., 2003; Horpibulsuk et al., 2004; Chew et al., 2004; Consoli et al., 2007; and Kamruzzaman et al., 2009) (Figure 13). Cementation also results in change in behaviour from plastic to brittle under drained conditions (Lee and Lee, 2002). However, the brittle behaviour (for a given cement content) has also been observed to transform to a ductile response, resulting in higher peak strengths, with increasing effective stress levels; although the confining stresses required for this transformation in behaviour are significant (i.e. 10 MPa) (Schnaid et al., 2001; Horpibulsuk et al., 2004; and Kamruzzaman et al., 2009) (Figure 15). These descriptions also demonstrably apply, to varying degrees, to: concrete, sedimentary rocks and CB. Manassero et al. (1995) and Soga et al. (2013) illustrated that if confining pressure is sufficient, and drained conditions prevail, then the material transforms from ductile response to strain-hardening from very low strains, and at this point, CB appears to deviate from the trends of other cemented soils.

7.2 Limitations When Comparing Behaviour of Cemented Soils with CB

Seeking to further understanding of CB behaviour with comparison to published behaviour of cemented soils appears unadvisable due to the considerable range of deformation responses encountered with cemented soils (Lade and Overton, 1989; Indraratna et al., 1995; Cokca, 2001; Lee and Lee, 2002; Moses et al., 2003; Rao and Shivanda, 2003; Chew et al., 2004; Horpibulsuk et al., 2004; Lee at al., 2005; Consoli et al., 2007; Tang et al., 2007; Chiu et al., 2008; Kamruzzman et al., 2009; and Horpibulsuk et al., 2012).

For example, Lee and Lee (2002) investigated cement-stabilised kaolin, and Kamruzzaman et al. (2009) studied cement-stabilised soft clay both in UCS (Figure 15) and triaxial tests. Strength increased with duration of curing for all mixes investigated, and the material increased in brittleness with age and increasing cement content. Strain-softening occurred post peak strength under both UCS and triaxial loading, even under high effective confining pressures. In undrained testing of cement-stabilised kaolin within the triaxial, failure was not brittle, instead strain-softening behaviour was encountered; samples were mostly failing through developing shear planes, and occasionally by crushing through growth of nearly vertical cracks at the bottom or the top of the sample. The failure behaviour observed in drained conditions within the triaxial once again was brittle, (especially pronounced at high cement contents and duration of curing with low confining pressures); plastic shearing in all drained triaxial tests and samples failed through development shear failure planes with associated sample barrelling. Lee and Lee (2002) concluded that the material was stiffer under undrained condition than under drained conditions, and the stiffness of cement-stabilised kaolin is greatly influenced by confining pressure. This behaviour bears similarities with the other cemented materials considered herein (i.e. drainage conditions and stiffness on CBs, Figure 4, and stiff clays, Figure 11; influence of confining pressure on CBs, concretes, and rocks). Feda and Herle (1995) observed the behaviour of undisturbed saturated samples of clay (naturally cemented clay) from western Bohemia (Figure 16) in TXCU (effective confining pressure range of 400 kPa to 1200 kPa), and suggested that the post-peak behaviour is a result of progressive debonding of such soils culminating in the total collapse of the cemented structure. Increasing the confining pressure resulted in increased peak deviator stresses, whereas no significant change in strain at failure was observed. Conversely, Moses et al. (2003) investigated undisturbed soft clays (Indian costal marine, naturally cemented clay) under TXUU (Figure 17) and did not observe a noticeable peak strength, instead strain-hardening was exhibited when applying confining pressures higher than preconsolidation pressure (75 kPa). CB material does not exhibit post-peak strain-hardening under TXUU conditions; and does not always undergo significant strain-softening behaviour under undrained triaxial loading, this appears to be a function of the cement-replacement material used within the mixture.

8 Conclusions

Stress-strain behaviour, shear strength, and failure modes of CB mixtures, for low permeability cut-off barriers, show significant variation in behaviour that depends on: duration of curing, type and proportion of cementitious material within the mixture, drainage conditions, as well as magnitude of confining pressure in drained conditions. CB would appear to be stiffer, yet weaker, in undrained conditions when compared to drained. In drained conditions the deformation response changes with confinement: from brittle to ductile, then to strain-hardening (apparent from low strains) the transformation from ductile to strain-hardening requires significant confining pressures, and it has been questioned if such confining pressures are likely to be encountered in shallow barrier installations.

Understanding of CB behaviour is limited, leaving many questions regarding barrier performance unanswered. In an effort to broaden understanding of CB behaviour, this paper has considered the deformation behaviour of other cemented particulate systems used in civil engineering to determine if the deformation response for these materials are comparable to CB; and thus if understanding developed from the significantly greater research effort into these materials could be applied to CB.

Concrete does not appear to be a particularly suitable material to compare to CB as the strength and brittleness of concrete are significantly greater than CB. Concrete does not suffer from the extreme variability of post-peak behaviour (in UCS and triaxial tests) encountered with CB. Plastic concrete is used in the same applications as CB, and has two common ingredients: cement and bentonite, yet plastic concrete does not appear to illustrate the transformation in deformation behaviour with magnitude of confining pressure. The comparison of plastic concrete with CB is less than ideal as CB can exhibit strain-hardening behaviour under drained loading as well as greater strains at failure (for lower peak strengths than plastic concrete).

The behaviour of rocks would appear a poor comparison to CB, even ‘weak’ sedimentary rocks are likely to be significantly stronger than the majority of CB used for low permeability cut-off wall applications. However, deformation behaviour of rocks has been widely studied, and it has been identified that microcracks will form (within brittle materials) and congregate well before the peak strength is achieved, and this could also occur within CB barriers. Microcracking prior to peak strength in undrained conditions could result in an increase in the hydraulic conductivity of the CB barrier, therefore potentially compromising its ability to achieve its primary role, and thus this merits investigation.

CB material has much lower strength than concrete, and hence it was argued that perhaps CB has more in common with clays. This suggestion does not seem wholly applicable; behaviour of ‘clay’ soils are subjected to extensive variability due to the significant spectrum of properties and composition of soils that can be classified as clay, and such variation makes direct comparison of behaviour between CB and published findings of clay deformation difficult.

Cemented soils would appear to be an obvious comparator with CB as, in general cemented soils exhibit brittle behaviour similar to CB materials. In addition, cemented soils are sensitive to: confining pressure, drainage condition, and age (in case of cement-stabilised clays). However, it is difficult to use published information on cemented soils when attempting to further understanding of CB behaviour due to the wide range of behaviours associated with these materials. This would also appear to be the case with clay soil deformation response.

Having considered, concrete, plastic concrete, sedimentary rocks, clay soils and cemented soils in an effort to further understanding of CB behaviour it is concluded that these comparisons are unsatisfactory. There is either too great a range of differences between the materials when compared to CB or, where there does appear to be comparability (for example cemented clays), there is such a range of behaviours exhibited by these types of materials that making such comparison could be either misleading or fundamentally flawed. Jefferis (2012) states that there is insufficient research undertaken on cemented clay systems and the authors fundamental echo these sentiments; increased understanding of CB is required if these materials are to be used efficiently.

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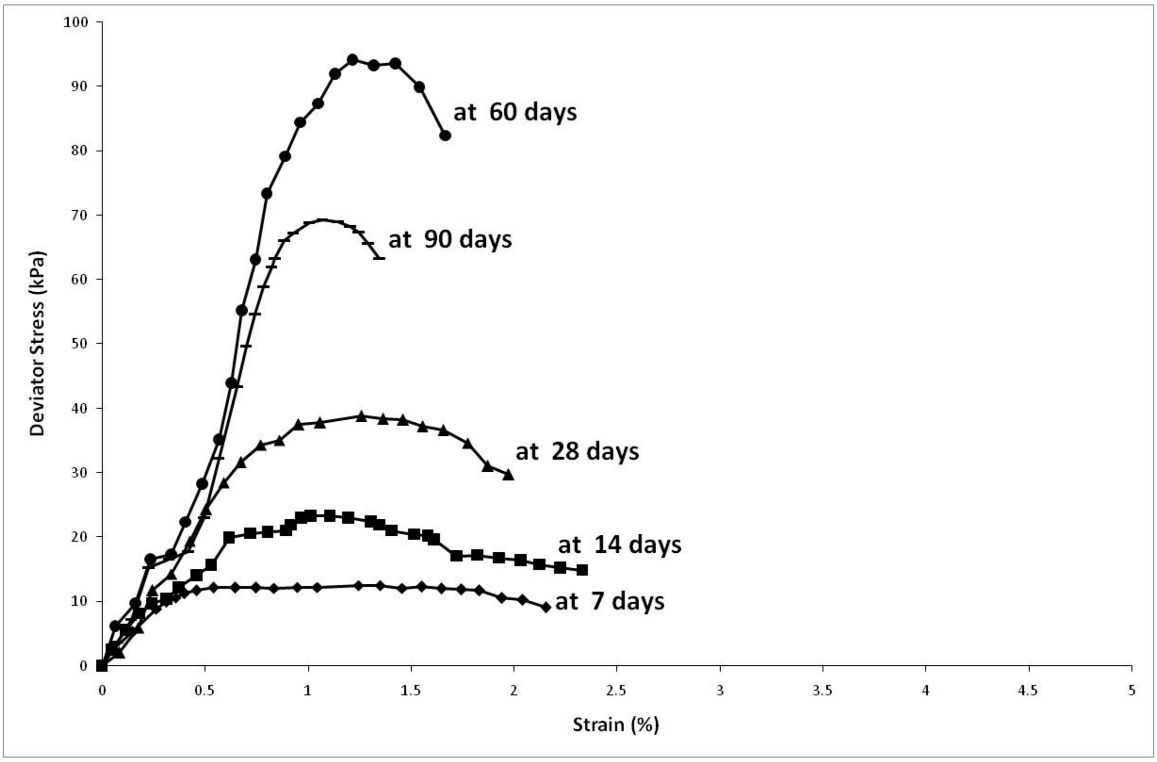

Figure 1: Mean stress-strain behaviour of CB samples in unconfined compressive strength (UCS) test at 1.0 mm/min rate of deformation. Mix design used: 40 g of bentonite per litre of water, and 200g of cementitious material per litre of water (with 28% PFA as cement replacement). Reproduced from Royal et al. (2013) with kind permission of Spinger Science+Business Media.

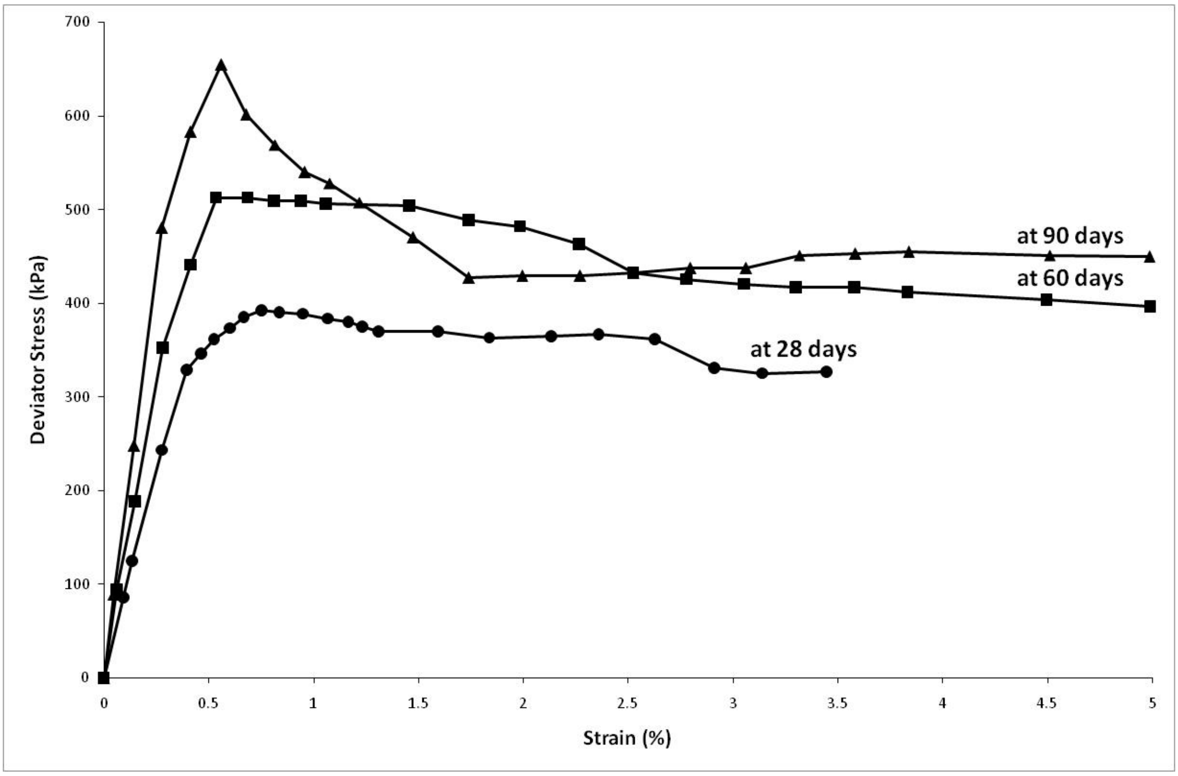

Figure 2: Stress-strain behaviour of CB samples containing 80% GGBS (proportion of cementitious material) in undrained triaxial tests at 0.4 mm/min at 60 kPa confining pressure, after Williams and Ghataora (2011). Mix design used: 37g bentonite per litre of water, and 160g of cementitious material (with 80% GGBS as cement replacement).

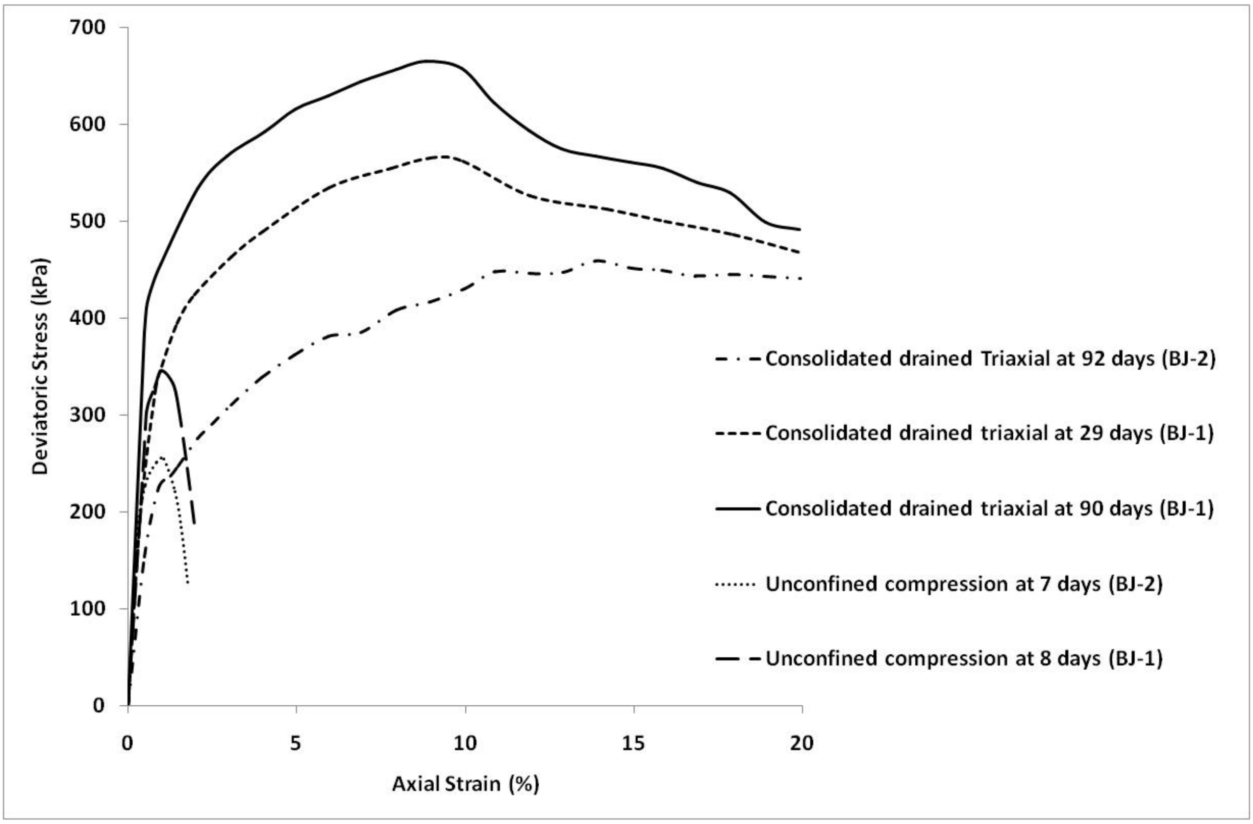

Figure 3: Typical CB stress-strain relationships in UCS tests and consolidated drained triaxial tests after Deschênes et al. (1995). The effective confining pressure was 100 kPa, and the back pressure was 690 kPa. Mix designs used:

* BJ-1(% by weight): bentonite/water= 3.72%; cement/water=35%; retarding agent/water= 0.17

* BJ-2(% by weight): bentonite/water= 3.68%; cement/water=30%; retarding agent/water= 0.11

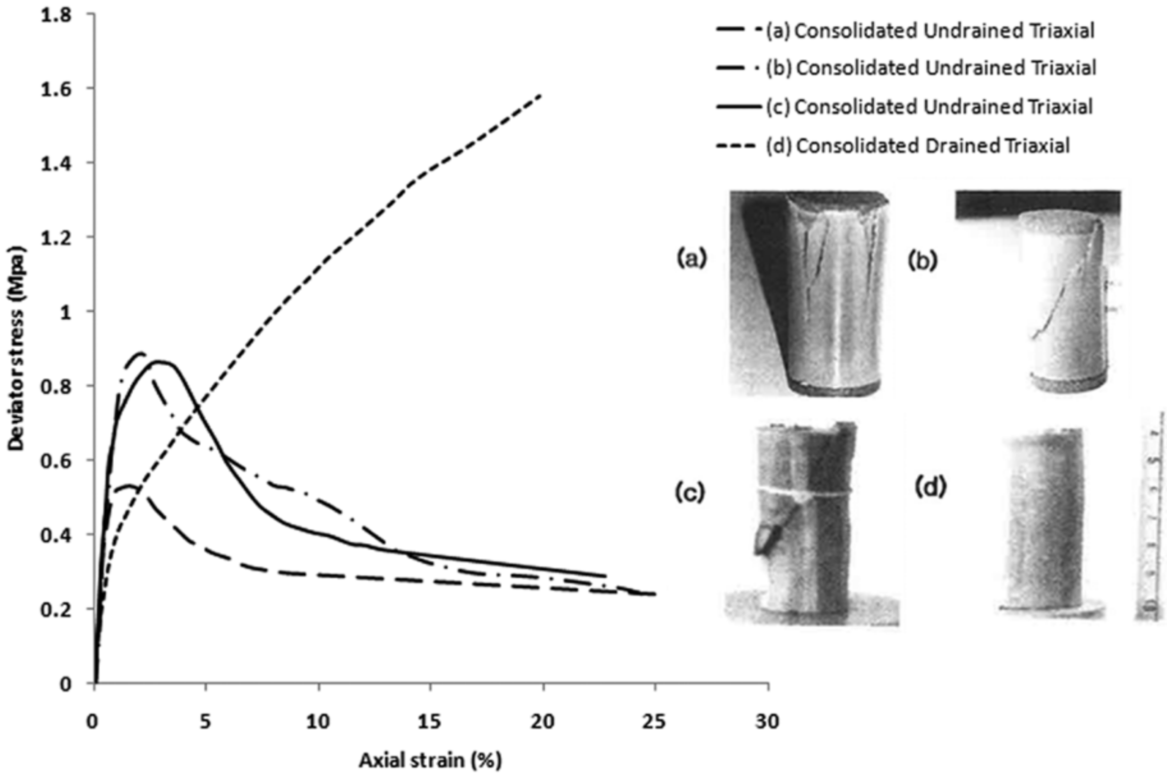

Figure 4: Stress-Strain relationships and modes of failure in different triaxial tests: (a) consolidated undrained triaxial (brittle-softening, tension failure); (b) consolidated undrained triaxial (brittle-hardening, shear failure); (c) consolidated drained triaxial (brittle-hardening, shear failure); (d) consolidated drained triaxial (ductile-hardening, uniform contractive failure). Curing age of samples tested vary from 5 to 7 months. Mix design (by weight): 76.8% water, 4% bentonite, and 19.2% blast furnace cement (containing 60% GGBS). Reproduced from Manassero et al. (1995) with kind permission from ASCE.

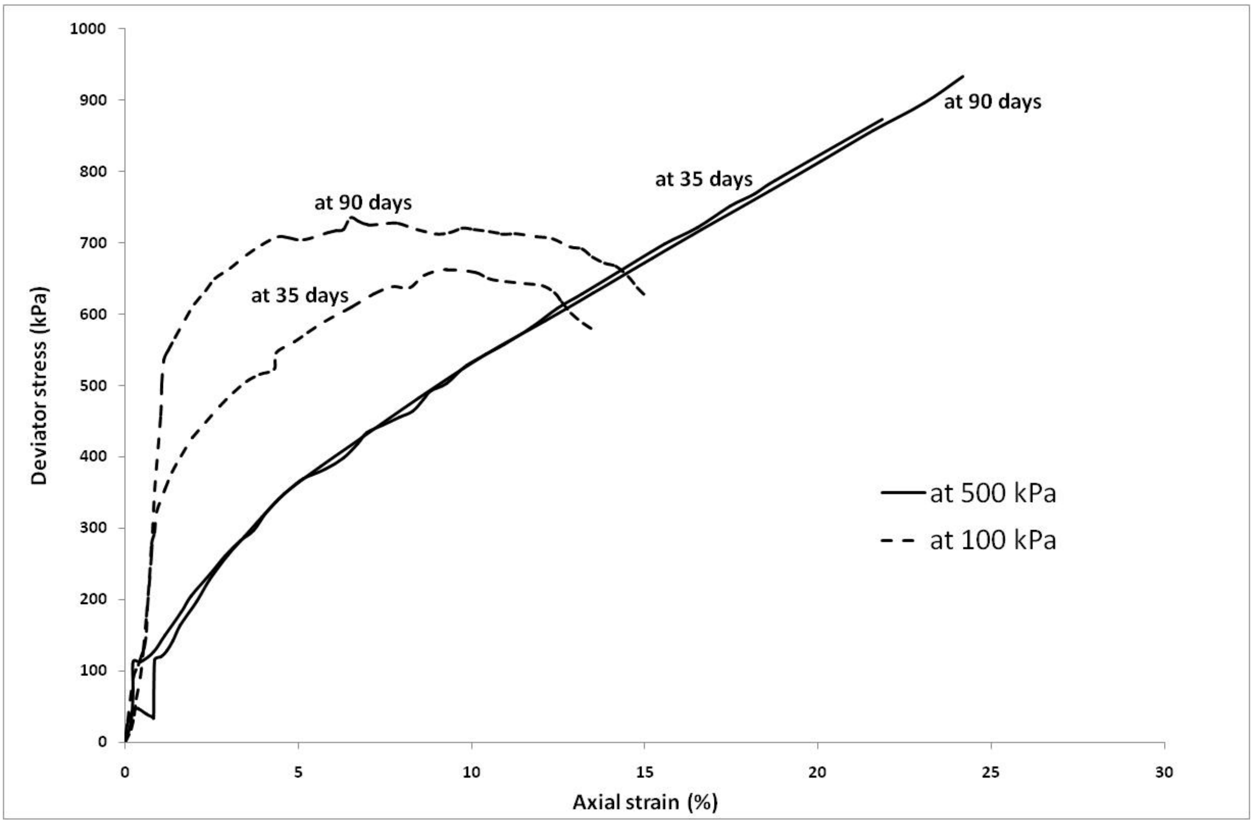

Figure 5: Stress-Strain behaviour in drained triaxial test at varying confining stresses (100 kPa and 500 kPa) on 'mixer-cast' samples at 35 days and 90 days. Mix design (by weight): 3.4% bentonite, 2.5% ordinary Portland cement, 10.1% GGBS (i.e. 80.1% cement replacement), and 84.0% water. Reproduced from Soga et al. (2013) (figure 23, page 159) with kind permission from Taylor and Francis Group.

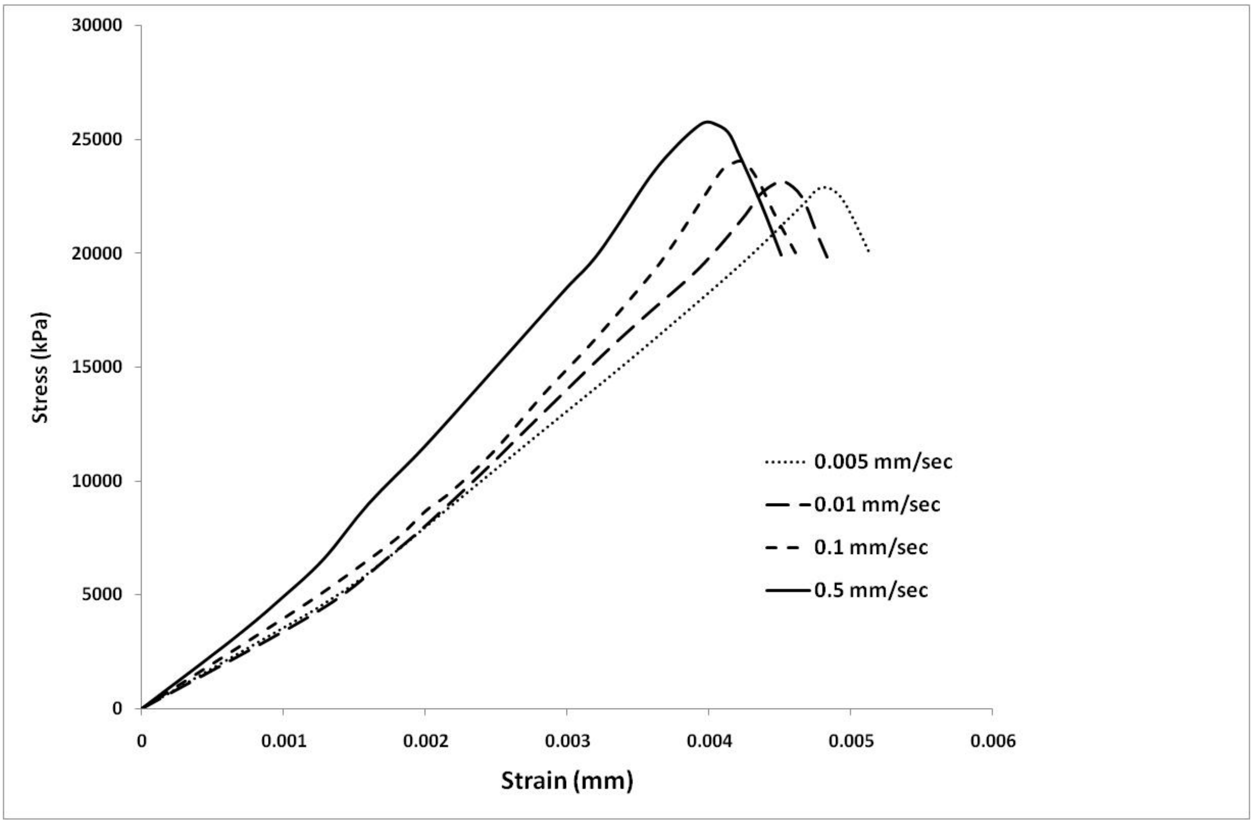

Figure 6: Typical stress-strain relationships for cement mortar mix at varying strain rates. Reproduced from Khandelwal and Ranjith (2013) with kind permission of Spinger Science+Business Media.

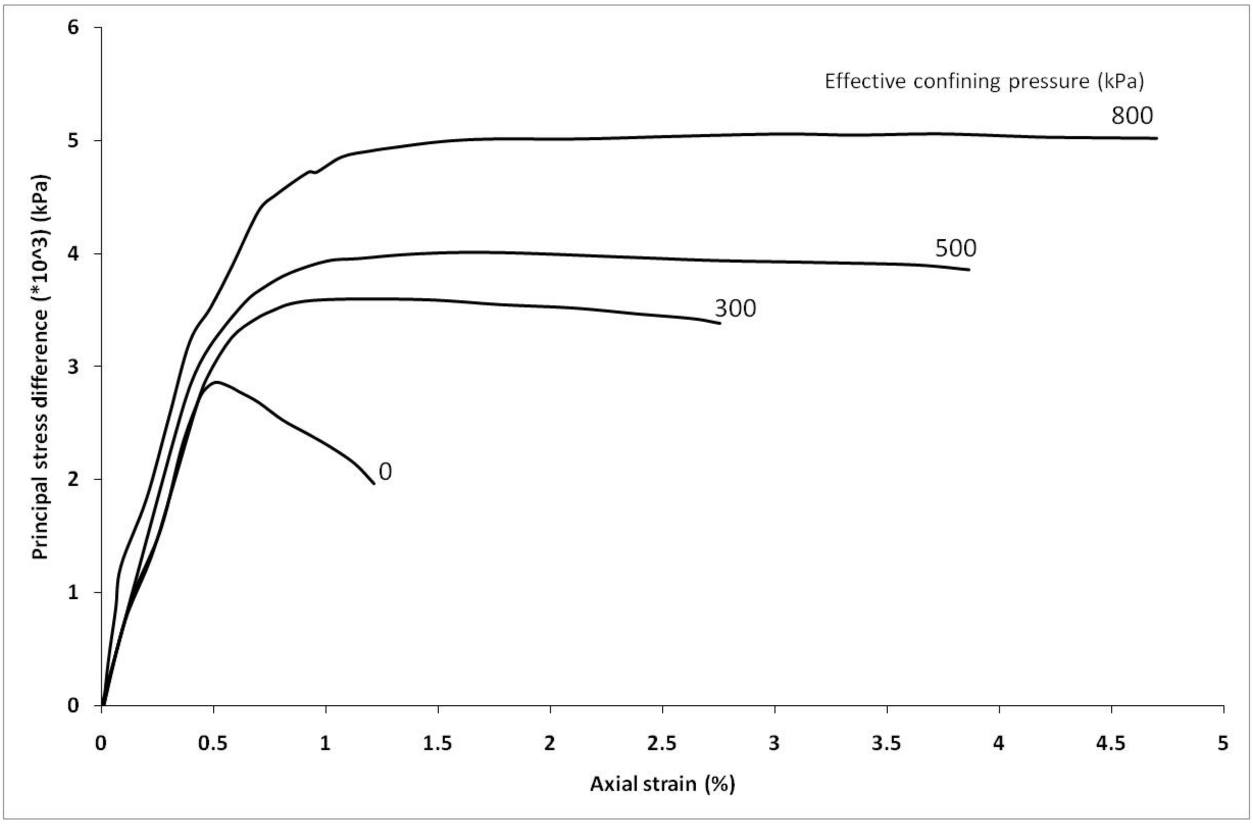

Figure 7: Effect of confining pressure on stress-strain behaviour of plastic concrete specimens (water-cement ratio= 1.59, and bentonite-cement ratio= 0.14) at 28 days. Reproduced from Mahboubi and Ajorloo (2005) with kind permission from Elsevier.

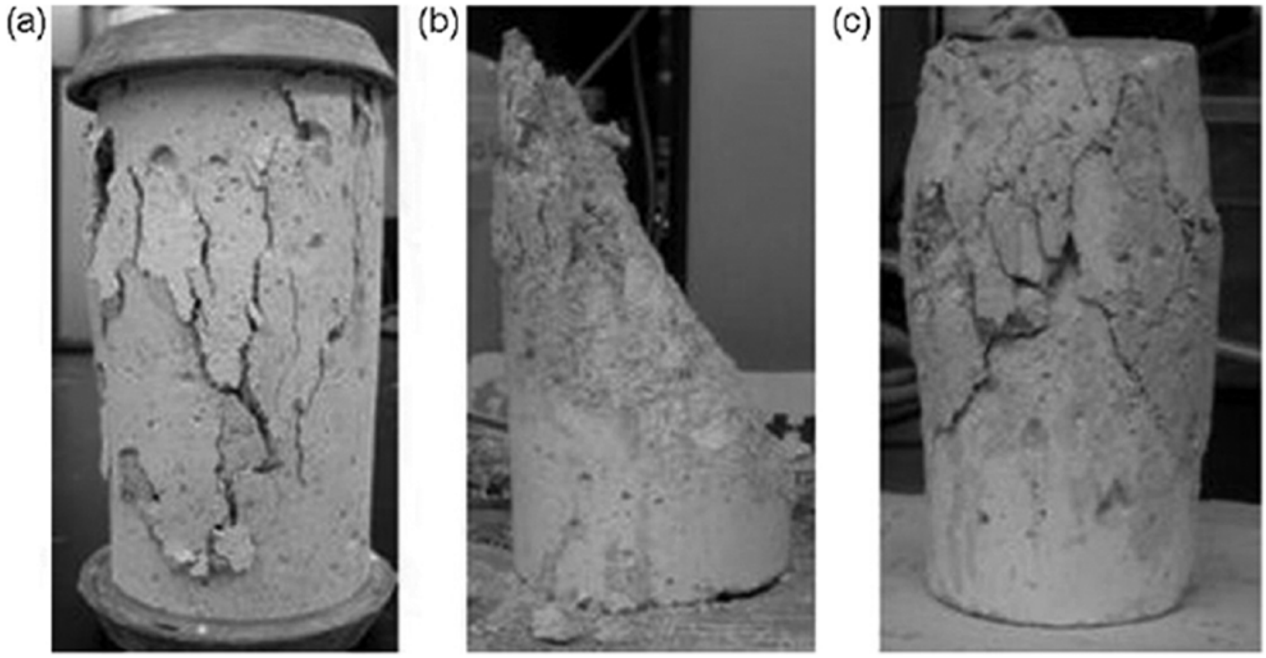

Figure 8: Crack patterns in plastic concrete for (a) low confinement (100 kPa), (b) intermediate confinement (400 kPa), (c) high confinement (900 kPa). Reproduced from Hinchberger et al. (2010) with kind permission of NRC Research Press, National Research Council of Canada in format Republish in a journal/magazine via Copyright Clearance Center.

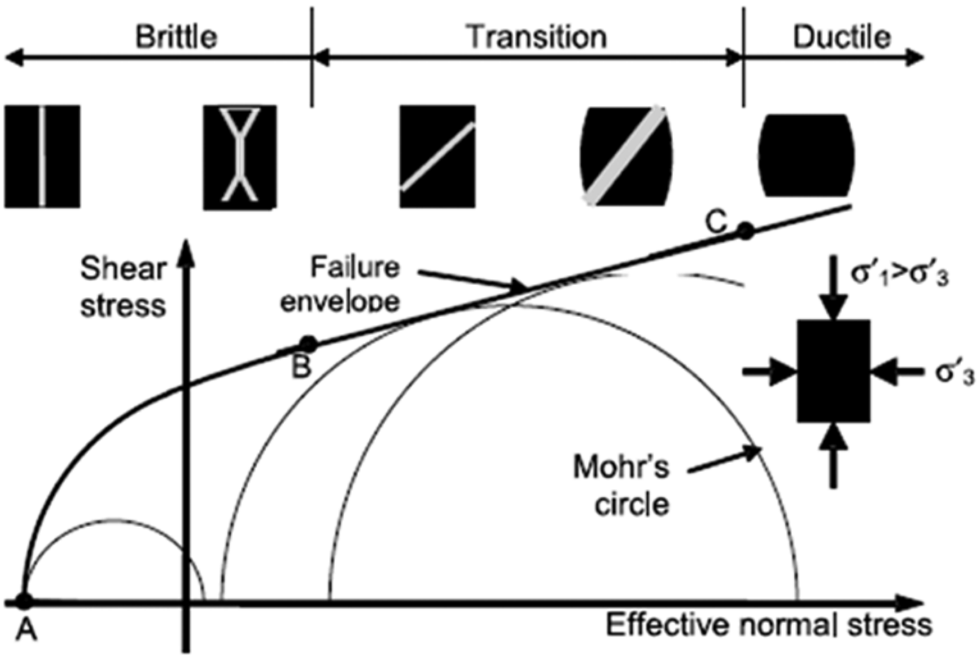

Figure 9: Effect of increasing effective confining pressures on the failure modes and fracturing of sedimentary rocks. Reproduced from Nygard et al. (2006) with kind permission from Elsevier.

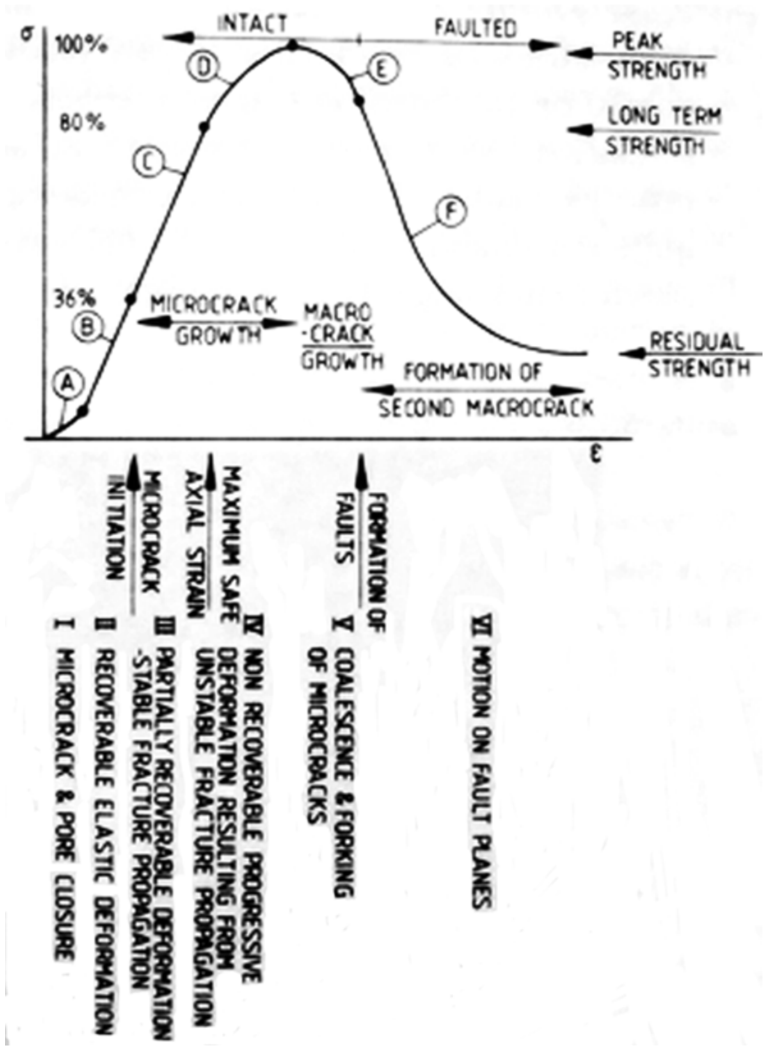

Figure 10: A description of rock deformation in UCS test (after Price 1979), taken from (Farmer, 1983)

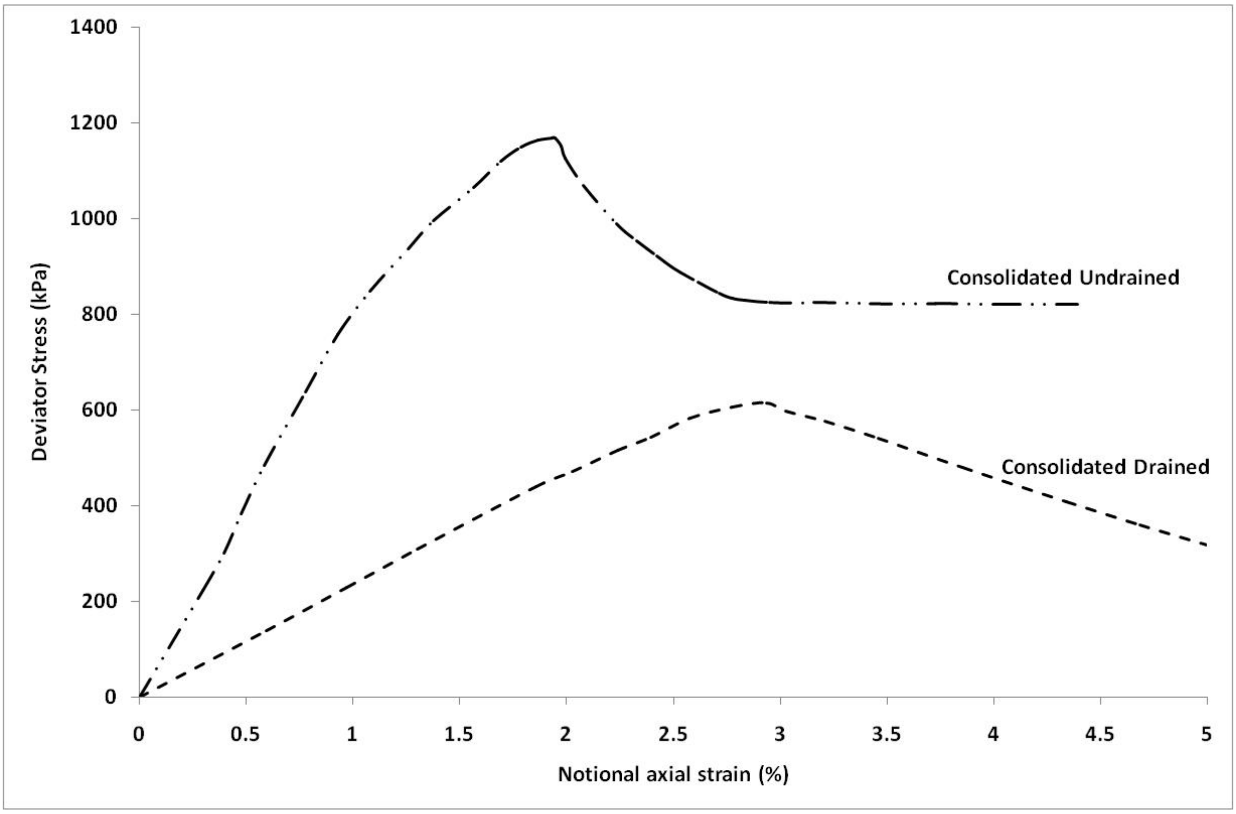

Figure 11: Typical consolidated undrained and consolidated drained tests on intact London Clay from Ashford Common (Level E (at 34.8 m depth): plastic index= 27%, liquid index= 70%, natural water content= 23.89%, effective overburden pressure= 386 kPa, K_0= 2.1) after Burland (1990)

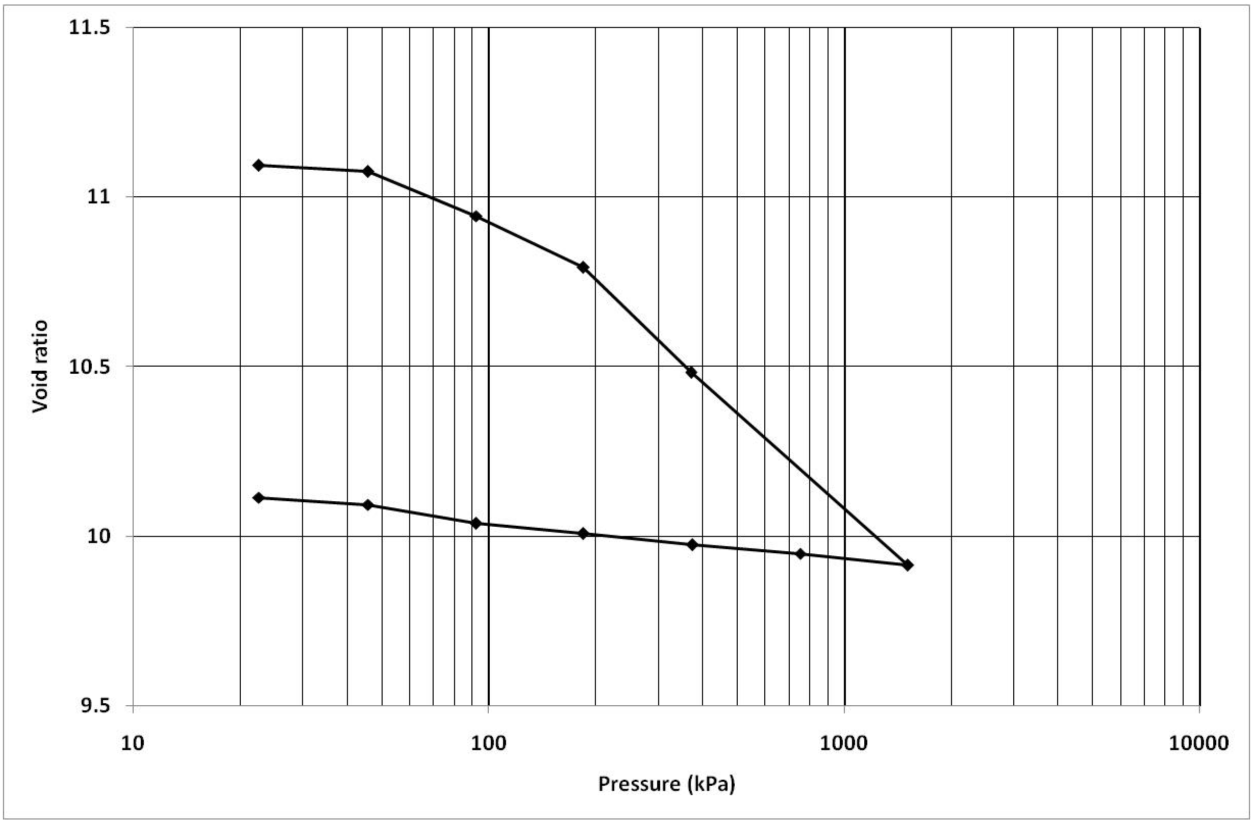

Figure 12: Consolidation behaviour of a CB mixture at 15 months age of curing (contains 15% cementitious material with 75% slag replacement). Reproduced from Opdyke and Evans (2005) with kind permission from ASCE.

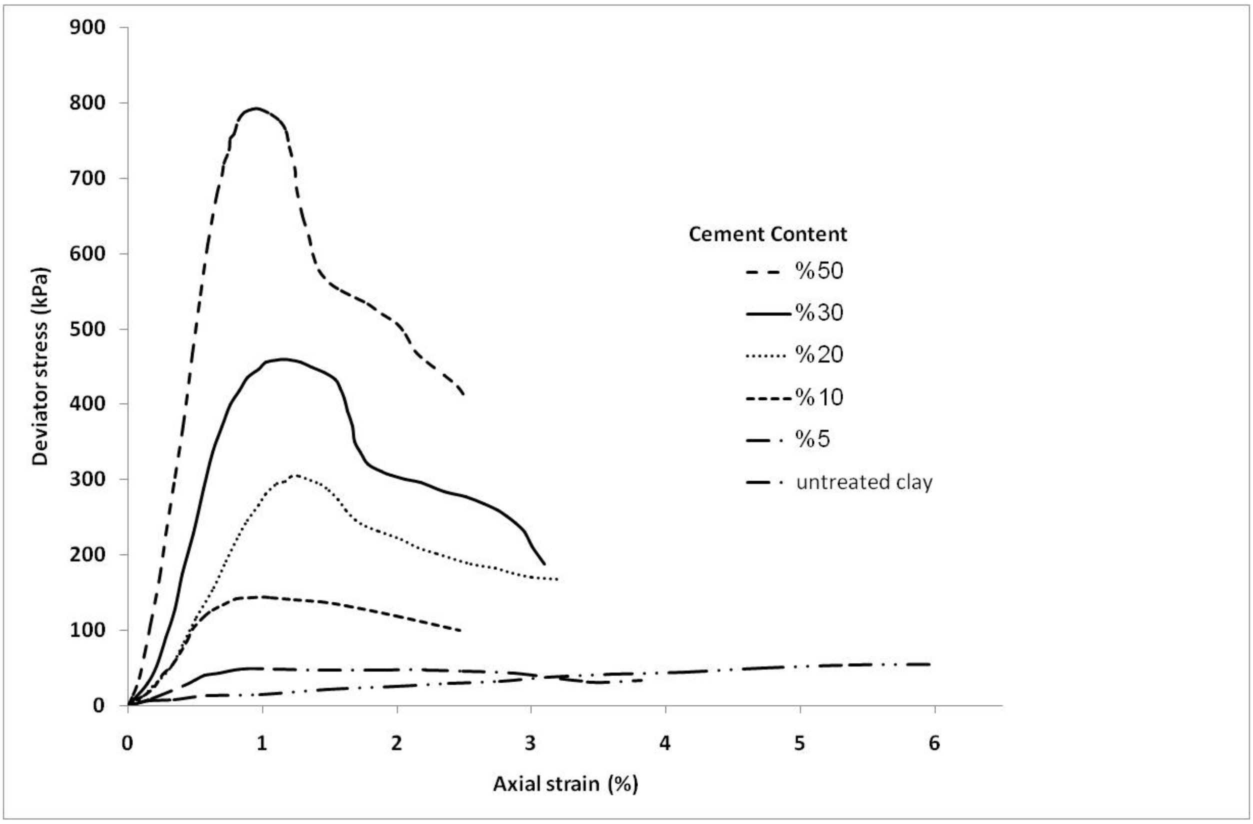

Figure 13: Stress-strain relationships of cement treated soft clays at varying cement content at 28 curing days. Reproduced from Chew et al. (2004) with kind permission from ASCE.

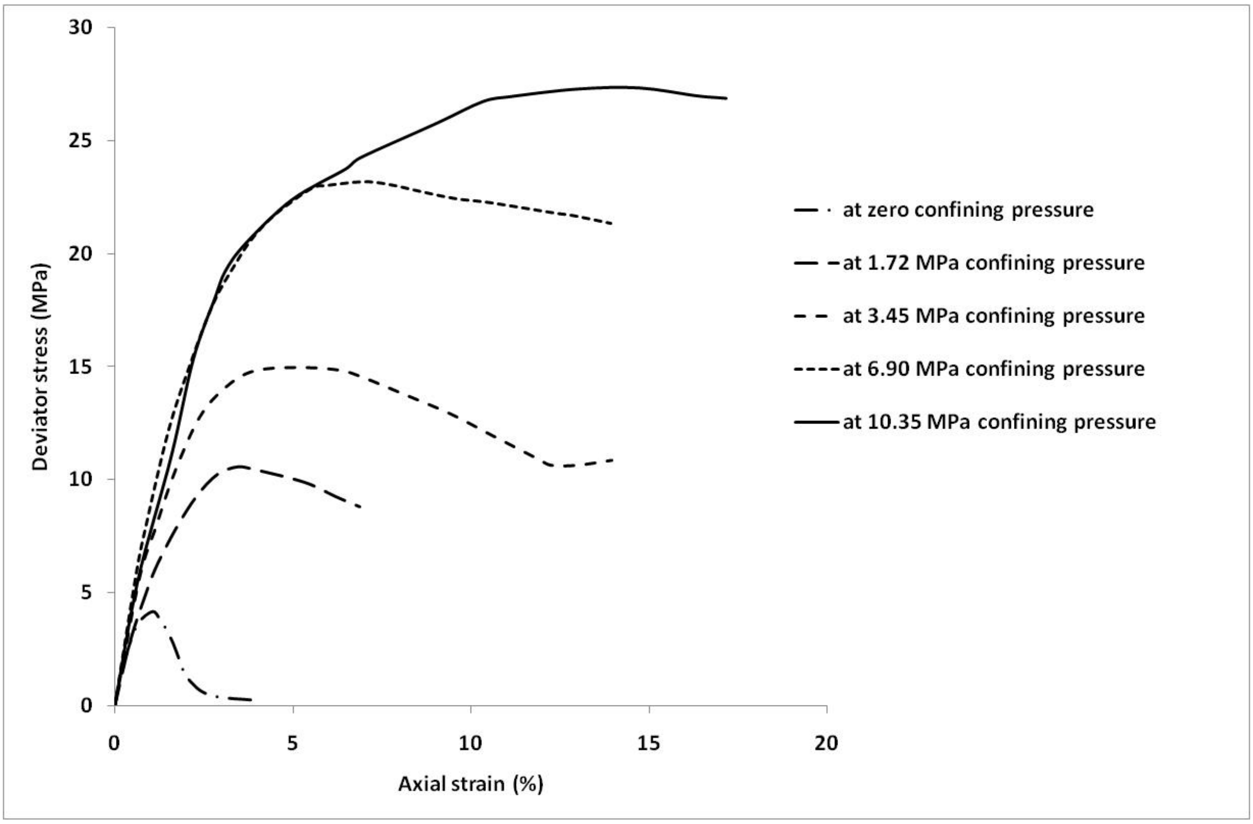

Figure 14: stress-strain relationship for cemented soil in consolidated drained triaxial testing (cement content = 6% by weight). Reproduced from Lade and Overton (1989) with kind permission from ASCE.

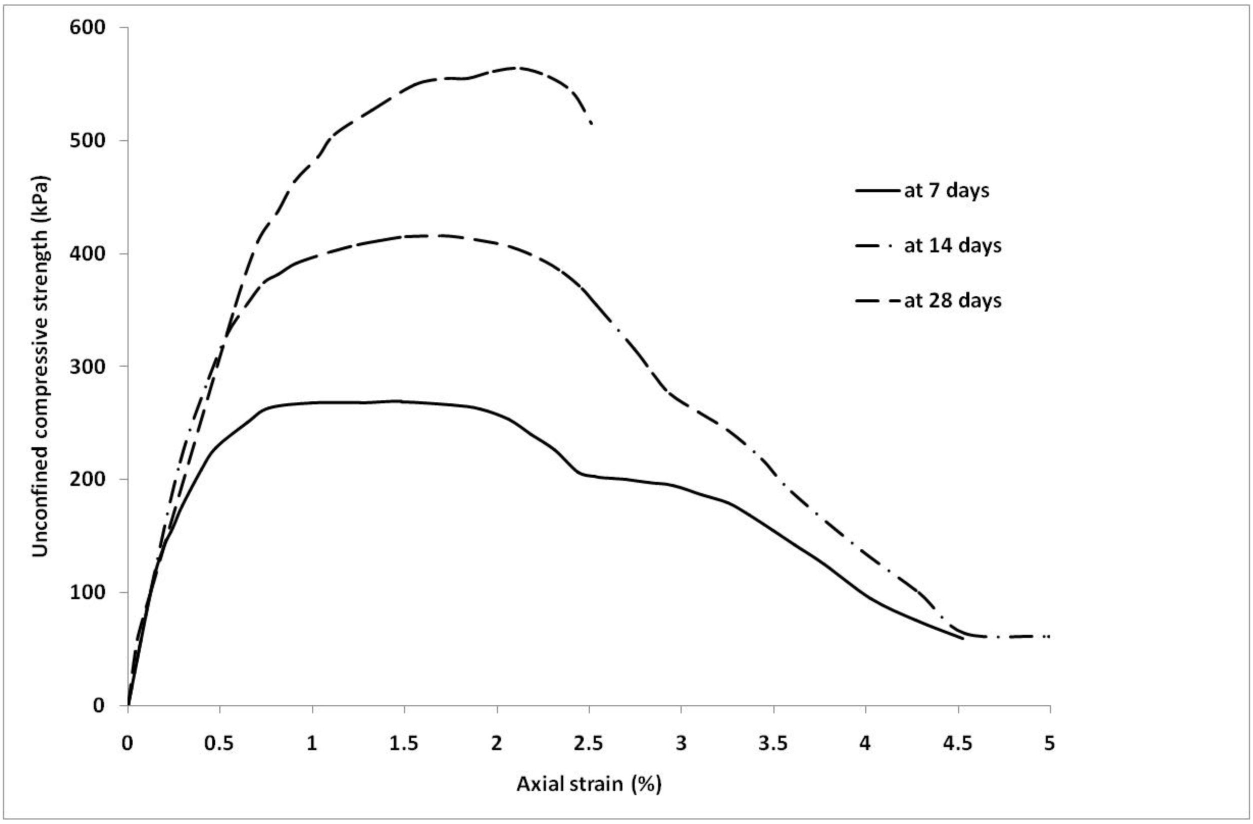

Figure 15: Stress-Strain behaviour in UCS tests for cement-stabilised kaolin at 5% cement ratio, humid cured for 7, 14, and 28 days. Reproduced from Lee and Lee (2002) with kind permission of Spinger Science+Business Media.

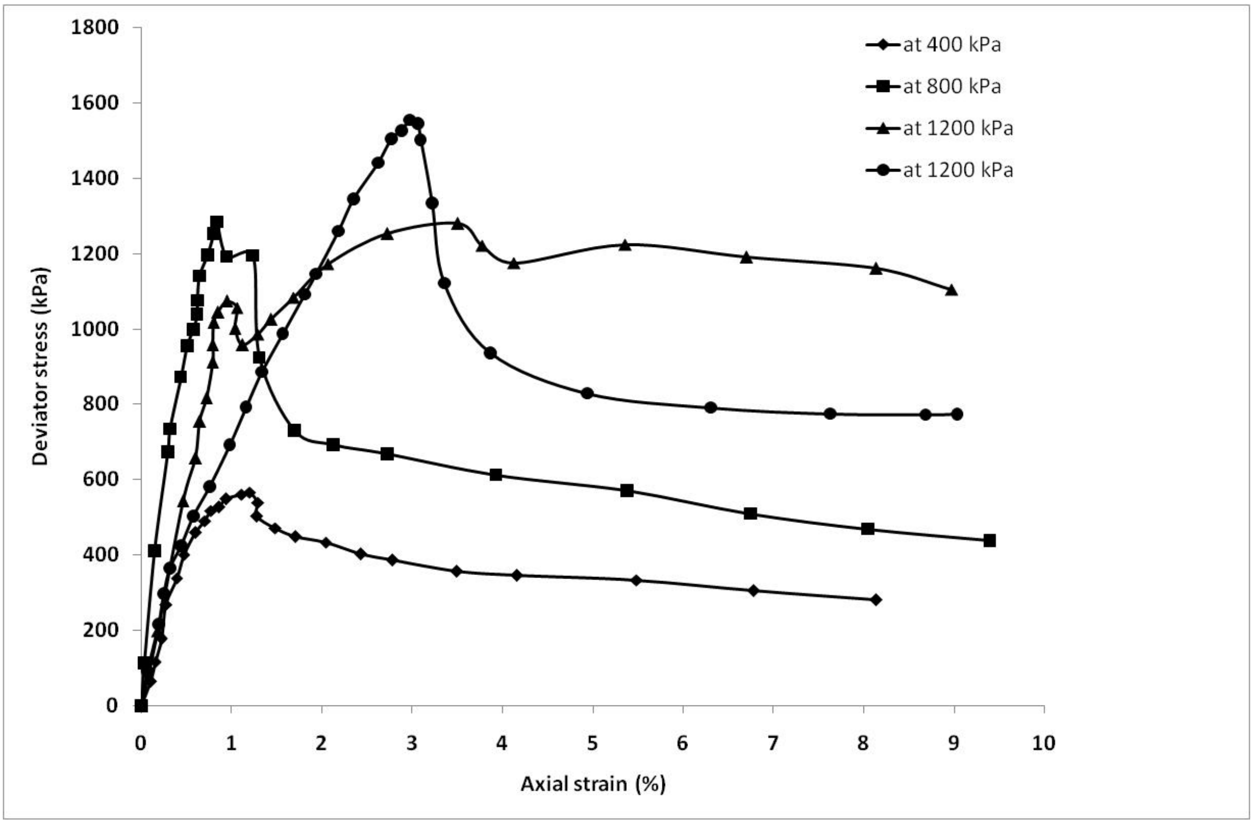

Figure 16: Stress-strain relationships of undisturbed saturated specimens of Miocene clay of Western Bohemia in consolidated undrained triaxial tests (plasticity index of about 38%, index of colloid activity of 0.6 to 0.8, consolidation cell pressure 400 to 1200 kPa) after Feda and Herle, (1993)

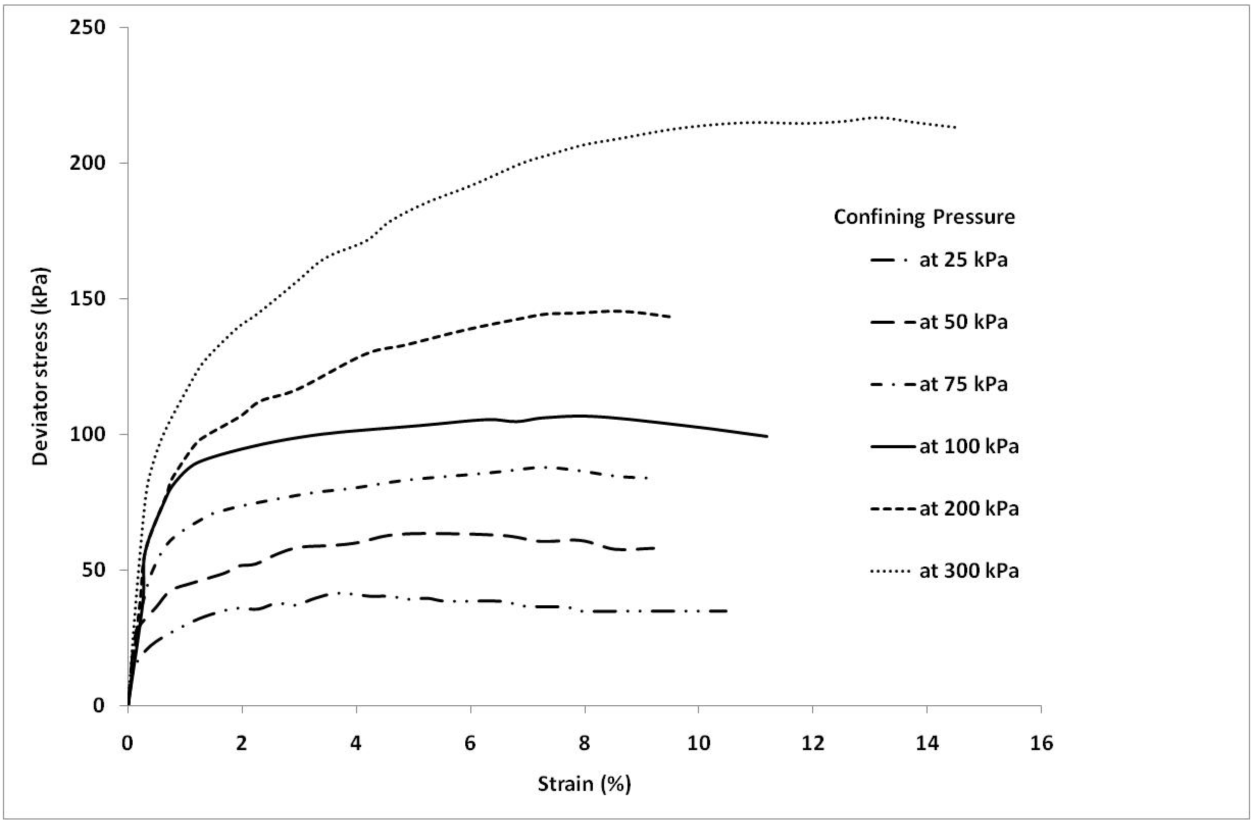

Figure 17: stress-strain relationship for undisturbed soft clay in undrained triaxial testing (natural moisture content= 80%-85%, plastic limit= 28%, liquid limit= 88%, clay= 54%, silt= 46%, preconsolidation pressure=75 kPa). Reproduced from Moses et al. (2003) with kind permission from Elsevier.

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Table 1: Nominal compressive strength of concrete and maximum strain at failure. Reproduced from Neville (1995) with kind permission from Pearson Education Limited.

Maximum Strain at Failure (10^{-3})	Nominal Compressive strength(*10^3) (KN/m^2)
4.5	7
4	14
3	35
2	70