

The mechanical behaviour of a rapidly cementing artificial soil at low confining stresses

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The behaviour of an artificially cemented clay was investigated physically and mechanically at relatively low confining stress. The study was conducted using both oedometer and triaxial tests. The results show that for a combination of β -hemihydrate and kaolin can reproduce the volumetric collapse present in naturally structured soil. Tests were performed on the material at a range of consolidation pressures and at different strain-rates. It was shown that the shear strength behaviour of the material exhibits complex characteristics that require explicit consideration of soil structure at the loading stage. Similar to other natural clays, critical state could not be achieved in the triaxial apparatus, rather, it was shown that a post-rupture state was achieved with soil states tending towards the intrinsic state defined by the reconstituted material.

KEYWORDS: structured clay; compressibility; artificial cementation; post-rupture

INTRODUCTION

The effects of soil structuration on the shear strength and deformation behaviour have been a topic of significant research for many years. Soil structure has been investigated for different natural clays on an individual basis due to the variability of natural samples and associated difficulty in soil sampling. Works such as Burland (1990), Leroueil & Vaughan (1990), Cotecchia & Chandler (1997), Gasparre *et al.* (2007) and Leroueil & Hight (2003) have considered the effects of structure on soil behaviour of both natural and reconstituted clays in order to identify common features that can be used to quantify the degree of structuration, intact behaviour, remoulded strength, and, the evolution of structure with both volumetric and shear deformation.

For a number of natural soils, structuration is imparted to them during their geological life through cementation and bonding between granular particulates (e.g. Airey, 1993; Carter *et al.*, 1988; Cuccovillo & Coop, 1999). In these naturally cemented soils, reductions in both peak strength and initial stiffness are often attributed to the disturbance of interparticle contacts and the breakage of cement bonds (Rotta *et al.*, 2003). However, identifying the extent of the structuration can be difficult as in such cases, the interpretation of experimental investigations can be compromised by disturbance to the structure during the sampling process (e.g. Clayton *et al.*, 1992).

It is also common that naturally structured soils, depending on their geological origin, can present a high spatial variability in both the degree of cementation and mineralogy (e.g. Leroueil & Vaughan, 1990). To understand the effects of structure and avoid the complications caused by natural variability most research has focused on the response of artificially cemented soils, where the behaviour of highly structured soil is replicated through the addition of a cementing agent, such as rapid-hardening Portland (e.g. Coop & Atkinson, 1993; Huang & Airey, 1998; Consoli *et al.*, 2000; Rotta *et al.*, 2003; Schnaid *et al.*, 2001; Dalla Rosa *et al.*, 2008; Xiao *et al.*, 2014). As originally proposed by Lambe & Whitman (1969), structure can be used to describe aspects of soil response which may be caused by "fabric", the arrangement of the component particles,

and "bonding", interparticle forces not of a purely frictional nature. Using the above definition of structure, the behaviour of artificially cemented soils has broadly been understood to be a result of bonding. The differences of fabric between samples tested are assumed to be negligible as all specimens were prepared in a similar manner.

The essential characteristics of a structured soil, relative to its intrinsic (reconstituted) state, are mobilised resistance, peak friction angle and greater compressibility upon virgin yielding. For both cemented sands and structured clay (ranging from sensitive to stiff overconsolidated), soil structure has similar effects on the mechanical responses, however, the magnitude of these effects can vary significantly with soil and cementing agent type. As such, general frameworks have been proposed to model the behaviour of soils which explicitly define the contribution of sensitivity, a parameter defining the degree of structuration present in the intact specimen compared to its reconstituted state (e.g. Leroueil & Vaughan, 1990; Cuccovillo & Coop, 1999; Cotecchia & Chandler, 2000; Liu & Carter, 2000).

As many of the existing models that incorporate soil structure are built upon critical state soil mechanics (CSSM) (Roscoe *et al.*, 1958), they inherit many of the limitations present in the original model (e.g. see Leroueil & Hight, 2003). The key limitation of CSSM is that it considers the shear behaviour of reconstituted soils as being controlled by a combination of specific volume, ν , mean effective stress, p' , and, deviator stress, q , relative to the critical state. For many of natural soils that exhibit structure, their behaviour during consolidation and shearing do conform to a model built upon reconstituted soils, that is, the final state of the soil, both in $\nu - \ln p'$ and $q - p'$ space, are also controlled by the structure present in the soil at moment of shearing and may not collapse into a single critical state (e.g. Burland, 1990). For structured soils, it is important therefore to investigate the fundamental interaction between structure and soil state.

Despite the large amount of literature on naturally and artificially cemented soils, there have been few systematic experimental studies on the behaviour of weakly cemented soils. This is especially true for cemented soils that yield at relatively low confining stress (<100 kPa). Natural soils that yield in this stress range are typically classified as "soft" and have been shown to be significantly influenced by sample quality (e.g. Burland, 1990; Hight *et al.*, 1992). Soft soils are commonly encountered in recently deposited

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coastal regions and pose many challenges for geotechnical engineers (e.g. [Mitchell & Soga, 2005](#)). Owing to the difficulty of obtaining large quantities of samples with similar degrees of structuration, this Paper will investigate the behaviour of an artificially cemented clay. Unlike previous studies involving artificially cemented soils, this research focuses on simulating the behaviour of a weakly structured/soft soil.

An initial batch of sixteen oedometer tests were carried out to first investigate the effect of cement content on the one-dimensional behaviour of the artificially soil. After determining the most appropriate sample composition, a further nineteen compression tests were carried out to characterise a single composition for a range of isotropic confining stresses and strain-rates. The objective of this research is, therefore, to contribute to the existing framework by providing new data on the evolution of soil structure at (a) different stages of destructure and (b) differing strain-rates for a weakly cemented soil.

The compressive response of the artificially structured soils is then evaluated in terms of both the standard CSSM framework and also following normalisation. It will be shown that the compressive behaviour fits within the modified framework proposed by [Cotecchia & Chandler \(2000\)](#).

EXPERIMENTAL PROGRAMME

Materials tested

The constituent materials used in the present work are commercially available kaolin (Q38) and dental gypsum (calcium sulphate). Both these materials are stored in powdered form and kept in a dry room. The kaolin is classified as an inorganic silt (ML) according to the Unified Soil Classification System. The kaolin powder is completely composed of grains smaller than 0.06mm and has a nominal size, D_{50} , of 0.03mm (performed to AS 1289.3.6.1 and AS 1289.3.6.3). The Atterberg limits of the kaolin material are liquid limit of 44 (AS 1289.3.2.1) and and plastic limit of 15 (AS 1289.3.9.1). The specific gravity of the solids is 2.64 (ASTM D 854).

The unhydrated cementing agent was determined to be β -hemihydrate ($\text{CaSO}_4 \cdot 1/2\text{H}_2\text{O}$) powder. Standard approaches to classifying the unhydrated material are not possible as the addition of water would initiate the crystallisation of the calcium sulphate minerals to form gypsum. In all the mechanical tests on both reconstituted and cemented specimens CaSO_4 is present in the form of gypsum. Classification tests on gypsum were performed after preparing the material as follows: (i) manufacturing samples of pure cemented gypsum, (ii) drying the samples in a temperature controlled oven, (iii) crushing the dried sample in a mortar and pestle. This material can then be characterised following the Standards without further alteration to the chemical composition. For the purposes of classification and determining the end of test moisture content, samples were dried in oven kept at under 100°C. At these temperatures, evaporation would result in the removal of pore fluid instead of water molecules from the gypsum's internal structure ([Taylor et al., 1997](#); [Sadeghiamirshahidi & Vitton, 2019](#)). The liquid limit of the portion of reconstituted material passing a no. 50 sieve is 62; no plastic limit was found. The specific gravity of the solids was determined to be 2.39.

It is worth noting that the material needed to produce reconstituted samples of gypsum kaolin mixtures is obtained in a similar manner to that described for producing reconstituted gypsum. However, instead of drying and crushing samples of pure gypsum, end of test specimens (dried triaxial samples) are used.

Scanning electron microscope

The nature of cementation and crystalline interactions between the kaolin and gypsiferous minerals is investigated by means of scanning electron microscope (SEM) imaging. SEM images were taken for samples of (a) kaolin, (b) β -hemihydrate, and, (c) a 1:1 mixture ratio kaolin-gypsum in their dry state, as shown in Figure 1, and following hydration, as shown in Figure 2. The images shown were taken of dry powdered material placed onto the imaging plate. SEM analysis was completed at the Australian Centre for Microscopy and Microanalysis facility at the University of Sydney, Camperdown Campus using a Quanta SEM 200. A detailed description of the process as well as additional SEM, environmental SEM, back-scatter SEM images (BSEM) can be found in [Le \(2018\)](#).

It is found that at high magnification, the dry kaolin powder (Figure 1a) is seen to contain a large fraction of silt sized aggregates ($<40 \mu\text{m}$) with some finer clay sized particles being evident between the larger clusters. Lower magnification images however show a uniform distribution of the kaolin particles. From hydrometer test results, it is believed that the silt sized grains are clusters of stacked kaolin particles that disperse upon wetting. From the SEM image, the clay particles are shown to be more platy, with an aspect ratio of approximately 10.

At low magnification, the unhydrated sample of hemihydrate is seen to be uniformly angular with a distinct fibrous texture, a result of its single perfect and imperfect cleave plane ([Howie et al., 1992](#)). At higher magnification (Figure 1b), the material is found to contain predominantly silt sized particles with a small amount of finer grains attached to the smaller silt sized particles (as marked in the Figure). These finer grains are believed to be precipitated gypsum crystals which have been hydrated by atmospheric moisture. The aspect ratios of the hemihydrate grains are found to range between 1 and 12.

At low magnification, the dry mixture of kaolin and hemihydrate appears to be a homogeneous mixture of the two raw materials. At higher magnification (Figure 1c), the hemihydrate crystals and kaolin grains appear to be uniformly dispersed throughout the image. Most importantly, the unhydrated hemihydrate is in direct contact with both the silt and clay sized particles of kaolin soil and there is no obvious bonding between the two soils.

Following hydration, SEM imaging was also performed on the raw material in order to observe the bonding between gypsiferous minerals with surrounding soil. For clarity of the microstructure present in a typical sample, imaging was taken on the surface of uniform thin samples produced by spreading a thin layer of the material over polyvinylidene chloride (PVDC) film and allowed to cure. Due to trace amounts of water present in the samples during preparation for high-vacuum imaging, slightly lower quality images were captured. This is a result of the electric charge left during gold coating.

As expected, SEM imaging of hydrated kaolin (Figure 2a), reveal no microstructure aside from the presence of occasional silt sized grains. These are believed to be clusters of kaolin that remain from the dry powder.

Calcium sulphate (Figure 2b) was found to change significantly with hydration. The image shows that the surface texture is made of angular, interlocking crystals of gypsum with regions of significant bonding between neighbouring particles. As such, the microstructure was able to maintain an open fabric. It should also be noted that the hydrated calcium sulphate appear less acicular than before hydration; this would suggest that there has been significant growth of smaller crystals during hydration.

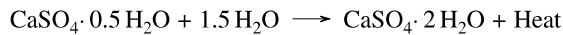
At a 1:1 ratio of kaolin to β -hemihydrate by dry mass, the hydrated material (Figure 2c) shows a much denser fabric, compared to the pure gypsum, with a distribution of gypsum crystals within a kaolin matrix. At higher magnification and by utilizing BSEM (where surface level electrical charge does not significantly reduce the quality of captured images), it was found that the microstructure of gypsum cemented kaolin samples show (i) a relatively random arrangement of clayey particles, (ii) the gypsum minerals are formed with kaolin particles surrounding the large crystal arrangements, and, (iii) small local voids can be found ranging in size, often 1–2 μm (Le, 2018).

Sample preparation, curing and testing program

All specimens were prepared by initially mixing predetermined quantities of dry kaolin and unhydrated β -hemihydrate; water was then added to the dry mixture and was folded into the soil until a uniform consistency was achieved. The initial conditions of the test specimen were found to be affected by the water requirements of the cementitious material, that is, the final pore structure and moisture content were affected by the hydration of the β -hemihydrate.

Lewry & Williamson (1994) found that when gypsum plaster was prevented from drying out completely, the strength rapidly increase to a maximum value and remained constant no further changes with time; the authors categorise these specimens as “wet” and noted that they were less affected by the duration of curing. The ultimate strength of the β -hemihydrate samples used in this study were achieved in approximately 2 hours time.

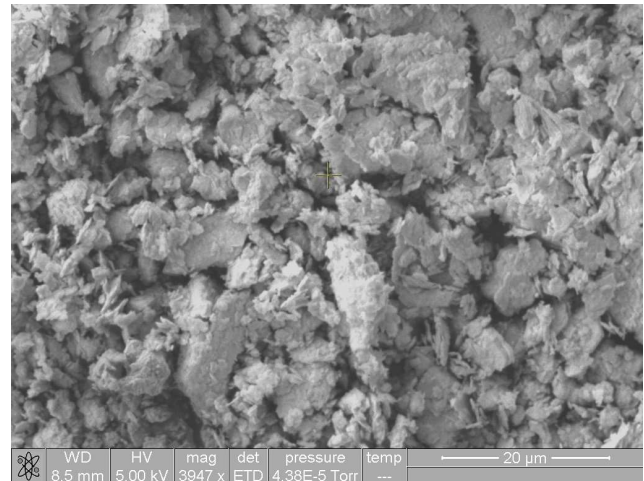
The hydration of hemihydrate can be described as:



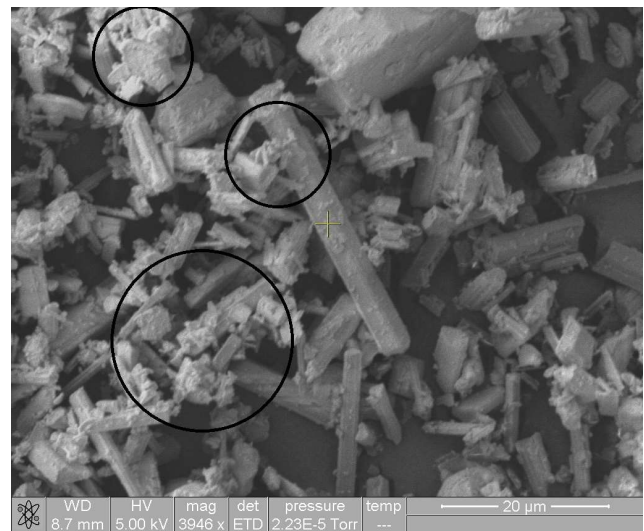
The above equation indicates that 100.0g of hemihydrate will react with 18.3g of water to produce 118.3 grams of dihydrate. Thus a water:hemihydrate ratio of 0.183 is theoretically required to supply the necessary water for complete hydration. Any additional water will be taken up by the soil mass and contributes to its water content.

With respect to the preparation of artificially cemented samples, the current study focuses on (i) the evolution of soil compressibility with vertical stress/isotropic confining pressure and (ii) sample reproducibility. As the objective is to manufacture samples replicating the behaviour of a weakly structured (soft) soil, such as Ballina clay (Pineda *et al.*, 2016) or Bothkennar clay (Smith *et al.*, 1992), it was first necessary to determine the influence of hemihydrate content on (i) initial compressibility, (ii) nature of yielding and (iii) rate of destructuration. A range of manual oedometer tests (up to 576kPa) and high-stress manual oedometer tests (up to 4MPa) with varying cement and water content were performed as listed in Table 1. As the cemented soil was selected to have a relatively low yield stress, a state of complete destruction was anticipated to be achieved by 4MPa.

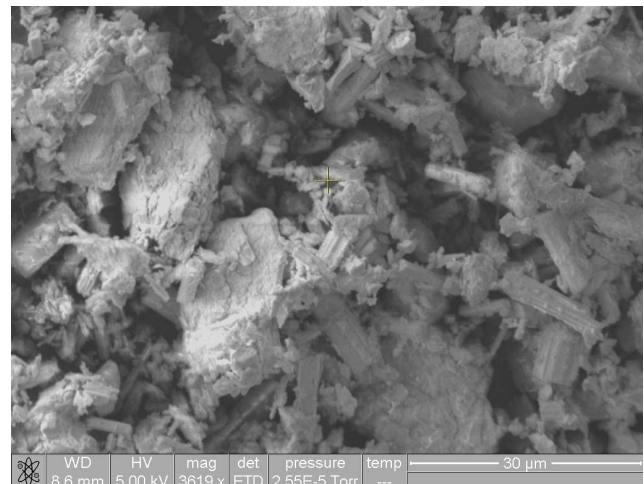
Oedometer samples were prepared by dry mixing the raw material (β -hemihydrate and kaolin powder) and folding in water equalling 60% of the dry soil mass, except for oGK07 where 90% was added. Samples were formed directly in the cell and allowed to cure before testing. The reduction in initial moisture content before saturation is noted to be a result of hydration in the hemihydrate crystals. The saturated conditions are obtained after allowing the sample to sit in a water bath under a small stress imposed by the top-cap prior to testing. This preventing further drying of the material and allows the water to permeate through the larger air voids present in the sample. The observed variations in the saturated moisture contents are believed to be a result sample variability. This is



(a)



(b)

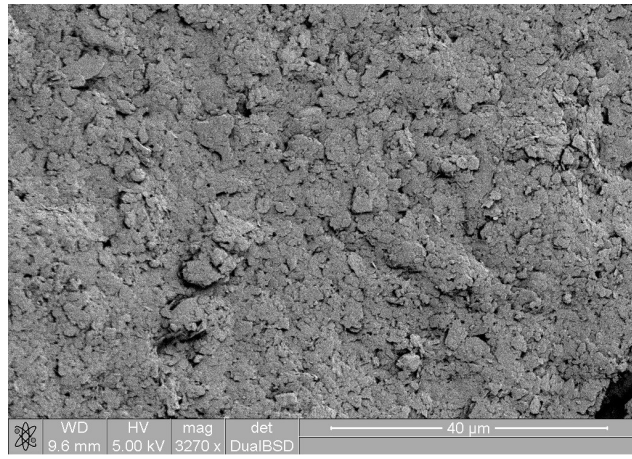


(c)

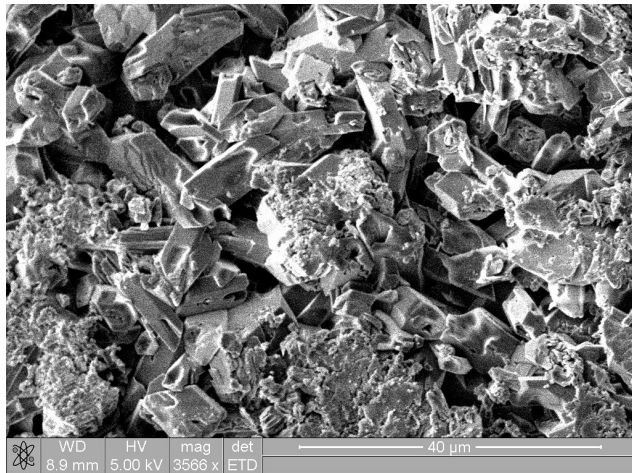
Fig. 1. Typical SEM images of unhydrated (a) kaolin, (b) β -hemihydrate, and, (c) kaolin-gypsum (1:1 ratio)

noted to occur more prominently in oedometer samples due to the small specimen volumes.

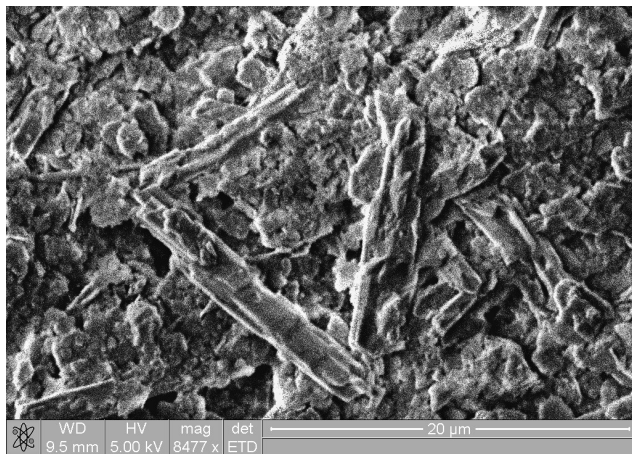
Results from oedometer tests were used to select an appropriate hemihydrate percentage and moisture content combination to produce a “standard” soft soil sample. This



(a)



(b)



(c)

Fig. 2. Typical SEM images of hydrated (a) kaolin, (b) gypsum, and, (c) kaolin-gypsum (1:1 ratio)

requires that the sample must exhibit relatively high stiffness up until yielding, with yielding occurring to the right of the reconstituted normal consolidation line (NCL) of the same composition and void ratio, whereupon the sample would collapse towards a reconsolidated NCL with further loading. 30% hemihydrate was chosen due to the strong tendency of samples to destructure once yielded. Further, 60% w/w added water was selected for mixing as further increases in the amount of water reduced the effect of destructure at yield. That

Table 1. Summary of experimental programme: oedometer

Test	β -hemihydrate content (w/w) [%]	initial m_c [%]	sat m_c [%]	e_i
oGK01	0	60	70	1.82
oGK02	10	51	55	1.45
oGK03	20	49	56	1.46
oGK04	30	60	62	1.59
oGK05	30	51	59	1.52
oGK06	30	48	59	1.55
oGK07 [‡]	30	48	51	1.31
oGK08	40	45	49	1.24
HSoGK01	0	60	68	1.76
HSoGK02	5	60	65	1.70
HSoGK03	10	60	66	1.72
HSoGK04	20	56	63	1.63
HSoGK05	30	59	61	1.56
HSoGK06	40	56	56	1.41
HSoGK07	50	46	51	1.28
HSoGKr01	30	60	60	1.54

[‡] 90% water by dry mass was added during preparation

is, the compression curve translated closer to the reconstituted curve with increase moisture content during mixing. This interpretation is however limited due to the there only being one sample tested at this higher moisture content.

For isotropic compression tests, a total of nineteen (19) triaxial tests were performed on artificially cemented samples sheared undrained at a range of confining stress and strain rates as listed in Table 2. Specimens were mixed and carefully transferred into a 50mm diameter by 200mm high mould (entrapped air was minimised by tamping mould during transfer) and left to cure until firm enough to handle (approximately 30–40 minutes). The sample was then vertically extruded from the moulded and ends trimmed using a split mould with dimensions of the final sample. They were placed in the triaxial cell and submitted to a 10kPa effective confining stress. The samples allowed to saturated by ramping the cell and back pressure. Following the ramp stage, samples were left cure, with the back pressure lines open, for 48 hours prior to performing a b-value test. It was found that a b-value of at least 0.95 can be consistently achieved by ramping the both the cell and back pressures by 500kPa.

Isotropic stresses were applied by slowly ramping the cell pressure (1kPa/hour) until the final stress state, except for those noted in Table 2. Assuming one-dimensional consolidation theory, this was sufficiently slow to avoid significant build up in excess pore pressure for a two-way drainage system. Prior to shearing, samples were left to equilibrate for at least 24 hours to minimise any possible build up of pore pressure in subsequent stages. Volumetric strain rates were negligible by the end of the equilibration stage.

In order to understand the mechanical development of strength in the artificially structured soils, a number of samples were placed into the triaxial apparatus immediately after wet mixing and then allowed to cure while bender element readings were performed. Continuous monitoring of the shear wave velocity was achieved using an automatic script with Fast-Fourier Transformation (Airey & Mohsin, 2013). The samples were supported by the mould but no stresses were applied during curing.

TEST RESULTS AND ANALYSIS

Oedometer tests

Sixteen oedometer tests were performed on kaolin specimens with varying percentages of hemihydrate by dry weight (w/w).

Table 2. Summary of experimental programme: triaxial

Sample	Test	e_i	p'_0 [kPa]	$\dot{\epsilon}_a$ [%/hr]	ϕ'_{peak}	ϕ'_{pr}
GKP01	CIU	1.55	10	0.32	TC	45
GKP02	CIU	1.59	10	3.24	TC	45
GKP03	CIU	1.67	15	0.29	TC	45
GKP04	CIU	1.61	30	0.32	67	43
GKP05	CIU	1.65	50	0.32	60	36
GKP06	CIU	1.63	75	0.18	57	—
GKP07	CIU	1.64	120	0.14	43	32
GKP08†	CIU	1.68	40	0.29	43	32
GKP09*	CIU	1.63	110	99.7	50	38
GKU01	CIU	1.65	40	0.10	52	35
GKU02	CIU	1.58	40	1.00	57	36
GKU03*	CIU	1.56	40	10.0	57	34
GKU04	CIU	1.61	40	0.50	67	—
GKU05	CIU	1.62	40	5.00	60	40
GKU06	CIU	1.60	40	5.00	56	33
GKU07	CIU	1.60	40	5.00	55	32
GKU08	CIU	1.58	40	50.0	67	36
rGK1*	CIU	1.29	40	0.60	—	35
rGK2	CIU	1.31	50	0.60	—	37

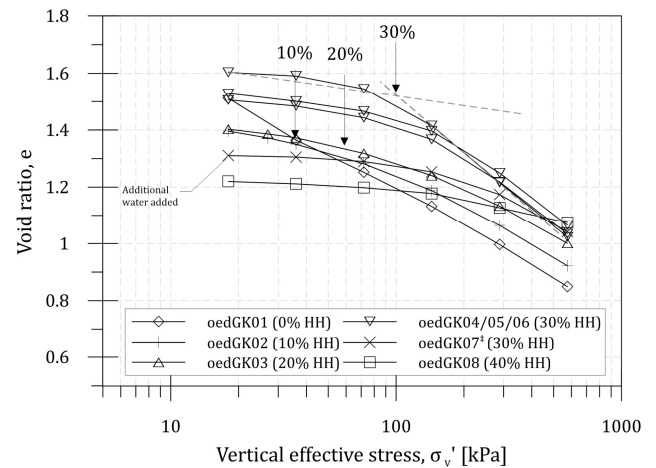
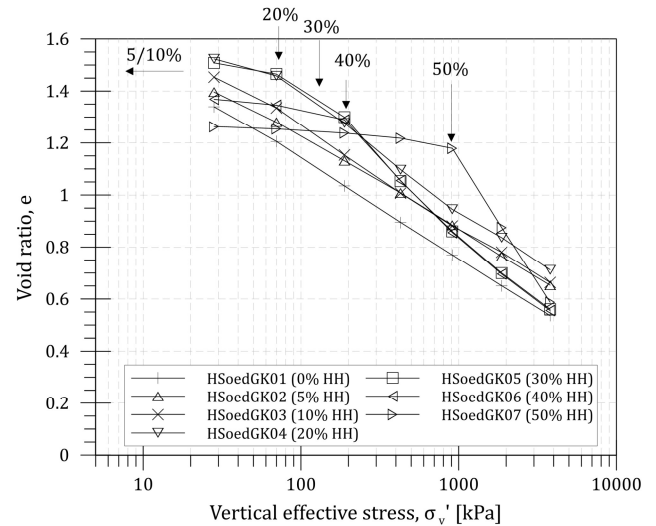
†OCR=4

*Single stage increment consolidation

Figures 3 and 4 present the one-dimensional compression curves for artificially cemented gypsum-kaolin specimens using the manual oedometer apparatus. Tests shown in Figure 4 were loaded to 4MPa. Hemihydrate dry mass ranges between 0-40% for the manual and 0-50% for the high stress apparatus. For both sets of results, a strong correlation can be seen between the percentage of hemihydrate and apparent yield stress in one dimensional compression. As all samples were mixed from dry and formed directly in the oedometer apparatus, no effective stress was applied before loading. The yielding of the soil is therefore a result of bonding within the microstructure and not mechanical overconsolidation.

The increase of hemihydrate content is also observed to impact the initial stiffness of the sample. Results indicate that with increasing cement content, compressibility prior to yielding decreases. The coefficient of volume compressibility, m_v , against vertical effective stress for the high stress oedometer samples are plotted in Figure 6. Similar to results for many naturally structured clays (e.g. [Leroueil & Hight, 2003](#)), the compressibility increases immediately following yielding are seen to be greater than the normal compressibility of the reconstituted sample and the compressibility of the cemented samples at high stress (approx. 1000kPa). An approximate one-dimensional yield stress are marked for each sample. The compressibility of different soils with varying cement content is shown to tend towards a similar coefficient compressibility curve traced by the uncemented sample with further loading.

Changes in composition are shown to influence the NCL location. This is believed to be a result of both the change in soil mineralogy and microstructure. Reconstituted specimens of 30% hemihydrate were repeated in order to provide a reference “unstructured” NCL line. Figure 5 plots the compression curves for the structured 30% hemihydrate and reconstituted samples with 0% and 30% hemihydrate content. The void ratio of the structured soil was found to be greater than for a reconstituted sample at equivalent vertical effective stress. This has been conceptually associated with structure ([Leroueil & Vaughan, 1990](#)) and the difference in voids ratio, Δe , can be used in modelling of structured clay (e.g. [Liu & Carter, 1999](#)). This additional, structure associated, void ratio reduces as stress

**Fig. 3. Manual oedometer test results for varying amount of hemihydrate content****Fig. 4. Manual oedometer test results for varying amount of hemihydrate content**

level increases, becoming insignificant at σ_v' approximately 1 MPa.

It is shown that for this selection of gypsum cement content, the behaviour is similar to that found in naturally structured soils. That is, there is a gradual decrease in additional voids ratio above the reconstituted soil with further loading beyond the yield stress. This artificial structured soil is therefore shown to closely reproduce the compressibility of naturally structured clays and sustain a greater voids ratio at an equivalent vertical effective stress compared to its reconstituted state.

Triaxial tests

Figures 7 and 8 present the results obtained from isotropic compression tests carried out on specimens consolidated to different confining stresses and for various strain rates, respectively. Results for the reconstituted samples, rGK1 and rGK2, have been included in Figures 7 for comparison. A unique post-rupture* curve (e.g. [Randolph et al., 1979](#); [Burland et al., 1996](#); [Cotecchia & Chandler, 2000](#); [Saihi et al.,](#)

*This has also been called *pseudo* or *quasi* critical state.

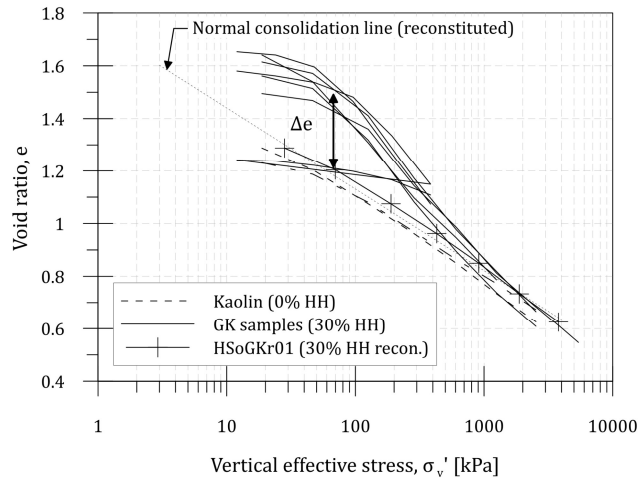


Fig. 5. Compression curves of structured and unstructured gypsum-kaolin specimens

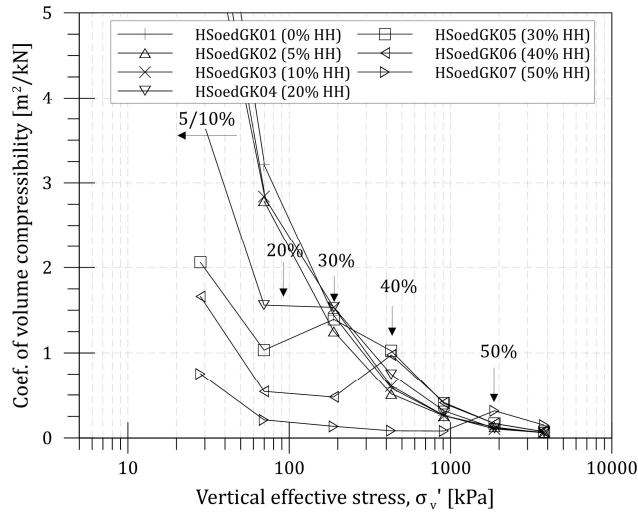


Fig. 6. Manual oedometer test results for varying amount of hemihydrate content

Table 3. State parameters for cemented and reconstituted gypsum-kaolin specimens

Sample	G _s	Γ	N	λ	κ	M _{pr}
cemented (post-rupt.)	2.55	3.37	3.61	0.249	0.05	1.418
reconstituted	2.55	2.69	2.75	0.135	—	1.418

2002) can be identified in $e - p' - q$ space and is plotted in both Figures. For the purposes of this study, post-rupture deformation is defined as the ultimate state where near-rigid body sliding occurs on the failure plane (Burland, 1990). The post-rupture (ultimate state for structured samples) and critical state parameters (ultimate state for reconstituted samples) are listed in Table 3. The normal compression line (NCL) of the reconstituted specimens found from the one-dimensional tests is noted to be consistent with that found during isotropic consolidation. Further, the NCL of the reconstituted material is measured to be less inclined than the post-rupture in $e - \ln p'$. A summary of the all triaxial test results in terms of peak and post-rupture (pr) friction angles is given in Table 2.

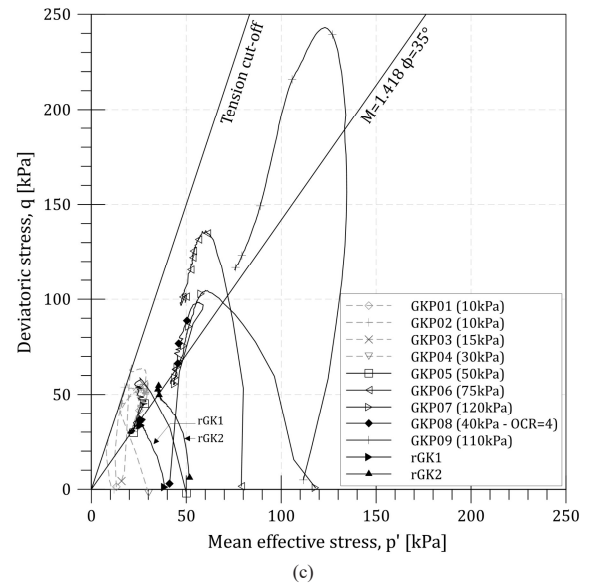
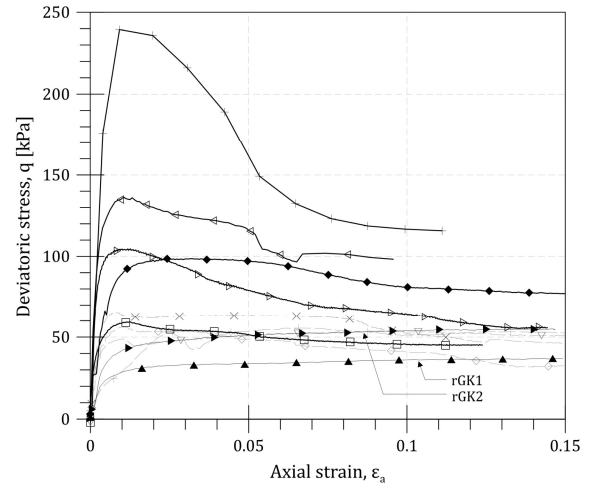
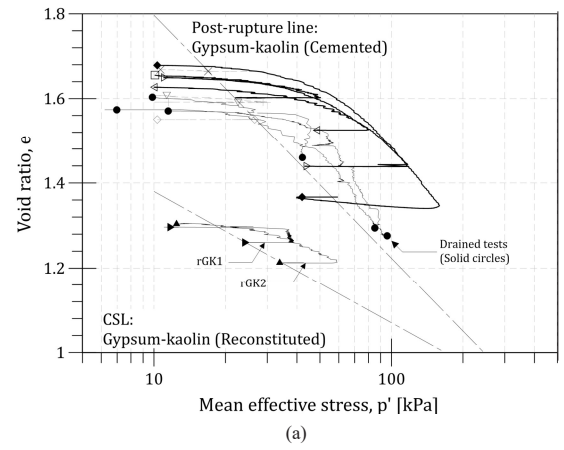


Fig. 7. Behaviour of gypsum-cemented specimens at different confining pressures

Figure 7a. plots the isotropic compression curves for samples from the curing pressure 10kPa. The first and last point of each curve are plotted in the Figure. Whilst the utmost care was taking during the preparation, there is still a scatter in the initial void ratios; this is believed to be a consequence of the hydration of gypsiferous minerals and the variability in

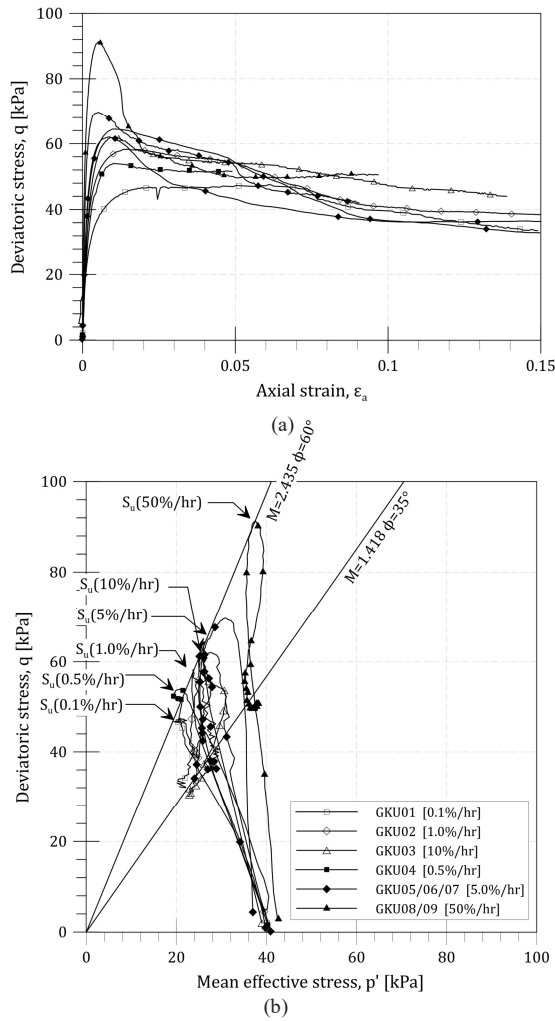


Fig. 8. Behaviour of gypsum-cement specimens sheared at different strain rates

sample cementation, as discussed previously. For the set of 18 specimens prepared, initial void ratio was found to have a mean value 1.61 with a coefficient of variation of 2.17%. Despite this however, a consistent post-rupture line is observed in $e - \ln p'$ space with $\lambda = 0.249$ and $\Gamma = 2.37$. The post-rupture slope is found to be consistent with the normal compression slope of structured specimens, in $e - \ln p'$, and greater than that found for the reconstituted material. The location of the post-rupture line is also supported by a number of drained compression tests performed on 30% hemihydrate cemented samples as shown in Figure 7a. In $e - \log p'$ plane, the stress state for the drained tests are shown to also trend towards the post-rupture line with continued strain. As this paper focuses on establishing the post-rupture line, only the void ratio response is considered. Further detail on the drained tests performed can be found in Le (2018). One interpretation of this data is that “critical state” may not have been achieved in these samples due to possible strain localisation and shear plane formation. It is likely then that the post-rupture line in $e - \log p'$ is dependent on the degree of structuration present in the sample and that a significantly structured soil may not reach a true critical state.

The compression curves plotted in Figure 5 suggest that for cemented gypsum-kaolin samples, stress states significantly greater than yield are needed to breakdown the microstructure

present in the specimens. It appears that despite the relatively low yield stress, approximately 100 kPa isotropically, stresses multiple times yield is needed to cause the structured and reconstituted compression curve to coincide. The ultimate state line in $e - \log p'$ space suggests that the slope does not reduce to its reconstituted value until more significant destructuration has occurred. This is similar to results for naturally sensitive (Saihi *et al.*, 2002), structured (Randolph *et al.*, 1979) and stiff (Cotecchia & Chandler, 1997) soil. Unfortunately data on naturally soft clays at elevated pressures in a triaxial is not currently found in the literature.

It is believed that since the stress path and response in $e - \ln p'$ space approach the post-rupture line from both sides, the behaviour indicates some intermediate “quasi-ultimate” failure where the aggregates of gypsum-clay are not completely broken down. Smith *et al.* (1992) found that, for a normally consolidated structured clay, the stress state would move towards a state boundary surface before collapsing towards an intrinsic (reconstituted) state. Due to the limitation inherent in triaxial testing, it is not however possible to reach the final intrinsic state for the whole sample. It is suggested that, instead, critical state is reached for a small section of the soil where a shear band formed.

For gypsum cemented kaolin samples and a number of examples in literature for structured soil (e.g. Randolph *et al.*, 1979; Cuccovillo & Coop, 1997; Amorosi & Rampello, 2007), undrained response depends on the degree of destructuration experienced prior to shearing. The behaviour of the cemented material can be separated into two distinct phases (i) stiff pre-yield response and (ii) post-yield structural degradation. It is believed that the stiff response of cemented material is first dominated by the bonds present in the microstructure; following a peak shear stress, these are gradually broken with further shear strain, observed in the stress-strain plot as strain-softening, until reaching an ultimate value as a shear plane develops through the sample. Peak shear stress is noted to occur at axial strains between 0.5–2.0% for the cemented samples prior to yielding. As shown in Figure 7b, for samples GKP07 – GKP09, isotropic yield results in a reduction of initial tangent shear modulus and delay in peak shear strain. This is interpreted to demonstrate a sensitivity of the soil structure to volumetric compression. The elastic shear stiffness was found to vary between 40–100MPa at very small strain and to quickly reduce at strains above 0.02%. Samples consolidated to higher isotropic effective stresses were found to exhibit lower stiffnesses throughout shearing.

Figure 7 plots the undrained effective stress paths for samples consolidated to a range of isotropic confining pressures. As listed in Table 2, three of the samples, GKP01–GKP03, reached zero effective lateral stress or tension cut-off (TC) and the results are believed to only be reliable prior to reaching this point. Samples that reached tension cut-off are interpreted as being too stiff for testing at the initial confining pressure. This behaviour is found to be consistent with other studies investigating artificial cementation (e.g. Kamruzzaman *et al.*, 2009; Xiao *et al.*, 2014). For samples which did not reach the tension-cut off, a strong trend in decreasing peak friction angle with isotropic confining stress is observed, as listed in Table 2. For Sample GKP09, which was consolidated to 160 kPa before unloading to 40 kPa, a peak friction angle of 43° was reached before reducing to an ultimate friction angle of 33°. The peak and ultimate friction angles reached is noted to be very similar to Sample GKP08 which was consolidated to 120 kPa. The set of tests to different confining stress would suggest that (i) the peak friction angle is dependent on the amount of destruction experienced prior to shearing and (ii) the ultimate friction angle is independent structure.

For samples which are consolidated to the same isotropic confining stress of 40kPa, both peak shear and effective stress path are shown to be strain-rate dependent. The increase of peak shear stress with strain-rate was found to be consistent with other structured soils reported in literature (e.g. [Graham et al., 1983](#); [Leroueil et al., 1985](#); [Liingaard et al., 2004](#)). That is, the peak undrained strength can be expressed as a power function of strain rate with the exponent being approximately 1.10. Figure 8b. plots the variation in stress-strain responses for the gypsum cemented kaolin samples sheared undrained at different monotonic strain-rates. The shearing behaviour of the samples can generally be described as brittle with the final shear strength being associated with the formation of distinct shear planes. In all samples except for GK01, the peak strength occurs at axial strains between 1-2% and drops to a non-unique ultimate shear strength at larger strains. For sample GK01, sheared at 0.1%/hr, the strain-softening behaviour is noticeably less apparent until 7% axial strain at which the sample develops a shear plane.

From the stress-strain responses, it can be said that the initial stiffness, peak shear strength and degree of strain-softening vary consistently with applied strain-rate. This suggests that the range for which the gypsum cemented kaolin shows “elastic” type behaviour is highly rate-dependent. The range of strain for which the sample shows a stiff response (elastic range) increasing strain-rate before reaching a peak and subsequently decreasing. It is shown that the rate dependency of the material is well described by [Di Benedetto & Tatsuoaka's \(1997\)](#) postulated theory of an elastic limit line. Particularly, the model explains the increase in elastic range with strain-rate. For gypsum cemented kaolin, the increased elastic range is linked to the limiting small strain stiffness as described by [Di Benedetto & Tatsuoaka \(1997\)](#) and is likely a result of the physical bonding present in the soil. It should be noted, however, that no “limit” stiffness was observed for the samples present due to the lack of reliable readings at very small strains. The scatter in both the peak and post-peak responses is believed to be associated with microstructural sample non-uniformity, natural variation in mechanical behaviour and strain localisation.

Similar to the observations made for the stress-strain response, the effective stress paths (Figure 8c.) indicate that the material behaves in a very stiff elastic manner until mobilising a peak friction angle after which the stress path heads towards a post-rupture friction angle. While there does not appear to be any significant influence of strain-rate on the values of either the peak or ultimate friction angle, it is shown that as strain-rate increases, the effective stress path tends to decrease in obliquity as it approaches the peak friction angle. While the pore pressures is only measured at the base of the sample, the permeability and relatively small samples size reduce the significance of transducer location on measured stress state. Similar to the results found for naturally structured clays, both the cemented and reconstituted samples reach a unique friction angle at large (>10%) strain ([Randolph et al., 1979](#); [Smith et al., 1992](#); [Burland et al., 1996](#)).

Bender element tests

Figure 9 shows the development of small strain stiffness, G_0 , with time for samples of kaolin/hemihydrate mixed with 60% water w/w. Results indicate that the small strain stiffness of the samples is proportional to the amount of cement used in the dry mixture. The time taken to completely cure is shown to be independent of the amount of cement and equal to approximately 40 minutes from the point of wetting; after which, no significant change in stiffness was observed for the samples. The first arrival could not be determined for the

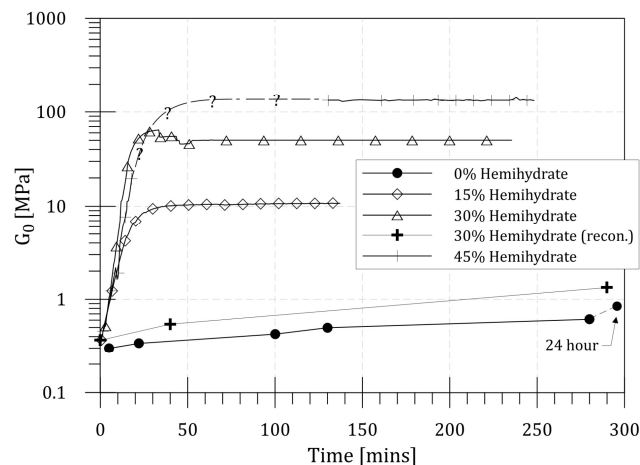


Fig. 9. Increase of shear modulus with time during cementation of gypsum-kaolin specimens

45% hemihydrate sample during the final curing stage (25-50 minutes). An estimated small strain stiffness with time curve is drawn in place of the missing section of test data for 45% Hemihydrate mixture.

Figure 10 shows the development and degradation of small strain stiffness during isotropic compression, unloading and undrained shearing for Test GKP08. The shear modulus is shown to increase and decrease with confining stress but possess a greater stiffness at equivalent stresses upon unloading. This would indicate that despite the degradation of soil structure during isotropic compression, the mechanical overconsolidation process has increased the sample stiffness at a greater rate than that lost from structural degradation. The continued increase in small strain stiffness during the loading stage is believed to be a result of creep during consolidation. Figure 6.56 then plots the small strain stiffness with shear strain during undrained shearing for the same test. A steady decrease in the small strain stiffness is observed for the material suggesting a gradual breakdown of the overall structure with strain. A comparison with the stress-strain behaviour, as shown in Figure 7b., suggests that the majority of the small strain stiffness degradation occurs following peak shear strength and is likely a reflection of the shear band propagating through the sample and breaking down the structure. It is worth noting that at the point of strain localisation, shear wave velocity is no longer reliable. The trend in shear modulus with strain is, up to the point of localisation, a reduction in shear modulus with strain level.

Still higher than reconstituted at post-peak failure—bonding still effective?

Framework for the mechanical behaviour

In order to highlight the role microstructure in the mechanical behaviour of gypsum cemented kaolin, it is worth comparing the test results in terms of both undrained stress-strain response and normalization with voids ratio. The trend in material behaviour can then be compared with natural clays using the generalised mechanical framework proposed by [Cotecchia & Chandler \(2000\)](#) or [Baudet & Stallebrass \(2004\)](#).

Figures 11 and 12 show the undrained stress-strain behaviour and excess pore pressure, Δu , normalised with respect to initial effective confining stress, p'_0 . For clarity, data has been separated to show results where consolidation pressure or strain-rates have been varied between tests.

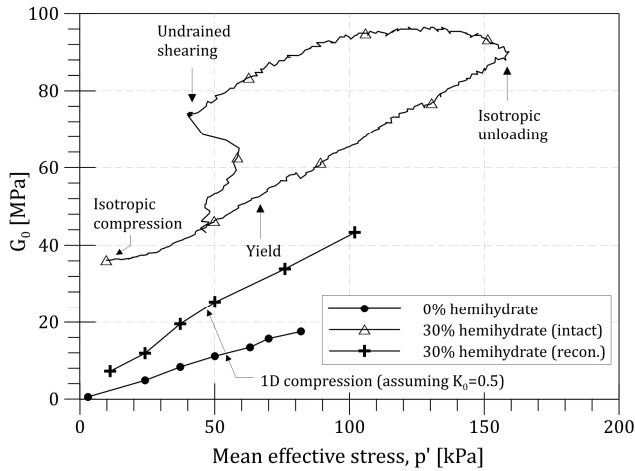
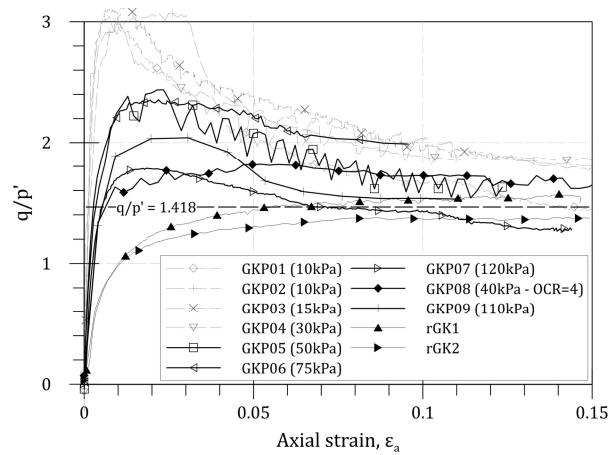
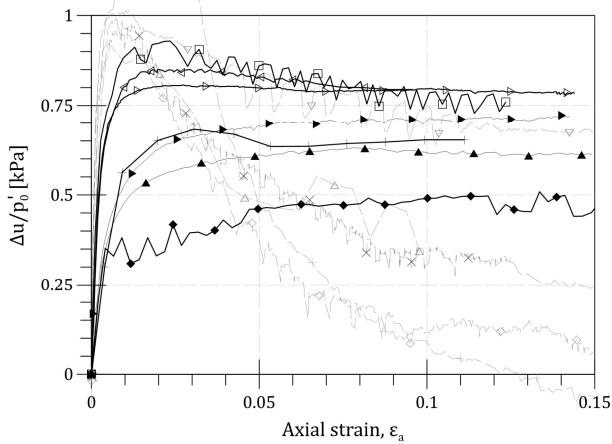


Fig. 10. Variation in G_0 with isotropic confining pressure and during shearing for a gypsum-kaolin specimen (GKP09)



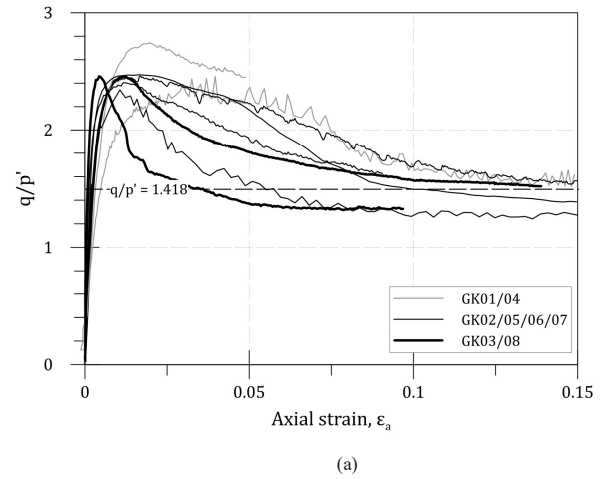
(a)



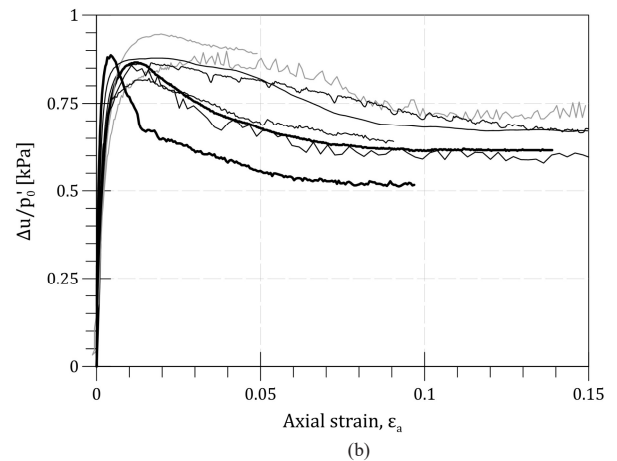
(b)

Fig. 11. Stress ratio and excess pore pressure against axial strain rate for specimens at different confining pressures

It can be seen that the both strain-rate and consolidation pressure have an effect on the normalised behaviour. In particular, the peaks of both the stress ratio and normalised pore pressure response are noticeably reduced with structural degradation. Where the sample was sheared at a very low confining pressure, zero effective lateral stress was experienced



(a)



(b)

Fig. 12. Stress ratio and excess pore pressure against axial strain rate for specimens sheared from $p' = 40$ kPa

by the sample as seen in the pore pressure tending towards zero. It can be seen from both Figures that the magnitude of normalised pore pressure is a function of consolidation stress. The maximum normalised value is found to reduce from a maximum of one with continued destructuration. Particularly, the process of mechanical overconsolidation significantly reduces the amount of excess pore pressure developed during shearing compared to a sample consolidated to the same pressure during its virgin consolidation. The generation of excess pore pressure is then thought to reflect the transfer of load from the structured fabric to the fluid; suggesting that as less pore pressure is generated, fewer bonds are degrading with shear strain. Excess pore pressure is seen to behave similarly to the stress-ratio, displaying a peak value and reducing to an ultimate state with further strain. Where no peak shear stress can be observed for the stress-ratio, excess pore pressure is seen to only increase.

As mentioned previously, intact samples show a stiff response followed by a degree of strain softening to a post-rupture state. As expected, peak stress-strain ratio is found to decrease with isotropic confining pressure. Tests from a common confining pressure are shown to present very similar peak responses. For all tests, however, a common ultimate stress-ratio is reached, suggesting the sample is at a post-rupture state. The ultimate stress-ratio is similar for both intact and reconstituted samples.

Due to natural variabilities of sample state prior to shearing, in terms of density, effective stress, stress history and, consequently, microstructural features, it is necessary to

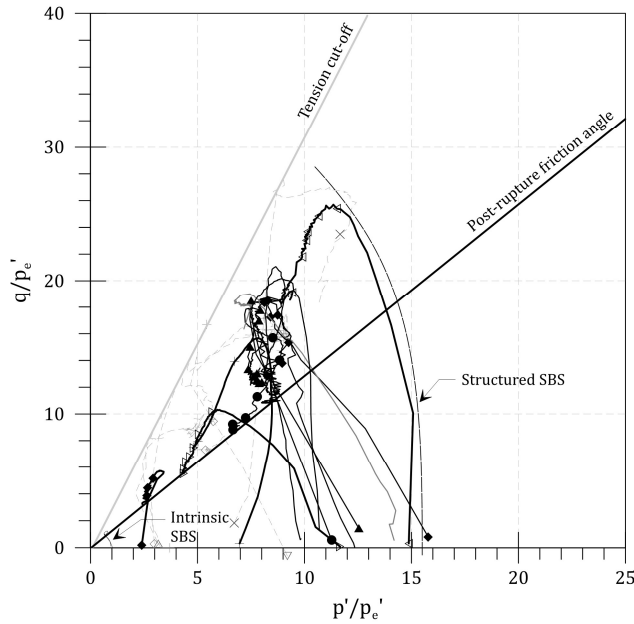


Fig. 13. Stress path for all tests of gypsum cemented kaolin specimens normalised for volume (p'_e)

introduce a normalisation procedure. For this study, all stress paths were normalised with respect to the equivalent pressure, defined as

$$p'_e = \exp[(N - \nu/\lambda)] \quad (1)$$

where ν is the equivalent specific volume at a mean effective stress, p' . Parameters N and λ define the isotropic compression line (ICL) for the reconstituted soil, as given in Table 3. Within the critical state soil mechanics (CSSM) framework, the specific volume of drained triaxial tests are predicted to move through an infinity of constant ν section of the Roscoe surfaces. By normalising the stress state with respect to the reconstituted specific volume, it is possible to obtain a bi-dimensional representation of the state boundary surface (SBS) (e.g. Atkinson & Bransby, 1978; Wood, 1990). Where soil states for intact samples exist to right of the reconstituted ICL in the $e - \ln p'$ projection, values of p'/p'_e are greater than unity.

Figure 13 shows the normalised undrained stress paths for tests described previously. Test results are shown to exhibit behaviour typical of a structured soil. It is believed that following peak, the stress state moves from a structured boundary surface towards an intrinsic state. This behaviour can be understood when considering the volumetric compression during yielding of the soil structure, as shown in Figure 7a. For the cemented soil, isotropic yield occurs at approximately 70kPa. The normalised undrained stress paths correspondingly plot furthest to the right and define the structured SBS. Additional isotropic compression prior to loading is shown to move the stress path inside the outer SBS further suggesting the soil structure is experiencing some form of progressive collapse and a reduction in strength sensitivity (Cotecchia & Chandler, 2000). It is worth noting that where mechanical overconsolidation (OCR=4) has been applied to the soil, GKP08, the stress state is well within the SBS, significantly closer to the intrinsic SBS and fails at the same post-rupture friction angle.

No unique Hvorslev surface or “dry side” was found for the soil on the normalised plot. This is believed to be the result of (i) encountering zero lateral pressure at low confining pressure and

(ii) progressive collapse of the Hvorslev surface with isotropic loading beyond yield. To clarify the normalised behaviour, Figure 14 plots only the peak states for the undrained stress paths; each point is labelled to indicate whether the sample was sheared from different confining pressure or at different strain-rates. Contrary to CSSM, no single ultimate state can be identified on the $q/p'_e - p'/p'_e$ plot. Instead an ultimate frictional angle can be observed; this is similar to observations made by Cotecchia & Chandler (2000) for Pappadai clay. As for gypsum cemented kaolin soil, strength sensitivity S_t , defined as the ratio of the undrained strength after consolidation to gross yield to that of the reconstituted clay normally consolidated to the same water content as the structured clay at gross yield, is shown to peak at a value of 15.5. As with natural clays, gypsum cemented kaolin samples sheared from the maximum yield stress ratio (YSR) or point exhibiting maximum strength sensitivity, trace the wet side bounding surface or Roscoe surface and exhibit their peak undrained strength at the apex of the SBS. The bounding surface is noted to be similar to other artificially bonded soils, such as Xiao *et al.* (2014), where the Roscoe surface is significantly distorted vertically and intersects the tension cut-off line.

Where samples were consolidated to a unique isotropic consolidation pressure and subsequently sheared at a range of strain-rates, a consistent peak friction angle can be seen in the normalised stress space. This would suggest that despite the increase in undrained shear strength, the slip surface angle is constant for the range of strain-rates tested in this study. As discussed by Burland *et al.* (1996), the coincidence of the critical state and post-rupture angle is likely to be a result of the formation of shear bands at peak strength. For highly structured or cemented soils, shear banding is likely to occur after a relatively small displacement. Burland (1990) has previously shown that ϕ'_{pr} is equal to the lower bound of the angle of shearing resistance along a single fissure.

The set of results from gypsum cemented kaolin samples suggest that the material can be treated as a surrogate for structured soils. The material does however not conform to typical CSSM and should be interpreted in a modified framework. Based on the work by Randolph *et al.* (1979) and Burland (1990), the behaviour of gypsum cemented kaolin is proposed to be modelled as depicted in Figure 15, which shows projections of the effective stress path on the $q - p'$ plane and on the $e - p'$ plane. In addition to the line of critical state, a post-rupture is required to model the behaviour of the structured soil. Similar to that found for other natural clays (Cotecchia & Chandler, 1997; Saihi *et al.*, 2002), the post rupture line is parallel to the normal consolidation line and would therefore be dependent on the degree of structuration present in the sample upon shearing. Following post-rupture, the soil will tend towards the intrinsic state given by the reconstituted sample at the equivalent moisture content. Due to limitation of strains achievable in the triaxial apparatus and the likely propagation of failure planes, shearing is not likely to result in complete destructuration of the soil fabric but rather the sliding of rigid elements along slip surfaces. Similar limitations have been observed for natural soft clays (Smith *et al.*, 1992) and stiff clays (Burland *et al.*, 1996).

Reflecting on constitutive models where structure is explicitly modelled with the use of a structuration parameter or Δe , it is recognised that such simulations would tend to over-predict the deformation/destructuration observed in intact triaxial sample during undrained shearing. It is likely that, even with triaxial tests being taken to the limits of strains possible in the apparatus, highly structured samples will not be able to reach the critical state obtained from reconstituted samples.

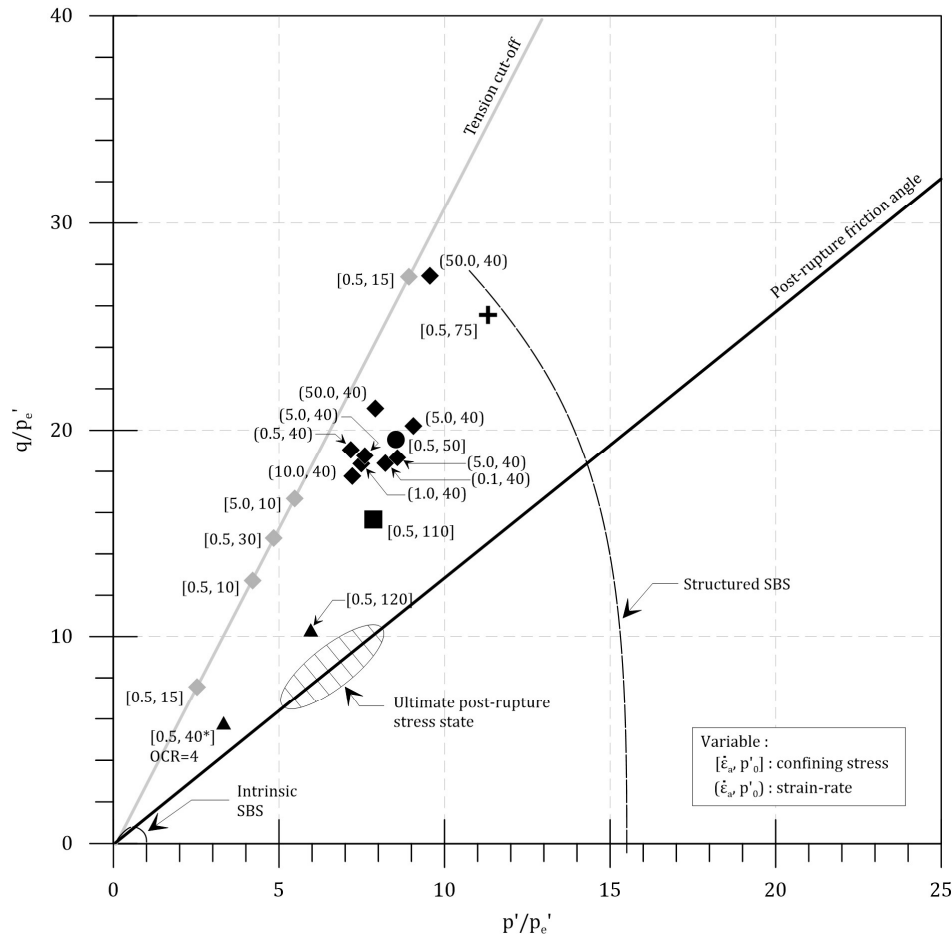


Fig. 14. Gypsum cemented kaolin specimens: peak normalised stress states, structured bounding surface, reconstituted SBS

From the results presented here it appears that, at low to moderate confining pressures, the better parameter for normalizing tests would be the equivalent mean effective stress on the ultimate state line (either critical state or post-rupture) corresponding to the void ratio of soil. The stress paths for both structured and reconstituted triaxial tests are plotted in Figure 16. The alternative normalization provides a much clearer representation of the soil behaviour in terms of soil structure and avoids the significant difference in scale between the reconstituted and structured samples. By further factoring the deviatoric stress by the stress ratio M , the theoretical final condition is easily identified when both p'/p'_{ult} and q/Mp'_{ult} equal unity.

CONCLUSIONS

The physical characteristics and mechanical behaviour of artificially cemented soil has been investigated and shown to reproduce many of the characteristic features of a naturally structured soil. Suites of oedometer tests on various combinations of β -hemihydrate and kaolin were performed to categorise the degree of cementation present in the final sample and observe the mechanical destructure with vertical effective stress. A combination of 30% unhydrated β -hemihydrate to 70% kaolin wetted to 60% of total dry weight was found to give vertical compressibility similar to naturally soft soils.

The mechanical behaviour of the material in the deviatoric plane was investigated using the triaxial apparatus. Tests were limited to isotropic conditions for simplicity and due to the artificial nature of the material. Structural degradation is known to involve damage to the interparticle bonding and reduction in both the micro and macro pores. Changes in soil structure will therefore depend on both the volumetric and deviatoric plastic strains during consolidation and loading. This was investigated by analysing the behaviour of both intact samples subjected to a suite of consolidation pressures and strain rates. The isotropic compression curve was found to be similar to that observed in the oedometer. Namely that yield occurred at a relatively low pressure, the intact samples exist to the right of the normal compression line traced by the reconstituted soil, and the compression curve of the intact sample tended towards the reconstituted line with further increases in stress.

The shear behaviour of the cemented soil has been shown to be controlled by mechanisms which differ from those for its reconstituted state and which are a result of its structure. These mechanisms are shown to be principally determined by the state of structure remaining in the sample prior to shearing. Similar to previous investigations regarding naturally structured soils, the mechanical behaviour can only be modelled if the process of destructure is explicitly considered. That is, changes in both the size and shape of the SBS were observed for different states of structure and for different strain rates. It was shown that volumetric compression more significantly

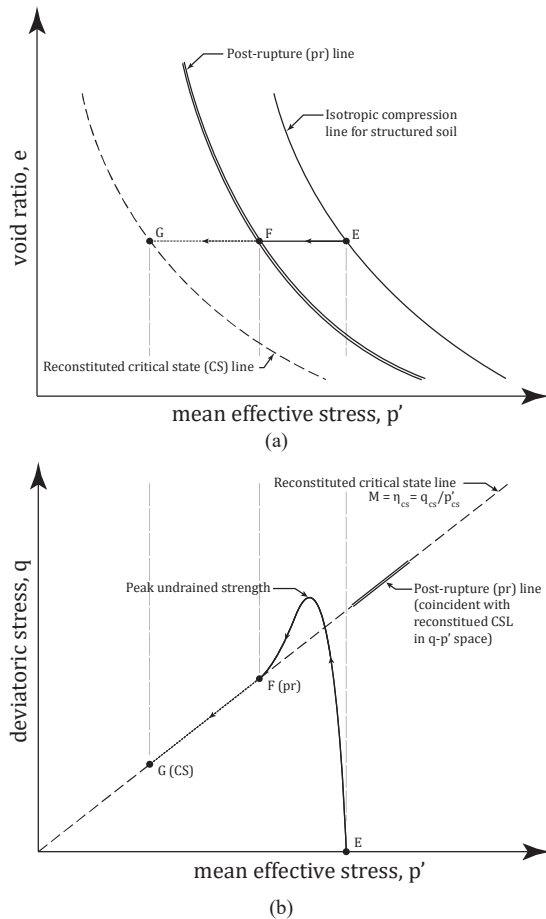


Fig. 15. Schematic diagram showing modes of shearing behaviour for a cemented soil at constant volume (undrained) (adapted from Randolph *et al.*, 1979)

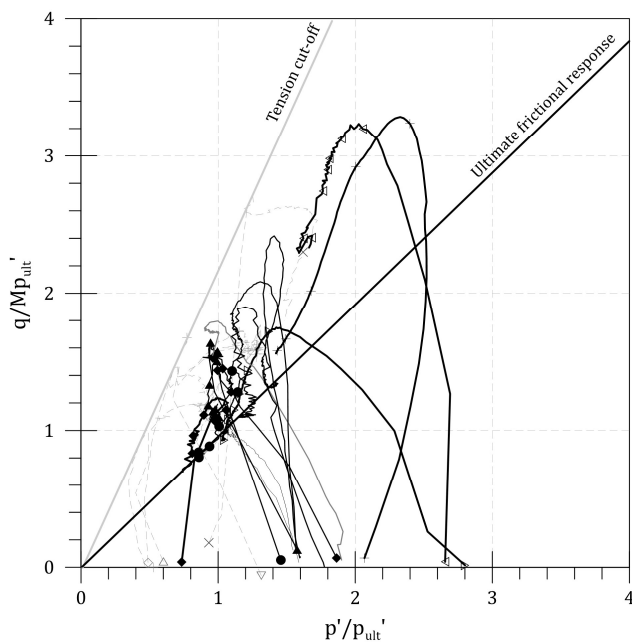


Fig. 16. Stress path for all tests of gypsum cemented kaolin specimens normalised for ultimate state (p'_{ult})

affects the peak frictional response than strain-rate. For all tests, a unique post-rupture friction angle can be determined. Due to limitations in the strains achievable in the triaxial apparatus and strain localisations, critical state could not be confirmed for material when sheared from an intact state. The stress state is however believed to be tending towards the intrinsic SBS at the termination for all tests.

ACKNOWLEDGEMENTS

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NOTATION

e	void ratio
e_i	initial void ratio
G_0	maximum shear stiffness
G_s	specific gravity
M	critical state stress ratio
M_{pr}	post-rupture stress ratio
m_c	moisture content
OCR	overconsolidation ratio
N	intercept of the NCL at $p' = 1 \text{ kPa}$ in $e:\ln p'$ plane
p'	mean effective stress
p'_e	equivalent mean effective stress
p'_0	initial effective confining stress
q	deviator stress
Δu	excess pore pressure
ϵ_a	axial strain
$\dot{\epsilon}_a$	axial strain-rate
Γ	intercept of the CSL at $p' = 1 \text{ kPa}$ in $e:\ln p'$ plane
κ	slope of swelling curve
λ	slope of virgin compression curve
ν	specific volume
σ_v	vertical effective stress
σ_v^*	intrinsic vertical equivalent stress
ϕ'_{peak}	peak friction angle
ϕ'_{pr}	post-rupture friction angle

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