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On the compression behaviour of reconstituted soils

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Abstract

Transitional behaviour has been recognised in a diverse range of soils in the literature to date, from gap-graded soils to well-graded silts and sands. It is typified by non-convergent compression paths and critical state lines that are non-unique and which are dependent on the initial sample density. Many soil mechanics theories assume soils to have unique normal compression and/or critical state lines, which is not the case for a transitional soil. For such a soil it would therefore be difficult to identify the effects of structure on the mechanics since there is no unique behaviour of the soil when reconstituted. This paper describes series of oedometer tests that were performed to investigate in more detail when and why non-convergent compression behaviour might be expected, as a first step to identifying which soils are transitional. The effects of mixing soils of different grain sizes and mineralogies were explored, the tests revealing that convergent or non-convergent behaviour could be brought about either by relatively small changes to the proportions of the soil particles or by changes to their nature. It was also found that in some soils with non-convergent compression behaviour there was particle breakage while in others there was not. Since the factors that influence the mode of compression behaviour were found to be complex, it is concluded that each new soil encountered must be assessed individually for whether it is transitional, and that the accuracy of void ratio measurements is of particular importance in establishing this reliably.
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Keywords: Compression; Particle breakage; Void ratio; Sandy soil; Artificially mixed soils; Laboratory tests

1. Introduction

Over the past two decades research in soil mechanics has shown that the behaviour of many clean sands can be described within a critical state framework similar to that for clays (e.g., Been and Jefferies, 1985; Coop and Lee, 1993; McDowell and Bolton, 1998). The main difference between the frameworks for sands and clays is the mechanism of plastic volumetric compression. In clays physical-chemical forces between the particles control the behaviour whereas in coarse grained soils the stresses are transmitted through inter-particle contacts in strong force chains that result in particle breakage if the applied stresses are sufficiently large. Another key difference is that sands are deposited at much denser states than clays, and so as they are compressed to higher stresses they may meet a unique Normal Compression Line (NCL) in v:logσ'_v or v:lnp' space (v, specific volume; σ'_v, vertical effective stress; p', mean normal effective stress) (Coop and Lee, 1993; McDowell and Bolton, 1998; Pestana and Whittle, 1995). The increase of gradient or "knee" point in the compression paths as they reach the NCL is generally believed to correspond to the onset of major particle damage and can be correlated with the strength of individual particles (e.g., Kwag et al., 1999), although it is not the true onset of plastic strains. The specific volume and stress level at which the compression paths reach

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a unique NCL will also depend on the initial density of the sand because of the effect that has on the coordination number. It is therefore typical that clean quartz sands reach their NCL at specific volumes in the range 1.3–1.6 and at stress levels typically of 10–100 MPa (e.g., Mesri and Vardhanabhuti, 2009). In contrast other sands with higher volumes and weaker particles can reach their NCLs at much lower stresses. For example the Dogs Bay carbonate sand tested by Coop and Lee (1993) reached its NCL at specific volumes in the region of 2.2–2.6 and at stresses in the range 1–3 MPa.

More recent research has identified that there are many intermediate graded soils which have behaviour that cannot be described within a simple critical state framework. These soils have been called 'transitional' in that they have a grading and a mode of behaviour between that of clean sands and plastic clays. The main feature of transitional soil behaviour is that unique normal compression and critical state lines cannot be identified (e.g., Martins et al., 2001; Ferreira and Bica, 2006; Nocilla et al., 2006). The compression paths of all soils must eventually become asymptotic to the minimum specific volume of 1 and in this sense they must all eventually converge. For data of Ferreira and Bica (2006) this must be beyond the maximum stress they used of about 25 MPa, but between this stress level and the possible locations of the horizontal asymptotes, it did not seem likely that there could exist a unique NCL since the void ratios had already reduced to very low values in the region of 0.2 at 25 MPa. Further aspects of the conventional critical state framework also break down for their shearing behaviour, such as the existence of unique State Boundary Surfaces (Nocilla et al., 2006).

Unfortunately, the term 'transitional' has already been used to describe two other aspects of soil behaviour, firstly an intermediate shearing mode between turbulent particle motion and sliding at the residual state (Lupini et al., 1981) and secondly the transitional or limiting fines content when soil behaviour changes from coarse grained to fine grained dominated (e.g., Pitman et al., 1994; Lade and Yamamuro, 1997; Yang et al., 2006). As yet it is unclear exactly how each definition is related. The type of behaviour associated with the most recent use of 'transitional', of concern in this paper, was first identified in gap-graded soils and was initially thought to be a feature exclusive to their behaviour. Martins et al. (2001) identified non-convergent compression behaviour for the gap-graded Botucatu residual sandstone. Ferreira and Bica (2006) confirmed that even if the compression was taken to very high stresses (25 MPa), convergence still could not be found and also that the location of the critical state line of the reconstituted soil was similarly dependent on the initial density, so that the soil might be defined as transitional. Critical states were defined to be those states of constant stress and volume at large strains in a triaxial apparatus, although it was realised that the non-unique volumes would mean that the fabric was not unique at that state.

Non-convergent compression paths were also found for a mixture of kaolin and a quartz sand (Fig. 1), which replicated the grading but with a simpler mineralogy. However, as

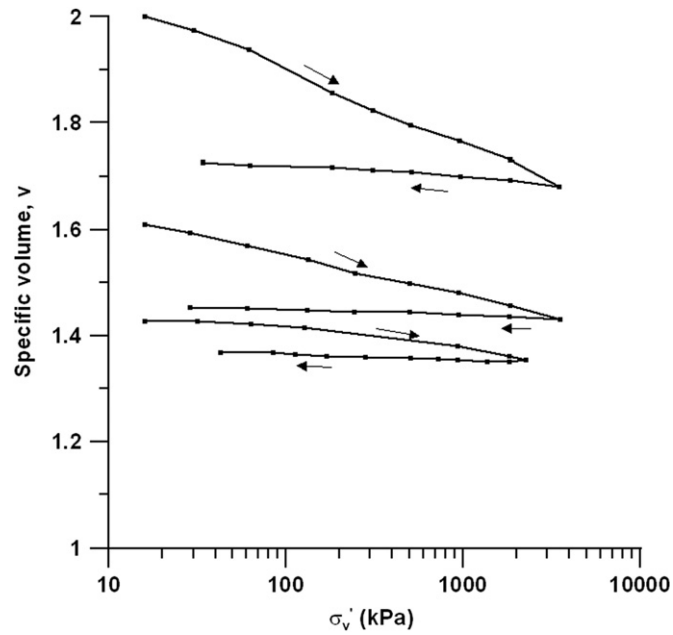

Fig. 1. One-dimensional compression paths of a model quartz sand–kaolin mixture containing 75% sand and 25% kaolin (redrawn from Martins et al., 2001).

further research was conducted, it was realised that this type of behaviour was likely also to be applicable to many intermediate graded soils (Nocilla et al., 2006), challenging a common assumption that most or all soils can be described within a critical state framework. Nocilla et al. (2006) searched for transitional behaviour in a well-graded Italian river silt, for which the clay content was controlled. As the clay content reduced the behaviour changed from that of a typical clay mode, with unique NCLs and CSLs, to a transitional one with non-unique NCLs and CSLs. Nocilla and Coop (2008) observed features of transitional behaviour in natural samples of the same well-graded silt, confirming that the transitional behaviour they had observed was not simply a function of the reconstituted fabrics created in the laboratory. More recently, Altuhafi and Coop (2011) have found that as they changed the grading of three sands of different mineralogies from poorly to well-graded, their compression behaviour changed from a "sand" type with a unique NCL associated with particle breakage, to a non-convergent one where there was no unique NCL. In each case when the non-convergent behaviour was observed there was no measurable breakage during compression, even up to 100 MPa.

The work described in the paper presents a new series of oedometer tests and particle size analyses that were conducted to investigate the role of grading, mineralogy and particle breakage in the compression of a wide variety of reconstituted soils. Various combinations of fine and coarse particles were used to create different soils to help identify which types of soil display non-convergent behaviour behaviour and to try to understand how the different particle types can influence the compression and particle breakage behaviour. Non-convergent compression behaviour may be seen as a first step in identifying transitional

Table 1
Summary of test results from this paper and the existing literature.

Soil type	Reference	Origin/principal mineralogy	Mixes tested	Observations on transitional and/or convergence of compression behaviour
Botucatu residual sandstone	Martins et al. (2001) Ferreira and Bica (2006)	Natural origin, residual aeolian. quartz/feldspar	Tested in reconstituted and intact state	Clear transitional behaviour for reconstituted soil, but not for intact.
Quartz sand/kaolin	Martins et al. (2001)	Artificial mix. Quartz/kaolin	75% quartz sand, 25% kaolin.	Non-convergent compression behaviour
Clayey silts	Nocilla et al. (2006) Nocilla and Coop (2008)	Fluvial soils from river embankments and sub-soil. quartz/calcite	Reconstituted samples with controlled clay contents (3.5–45%) and intact samples with variable clay contents (13–53%).	Reconstituted: transitional for clay contents of 3.5, 8 and possibly 20%, not at 45%. Intact samples probably transitional for 13–16 and 20–26% clay contents, probably not for 39–53%.
Thanet sand	Ventouras (2009) Ventouras and Coop (2009)	Predominantly quartz	Reconstituted samples of high fines stratum (15–30% *fc*) and low fines stratum (5–10% *fc*)	Non-convergent compression behaviour in high fines stratum but not in low fines stratum.
Sands	Altuhafi et al. (2010) Altuhafi and Coop (2011)	Reconstituted single mineralogy sands	Leighton Buzzard quartz sand, Dogs Bay carbonate sand and basaltic glacial sand	In all three sands evolution from convergent sand mode of compression behaviour to non-convergent mode as grading changed from poorly to well-graded
Carbonate sand/kaolin	This paper	Artificial mix. carbonate/kaolin	Mixes with 5, 10, 40 and 70% sand.	Non-convergent compression behaviour for 5, 10 and 40% sand. Convergent sand mode of compression behaviour for 70% sand.
Carbonate sand/crushed quartz silt	This paper	Artificial mix. carbonate/quartz	Mixes with 0, 40 and 60% sand.	Convergent compression behaviour for all mixes.
Carbonate sand/quartz sand	This paper	Artificial mix. carbonate/quartz	0, 10, 30, 50, 70 and 100% carbonate sand	Non-convergent compression behaviour for 10 and 30% carbonate sand. Convergent sand mode of behaviour for all other mixes.
Quartz sand/kaolin	This paper	Artificial mix. quartz/kaolin	Mixes with 0, 25, 35, 50, 55, 60, 65, 70, 75, 90 and 100% sand.	Non-convergent compression behaviour for 75 and 90% sand. Clay mode of behaviour for sand $\leq 70\%$. Convergent sand mode for 100% sand.
Quartz sand/crushed quartz silt	This paper	Artificial mix. quartz/quartz	Mix with 75% sand.	Non-convergent compression behaviour.
Quartz sand/crushed quartz silt/kaolin	This paper	Artificial mix. quartz/quartz/kaolin	Mix with 25% sand, 25% silt, 50% kaolin.	Non-convergent compression behaviour.
Clays	This paper	Kaolin, London clay (illite) and Oxford clay (illite)	Reconstituted samples	Slight non-convergent compression behaviour with very slow convergence especially for kaolin and Oxford clay.

soil behaviour, although it is not necessarily diagnostic as a soil may have non-unique compression paths but still have a unique CSL. The characteristics of the soils tested and discussed, both in the existing literature and in this paper are summarised in Table 1.

2. Materials and fabricated mixtures

The behaviour of intermediate graded, mixed graded and transitional soils is not well understood, mainly due to the complex interactions between the fine and coarse grains present in the soils and the broad range of soils which these can include. To understand clearly the fundamental behaviour of such soils it was therefore decided that much of the laboratory research should concentrate on testing model soils. The main advantages of testing artificially created soils are that the specimens can be designed to have controlled and simple gradings and mineralogies, eliminating any more complex effects that these might have on the mechanical behaviour of the soil, and that it facilitates experimental repeatability. Most of the soil mixtures tested were mixed or gap-graded in nature.

The grading curves and properties of the materials used are given in Fig. 2 and Table 2. Dogs Bay sand, Thames Valley sand and Redhill sand were used to form the coarse fraction of most of the oedometer test samples. Dogs Bay sand is a biogenic carbonate sand from Ireland consisting mainly of unbroken angular mollusc and foraminifera shells, many of which have intra-particle voids. These features result in a poorly-graded soil with fragile particles and high specific volumes. A comprehensive investigation of the behaviour of Dogs Bay sand was carried out by Coop (1990). Thames Valley sand is a poorly-graded river terrace sand from the UK that was chosen as a replacement for the extensively used Ham River sand (Bishop et al., 1965), and so has very similar properties (Takahashi and Jardine, 2007). It is sub-rounded

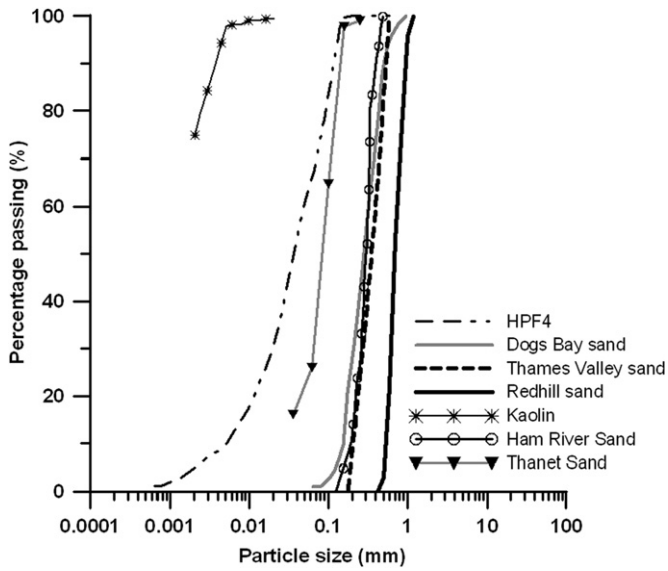

Fig. 2. Grading curves of soils tested.

Table 2
Material properties (references in text).

Material	ϕ'_{CS} (deg.)	G_s	LL (%)	PL (%)
Dogs Bay sand (Ireland)	40.3	2.71	–	–
Thames Valley sand (UK)	33	2.67	–	–
Redhill sand (UK)	–	2.65	–	–
Kaolin (UK)	23	2.61	62	32
Crushed quartz silt (HPF4) (UK)	33	2.65	–	–

to sub-angular and consists predominantly of quartz. The third sand, Redhill sand, is from the Cretaceous, Lower Greensand also of the UK, and has sub-rounded particles and again is predominantly quartz.

The material used to constitute the fine fraction of the mixtures tested was either Speswhite kaolin or a crushed quartz silt known as HPF4. Both are commercial soils which are readily available from UK suppliers. The crushed quartz silt has angular and elongated particles and its behaviour was investigated by Zdravkovic (1996). Most of the fabricated mixtures created for testing were limited to two types of soil particle, one fine and one coarse.

3. Experimental procedures

The proportions, mineralogy and particle sizes of the fine and coarse grains in the mixtures were varied and the samples created were tested in the oedometer at a range of initial densities and to a variety of stress levels. Particle size analyses using hand sieving and also a Sympatec QicPic particle analyser were conducted on selected samples before and after testing to investigate the role of particle breakage for the soils tested. For perfect spheres different techniques of measuring particle size should give the same result, but for irregular particles the measured size will depend on the technique used and the definition of particle size that is adopted. The QicPic apparatus scans particles falling under gravity with a laser that allows a randomly orientated outline of the particles to be recorded, which is a very different process to sieving. Of the various diameters that the apparatus calculates, the Feret minimum diameter was used as this gives particle sizes that are closest to those measured by sieving (e.g., Altuhafi and Coop, 2011). The limitation of its resolution means that it can only be used, however, with the coarser fraction.

Because a memory of the initial void ratio of a soil can only really reside in its fabric, a variety of preparation methods was used so that it could be seen whether fabric and the current void ratio was simply related to the current stress and initial void ratio, or whether the preparation method was also important. However, the majority of samples were created as slurries at a variety of initial water contents by mixing the dry components with deionized water before carefully placing the resulting slurry directly into the oedometer ring. Care was taken to ensure saturation by de-airing the slurries under vacuum. The de-aired slurry was then carefully spooned into the oedometer ring with great care taken to avoid entrapment of air. It was then carefully levelled across the top of the ring. Other samples were made by either dry compaction or wet compaction, with the initial void ratio being controlled by the compactive effort. Compaction was carried out by tamping, a greater number of tamps and/or a greater tamping rod weight being used for the denser samples. For wet compaction a small initial water content was used to allow the soil to be placed in very loose states, although it was compacted sufficiently to ensure that there were no macro-voids.

In all cases, the samples were saturated through inundation of the water bath of the oedometer after an initial load increment, typically of 10–15 kPa, had been placed. This initial load was to ensure good contact from the sample through the platen to the loading arm so minimising errors due to bedding of the platens or lack of good contact with the loading system, while also preventing swelling while the water bath is filled. All of the oedometer tests were incrementally loaded, using sample diameters of 50 and 38 mm so that the elevated stress levels required could be reached. The initial heights were around 20 mm. For each mixture a check was made of the required time for each loading stage to ensure that the continuing settlements from creep or undissipated pore pressure were not significant. Typically loading stages of around several hours each were adequate.

4. Calculation of the initial specific volume

For the identification of a transitional soil or at least confirmation of whether the compression behaviour is convergent or not, the accuracy of the calculation of the initial specific volume, v_i, is of paramount importance, ensuring that the locations of the compression paths in v:logσ'_v space are accurate. This will determine whether it can be conclusively determined whether the compression

curves from different initial densities are convergent towards a unique NCL or not. Fourie and Papageorgiou (2001) discussed the errors in the calculated void ratio from measurements of the height and diameter of samples using callipers. It was found that the resulting error in void ratio was 0.003 to 0.006 for an aluminium dummy specimen but the error was thought to be larger for an actual soil specimen. Because of the need to prove rigorously whether the compression curves converge or not, it was felt it was not sufficiently accurate simply to estimate the likely accuracy in this way, but that actual comparisons of different means of measuring v_i should be made.

Table 3 shows an example of how the initial specific volume, v_i, was calculated using four different equations. This has been deliberately chosen as a typical case, and indeed one value of the four calculated is poor. The four equations used were chosen to be as independent of each other as possible, although there is of course some linkage between them. They were based on the initial water content, w_i, the initial dry unit weight, γ_{di}, the initial bulk unit weight, γ_i, and the final water content, w_f. In the case of the final water content method the initial specific volume is derived from the final value by means of the overall volumetric strain measured during the test. Each method depends on and is sensitive to various measurements such as the initial height, the final wet mass and the specific gravity, G_s. They also ignore any possible effect of the compressibility of the solids. Because there are four methods to calculate v_i, there is redundancy in the calculations and so there is no need for the methods based on w_i, and γ_i to assume that the soil is fully saturated, even if it is likely for the samples made as slurries. An average value of the degree of saturation, S_r, can be then calculated from Eqs. (1) and (2).

$$S_{ri} = \frac{w_i G_s}{v_i - 1} \tag{1}$$

$$\gamma_i = \frac{G_s + S_r(v_i - 1)}{v_i}\gamma_w \tag{2}$$

The value of v_i used in Eqs. (1) and (2) is the mean of the two values calculated from the γ_{di} and the w_f methods (Eq. (2) and 4 of Table 3), the latter assuming that the sample is fully saturated at the end of the test, which is more likely than at the start of the test. Not assuming saturation at the start of the test was found to reduce the scatter in the values of v_i and so improve its accuracy. Despite de-airing under vacuum after mixing, the resulting initial degrees of saturation of the slurries were typically around 0.9, although the degree of saturation is likely to have improved through immersion during the test, which emphasises the need to account for this in calculating the initial void ratio.

Table 3
Example of calculation of the initial specific volume.

Dogs Bay sand : Kaolin (%)	Test	v_i equation [a,b]				Chosen v_i [c]	Estimated estimated accuracy
		1	2	3	4		
5:95	5DBS-k 03	2.49	2.48	2.46	***2.68***	2.47	±0.02

[a]Calculated from: (1) $v_i=(w_i G_s/S_r)+1$, (2) $v_i=G_s\gamma_w/\gamma_{di}$, (3) $v_i=(G_s-S_r)/(\gamma_i/\gamma_w-S_r)$, (4) $v_i=(w_f G_s+1)/(1-\varepsilon_v)$.
[b]Values shown in ***bold italics*** not considered for 'Chosen v_i'.
[c]Average of v_is calculated from Methods 1–3.

For each oedometer test conducted, the values of v_i calculated from the four methods outlined above were compared and any anomalous values were discarded (value No.4 in Table 3). An average of the remaining values was taken and the resulting v_i was the value used for analysis. It is difficult to calculate a meaningful accuracy of the v_i data with only four values and so the approach adopted has been to use the maximum difference between any individual value and the mean (0.02 in this case), although this is likely to be an overestimate. The acceptable error for a given soil depended on how large the differences were in the location of the compression curves. For larger differences in the compression curve locations larger errors could be tolerated without jeopardizing the proof that the soil was transitional. For most soils tests were typically discarded if they had estimated errors of greater than about ±0.05, and the typical mean error was around ±0.02–0.03. It is interesting that despite the great care taken with the testing techniques, the estimated accuracies are rather poorer than those proposed by Fourie and Papageorgiou (2001), but they are similar to those of others investigating transitional soils and using similar techniques (e.g., Nocilla et al., 2006).

5. Laboratory results

5.1. Carbonate sand–clay mixtures

Four different mixtures were created which had a coarse fraction of Dogs Bay sand and a fine fraction of kaolin. The proportions by dry weight of sand were 5, 10, 40, and 70%. The size range of the Dogs Bay sand was limited to the coarser gradings (0.3–0.425, 0.425–0.6 and 0.6–1.18 mm), which, as will be discussed below, was to highlight any particle breakage that occurred.

The oedometer test results of the four mixtures are shown in Fig. 3. Also included for comparison are the one-dimensional NCLs for the pure kaolin and for Dogs Bay sand, although the latter is for the natural grading (unfortunately the coarser gradings of the Dogs Bay sand are only present in small quantities and so there was insufficient soil to determine the NCLs of the specific gradings used). In some cases different sand particle sizes were used (Fig. 3a, b and d) but this has no clear effect within the accuracy of the tests.

It is surprising how the addition of relatively small amounts of sand affect the compression behaviour quite severely and already at 5 and 10% sand contents non-convergent compression behaviour is seen. The mean of

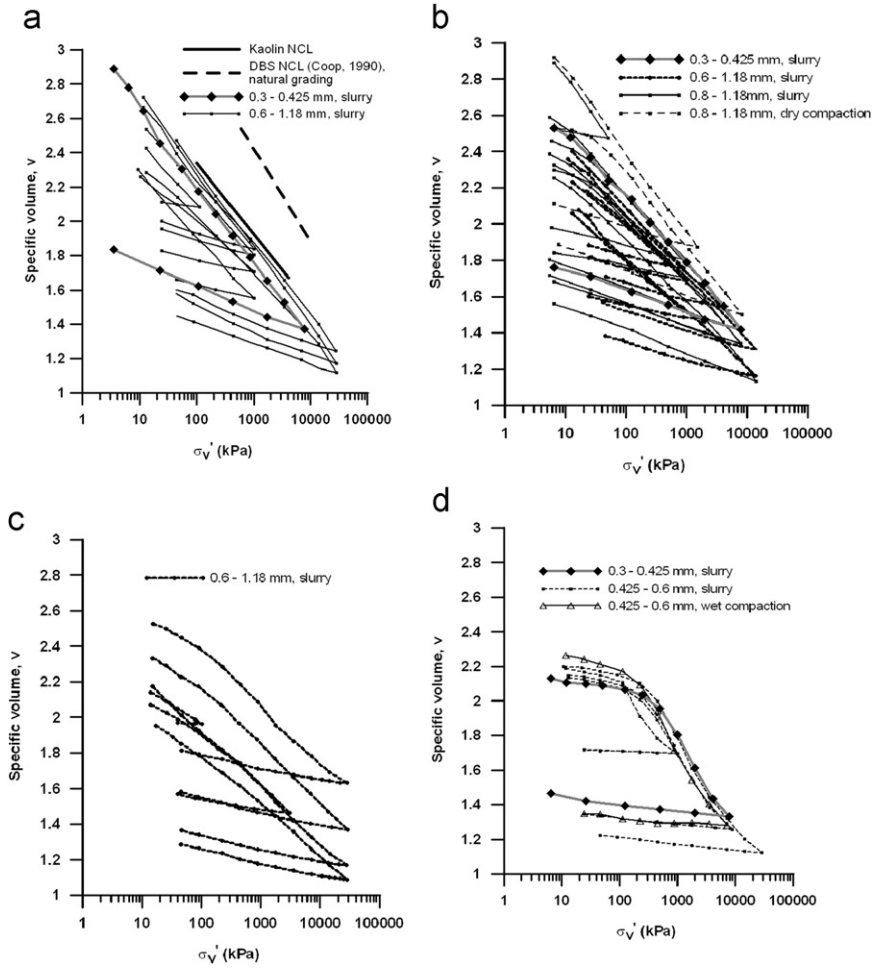

Fig. 3. Oedometer compression and swelling curves for Dogs Bay carbonate sand–kaolin mixtures. (a) 5% sand. (b) 10% sand. (c) 40% sand. (d) 70% sand.

the estimated accuracies for all of the tests on the Dogs Bay sand: kaolin mixtures was ± 0.02 and the spacing between the compression paths is rather larger than this. The compression paths are not completely parallel and there is a gentle convergence, the denser samples having slightly lower compression indices, C_c, but the convergence is far too slow for there to be any possibility of arriving at a unique Normal Compression Line before the compression curves start tending towards their horizontal asymptotes as they approach $v=1$ at very high pressures.

Because the C_c values tend to change with the initial density, it is not easy to make direct comparisons between the different gradings, but as the sand content is increased, the average compression index tends to decrease initially before increasing again at the higher sand contents. There have been many investigations of how the gradient and location of NCLs and CSLs vary in the volumetric plane. Some have seen this type of pattern of decreasing and then increasing gradient as the fines content increase (e.g., Carrera et al., 2011) while others have only seen a continuously increasing gradient (e.g., Nocilla et al., 2006; Yin, 1999). In some cases (e.g., Thevanayagam et al., 2002) the dominant effect of changing the fines content is a translation in the v:lnp' plane rather than a change in gradient. Here the overall locations of the compression curves do not however change that much, remaining slightly below the NCL of the pure kaolin, which seems to form an upper bound to the data for the various mixtures. In contrast, the swelling index, C_s simply reduces steadily with increasing sand content.

At 40% sand content there is very clear non-convergent compression behaviour and a knee point starts to appear in the compression paths. This knee is much more apparent for the 70% sand content, but for this mixture the behaviour has returned to one of a unique NCL. It is interesting that the mixed soil reaches its NCL at rather lower specific volumes than the original Dogs Bay sand, which might be explained by the infilling of the voids with fines, but also the stresses at the point of reaching the NCL are much lower, which would not be expected for a denser soil. However, these comments are somewhat speculative as the gradings of the Dogs Bay sand used are slightly different.

For clean sands the steepening or knee of the compression paths as the NCL is reached has been found to be due to the onset of significant breakage (e.g., Coop and Lee, 1993;

McDowell and Bolton, 1998) and so the straighter compression paths at higher clay contents might be taken as an indication that particle breakage did not significantly influence the compression behaviour.

Throughout this paper the compression data have been presented with conventional linear v: log stress axes, as are typically used in critical state soil mechanics. Other authors have pointed out that the Normal Compression Line may be curved with these axes and have proposed alternatives. Butterfield (1979) proposed using log v while Pestana & Whittle (1995) used log e (void ratio). McDowell (2005) provided a micro-mechanical justification for linearity of the Normal Compression Line with a log e axis for clean sands, based on the fractal theory of particle crushing. The principle purpose of the work presented in this paper, however, is to observe whether convergence is seen in compression or not, rather than to examine which axes provide the best linearity of the Normal Compression Line. With such a wide range of soils that are investigated here, for which the micro-mechanical processes are also likely to be varied, it is also most unlikely that a single set of axes would be optimum for all the soils. A logarithmic void ratio axis was tried for Fig. 3(d) but the resulting plot gave a Normal Compression Line that was still not straight but convex upwards rather than concave.

The effect of changing the fines content observed here is also somewhat in contrast to that seen by Nocilla et al. (2006), who found that as they varied the clay content of a well-graded silty sand they obtained a transitional type of behaviour at low clay contents (8 and 3%) whereas a unique NCL and CSL was found at higher clay contents (40%). For these gap-graded soils a NCL of a sand type, with a well-defined knee in the compression paths is seen even with a clay content of 30% and no unique NCL of a clay type could be found for a clay content of 95%. As for Nocilla et al., a variety of sample preparation methods was used, most of the samples being made by mixing the soil into a slurry, but some being created by dry compaction and some by compaction in a partially saturated state (wet compaction). Again, no effect of the sample preparation method could be found, so that the effect of the initial density seems to be far more important than any influence of the initial fabric created by the different techniques.

Fig. 4 shows an analysis of the breakage that occurs in the sand fraction using Hardin's (1985) relative breakage (defined in Fig. 4a). There are several means available for quantifying particle breakage, for example the total surface area of the particles, as used by Miura and Yamanouchi (1977) and Miura and O-Hara (1979). However this makes the assumption that the particles are spherical, which would be a poor assumption for the Dogs Bay sand. The key disadvantage of relative breakage is that it ignores any breakage in the fines. This is not so much of a difficulty for these gap graded soils, with the exception perhaps of the silt where some breakage might have occurred, but as discussed below, in other tests on the pure silt no breakage could be measured.

For all of the tests the soil was wet sieved to separate out the fines passing 63 μm and then the particle size distribution of the coarse fraction was determined using the QicPic or for the 40% sand content data by careful dry sieving in the case of the 0.6–1.18 grading. The QicPic was especially useful for the very small amounts of sand

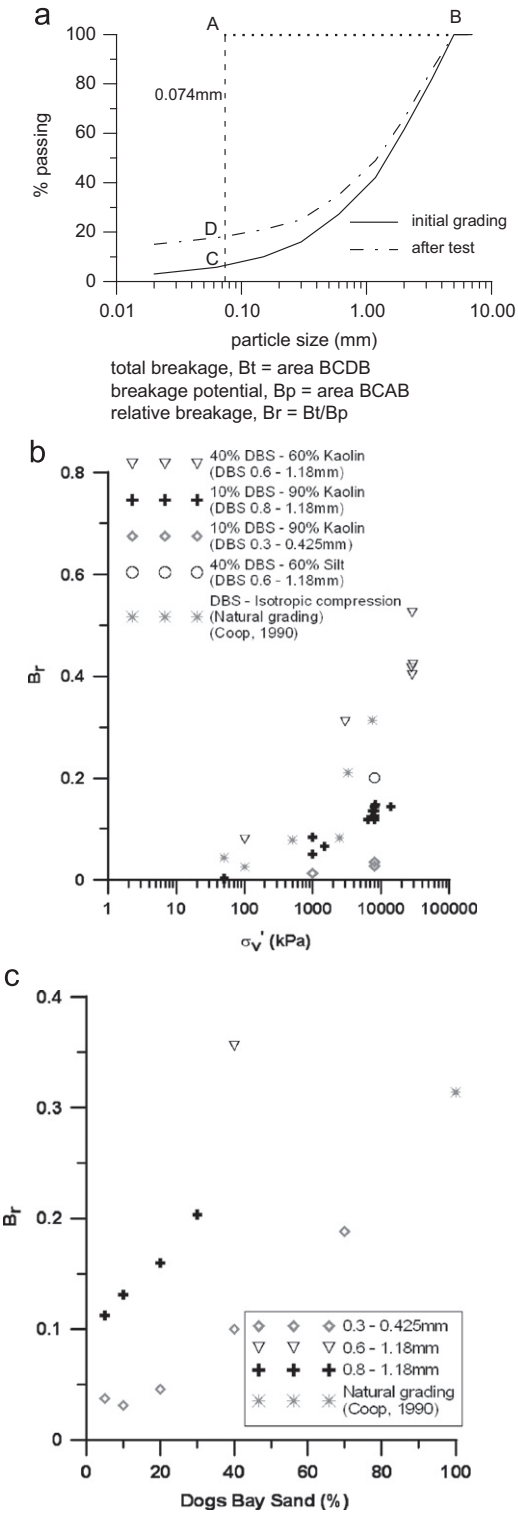

Fig. 4. Particle breakage of the sand fraction for the Dogs Bay carbonate sand: kaolin mixtures. (a) Definition of relative breakage (Hardin, 1985). (b) Influence of stress level. (c) Influence of sand content.

used in the low sand content samples, often as low as 1–2 g, which would have been very difficult to sieve accurately. For comparison the data for isotropic compression of the Dogs Bay sand in its original grading ($D_{50}=0.20$ mm) from Coop (1990) are included on the plots, which were again done by sieving.

Fig. 4b shows the effect of the stress level and Fig. 4c that of the sand content for tests taken to maximum stresses of about 8 MPa (± 0.5 MPa). For the isotropic compression data and those for the 40% sand content of the 0.6–1.18 mm grading a mean interpolated point was used as there was no specific data point near 8 MPa. The effect of stress level for the 40% sand content data resemble that which was determined by Coop (1990) for isotropic compression of the natural grading, with breakage starting at around a few tens of kPa. Although the amount of breakage is very much less for the 10% sand content, it is still significant, which is perhaps surprising since the granular void ratios $\{e_g=(e+fc)/(1-fc)$, where fc is fines content$\}$ of this mixture are in excess of 20, so that the sand particles are most unlikely to touch each other. It might have been expected that the cushioning effect of the fines around the "floating" sand particles would have prevented breakage and further research is needed to determine the nature of this breakage and why it occurs.

The much larger amount of breakage in the 40% sand samples and the fact that the breakage starts at around 100 kPa, which is also about where the knee occurs in the compression paths, tends to confirm the idea that at these sand contents the compression behaviour is essentially of a "sand" type, even if the values of e_g are still in the approximate range 1.8–7.8, which might indicate that the number of direct sand–sand particle contacts may be limited, especially at the higher void ratios.

It is also interesting that the amount of breakage depends on the particle size of the Dogs Bay sand, even if there was no discernable effect on the compression curves, indicating that these small amounts of breakage in a small fraction of the soil have no measurable effect on the volumetric behaviour. This is highlighted in Fig. 4c, where it is clear that the coarser particles are more prone to break and also that the amount of breakage decreases as the proportion of sand reduces. The data in Fig. 4c seem to show some incompatibility between the two data sets for the different techniques for measuring the particle size distribution, the data for size ranges 0.6–1.18 mm that were done by sieving perhaps giving slightly higher B_r values than those for the 0.8–1.18 mm range that were measured by the laser analyser.

The particle breakages seen here for gap-graded mixtures of clay with a fragile carbonate sand are in contrast to Nocilla et al. (2006) who found that for well-graded soils with transitional behaviour there was no measurable breakage. It therefore appears that particle breakage may occur in soils with transitional or non-convergent compression behaviour, but it is not a prerequisite.

5.2. Carbonate sand–quartz silt mixtures

Oedometer tests were also conducted on mixtures of Dogs Bay sand and crushed quartz silt (HPF4) to investigate whether the mineralogy and particle size of the fine fraction had an effect on the compression and amount of particle breakage. The Dogs Bay sand grading used was 0.425–0.6 mm and the average estimated accuracy of the specific volumes for these tests was about ± 0.01.

For the pure silt (Fig. 5a) the compression curves gently converge at high stress levels, although a QicPic analysis of the tested soil was unable to identify any breakage. This is an especially interesting result because it means that in clastic soils the knee and large plastic strains in compression are not necessarily associated with any major particle breakage. The QicPic has a resolution, however, of a few μm, so although it would identify any significant change in the major part of the grading curve, it may miss breakage of the finest particles.

Convergence is also seen for the 40 and 60% Dogs Bay sand mixtures (Fig. 5b and c) and the knee point in the compression curves tends to become sharper and move to lower stresses and higher specific volumes as the proportion of the Dogs Bay sand increases. It is clear from these tests that whether convergent compression behaviour is seen or not depends critically on the nature of the fines, especially whether they are plastic or non-plastic. For one sample of the 40% Dogs Bay sand: 60% silt mixture the B_r value was measured for the sand fraction taken after a test to about 9 MPa (although for this test the sand particles were between 0.6 and 1.18 mm). The data point is shown on Fig. 4b and emphasises that the amount of breakage in the sand–kaolin and sand–silt mixtures is fairly similar, confirming that the mineralogy and particle size of the matrix material does not affect greatly the amount of particle breakage.

5.3. Carbonate sand–quartz sand mixtures

Mixtures of two sands of very different mineralogies and particle types were made; Dogs Bay sand and Thames valley sand. The carbonate Dogs Bay sand has weak shelly particles that are more angular while the quartz Thames Valley sand has much stronger, more rounded and solid particles. The Thames Valley sand was tested in its natural grading shown in Fig. 2 and for the Dogs Bay sand the 0.212–0.3 mm sieve interval was used due to constraints on the amount of sand available. The samples were all made by dry compaction at a variety of initial densities.

In total six different mixtures were tested, the proportions of Dogs Bay sand being 0, 10, 30, 50, 70 and 100%. The pure Dogs Bay sand and the pure Thames Valley sand gave unique one dimensional NCLs with well-defined knee points as expected from previous research on clean, poorly graded sands of single mineralogy (e.g., Coop and Lee, 1993; Pestana and Whittle, 1995). The compression curves for two of the intermediate mixtures are given in Fig. 6.

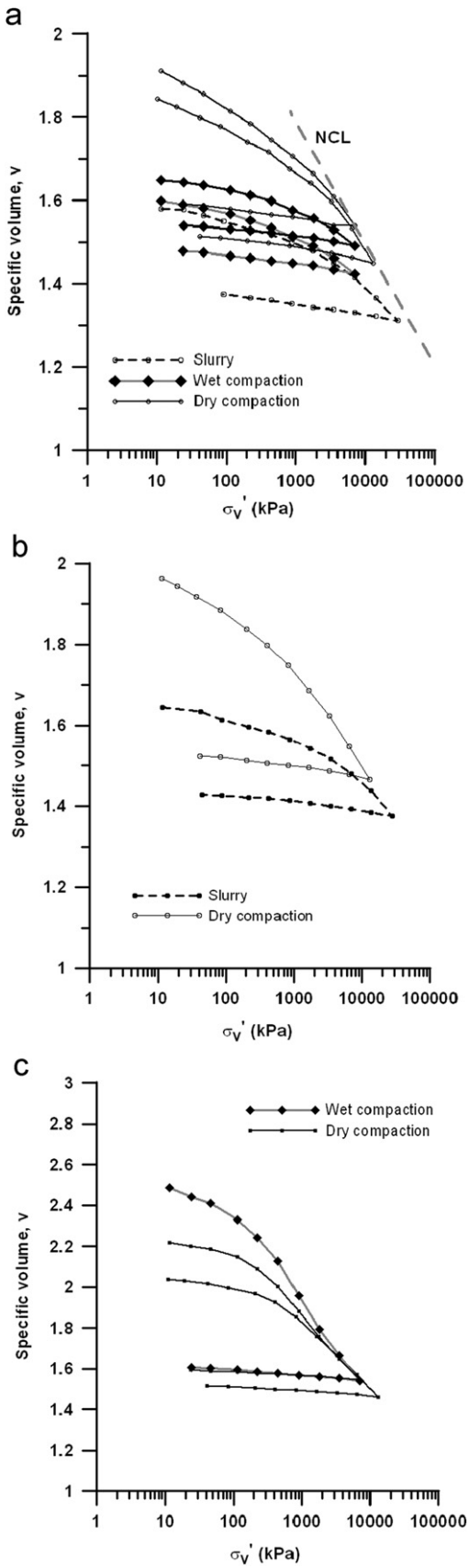

Fig. 5. Oedometer test data for mixtures of Dogs Bay carbonate sand of 0.425–0.6 mm particle size with crushed quartz silt. (a) 100% silt. (b) 40% Dogs Bay sand and 60% silt. (c) 60% Dogs Bay sand and 40% silt.

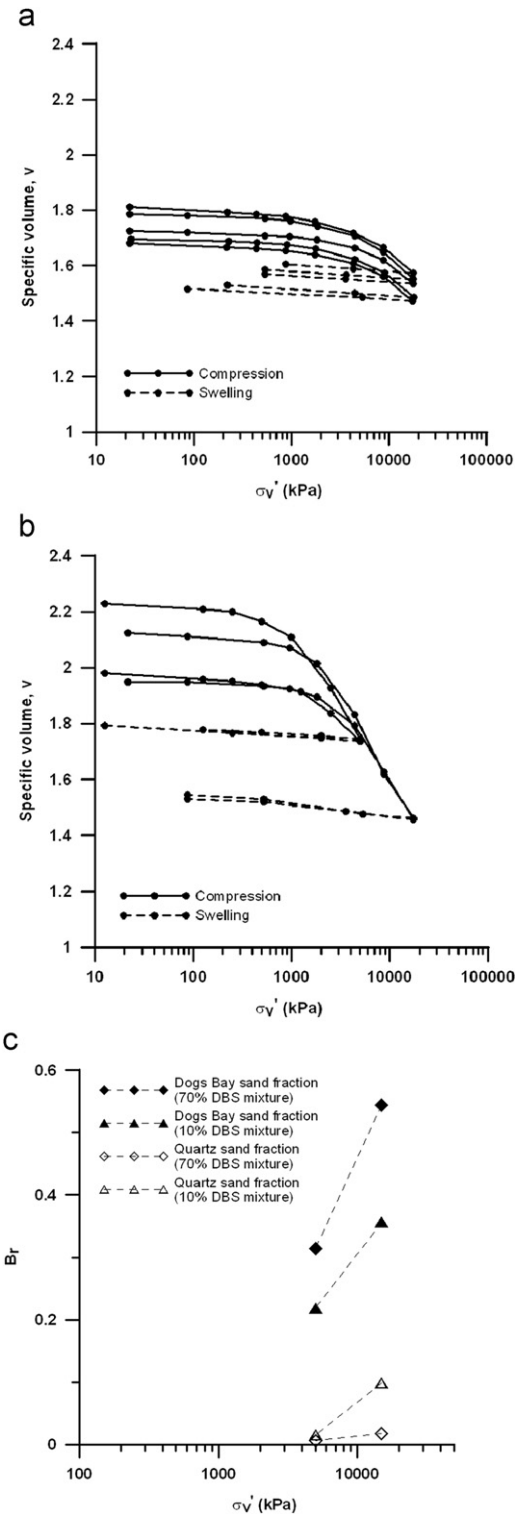

Fig. 6. Oedometer test data and relative breakage for mixtures of Dogs Bay carbonate sand and Thames Valley quartz sand. (a) 10% Dogs Bay sand. (b) 70% Dogs Bay sand. (c) Relative breakage of the two sand fractions.

As might also be expected, as the Dogs Bay sand content is increased, the initial specific volume increases, due to the particle angularity. The knee point where the NCL is reached tends to occur at lower stresses and higher specific

volumes and the gradients of the compression paths at higher stress levels increase. However, at 10% Dogs Bay sand content there is a clear non-convergent compression behaviour and a less distinct knee, while at 70% content the compression curves converge, with more distinct knee points as the NCL is reached. The average accuracy of the specific volumes for these tests was again estimated to be about ± 0.02. The 50% Dogs Bay sand mixture also gave a unique NCL while the 30% mixture did not (these data have not been included for brevity). The type of compression behaviour seen is therefore quite sensitive to the proportions of the different sands used.

The breakage of each of the sand fractions was measured by first taking the final grading of the whole sample, dissolving and washing out the carbonate sand with Hydrochloric acid and finally repeating the grading. Because the QicPic laser scanner destroys the samples it tests, the gradings were done in this case by careful wet sieving. Clearly there is substantial breakage whether or not a unique NCL is found and as expected there is much more breakage in the carbonate sand than the quartz. It is also interesting that the amount of breakage in the Dogs Bay sand actually decreases as the proportion of that sand reduces. This is possibly the effect of the increasing initial density of the samples with greater quartz sand content, giving higher coordination numbers, and hence lower contact forces between particles. It is also possible that the quartz sand has a protective effect, providing a rigid matrix sheltering the weaker Dogs Bay sand particles.

5.4. Quartz sand–clay and quartz sand–crushed quartz silt mixtures

Oedometer tests were carried out on mixtures of Thames Valley sand and kaolin. A range of proportions of quartz sand were used from 25 up to 90% at intervals of 5–10%, along with tests on the pure kaolin and pure sand. With the exception of the clean sand all of the samples were made as slurries at a variety of different initial water contents.

Data from selected gradings are shown in Fig. 7. They are presented in terms of granular specific volume ($v_g = 1 + e_g$) for the sand dominated samples and fines specific volume ($v_f = 1 + e_f$ where $e_f = e/fc$) for the clay dominated ones. The estimated accuracy of the tests was about ± 0.03. The effect of using either is merely to translate the compression curves but not to change the convergence. For low sand contents (25%, Fig. 7a) the effect of the addition of the sand is to translate the compression curve downwards in the v:logσ'_v plane, maintaining the same gradient as for pure kaolin, but when the data are plotted in terms of v_f the translation is

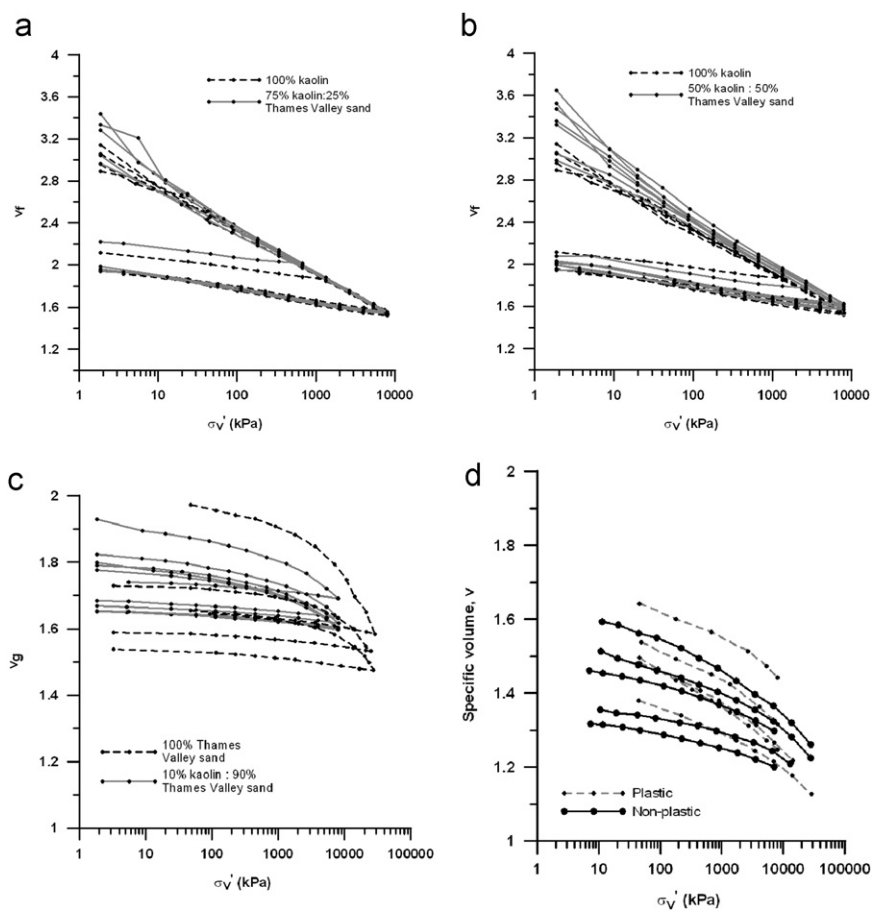

Fig. 7. Oedometer test data for Thames Valley quartz sand: kaolin and Redhill quartz sand: crushed quartz silt mixtures. (a) 0% and 25% sand. (b) 0% and 50% sand. (c) 100 and 90% sand. (d) 75% sand with kaolin ("plastic") and crushed quartz silt ("non-plastic").

removed, indicating that the coarse fraction essentially has no influence on the compression behaviour and is just an inert addition making the soil slighter denser. At higher sand contents (50%, Fig. 7b) the effect of adding the fines can no longer be normalised with v_f and the compression curves of the mixture plot above those of the kaolin when normalised in this way. The coarse fraction is therefore starting to affect the compression. The data are a little scattered for the 50% sand content, but still within the estimated error of the values of v_f. For the pure sand (Fig. 7c) the data tends towards a unique line at large stresses, although they were not quite loaded far enough to reach it. However, with a 10% clay content, although the compression curves plot in the same area when v_g is used, the data no longer converge so convincingly and the knee is less clear. The same was found for a 25% clay content, but from 30% upwards the behaviour was of a clay type similar to that in Fig. 7a and b, with no clear knee point and a unique NCL. These tests may be contrasted with those on the carbonate sand: kaolin mixtures, for which a much clearer non-convergent compression behaviour was seen over a much wider range of sand contents, and at much lower sand contents, emphasising how sensitive the mode of behaviour is to the sand type.

Data for the 75% sand content are shown in Fig. 7d where they are compared with oedometer tests on a quartz sand: crushed quartz silt mixture. The latter mixture is especially interesting as it has a uniform mineralogy but a gap grading, although problems with sand supply meant that Redhill sand had to be used, which is slightly coarser than the Thames Valley sand (Fig. 2). Although in both cases there is some convergence of the compression curves, they clearly cannot reach and form any unique NCL if a linear v axis is used before the curves approach the limiting $v=1$, at which point the curves must converge as they approach their horizontal asymptote. This is consistent with the findings of Martins et al. (2001) for their quartz sand: kaolin mix. The data for the "non-plastic" quartz sand: crushed quartz silt mixture contrast strongly with the carbonate sand: crushed quartz silt mixtures for which convergent compression behaviour was found. The effect of the plasticity of the added fines was therefore quite different in the quartz sand to the carbonate sand, since for the carbonate sand transitional behaviour was only seen for some mixtures with plastic fines (i.e., kaolin) and not for any grading with the non-plastic fines (i.e., the crushed quartz). Again in contrast to the similar mixtures using the carbonate sand, no breakage was detectable in the quartz sand fraction after the tests using the QicPic analyser.

5.5. Sand–silt–clay mixtures

A short series of tests was carried out on a mix of quartz sand, crushed quartz silt and kaolin, in the proportions 25%: 25%: 50% as measured by dry weight. The quartz sand used in this case was the original Ham River sand, the gradings curve of which is also in Fig. 2. The samples were compressed from slurries and the estimated accuracy of these tests was again ± 0.03. The compression data in

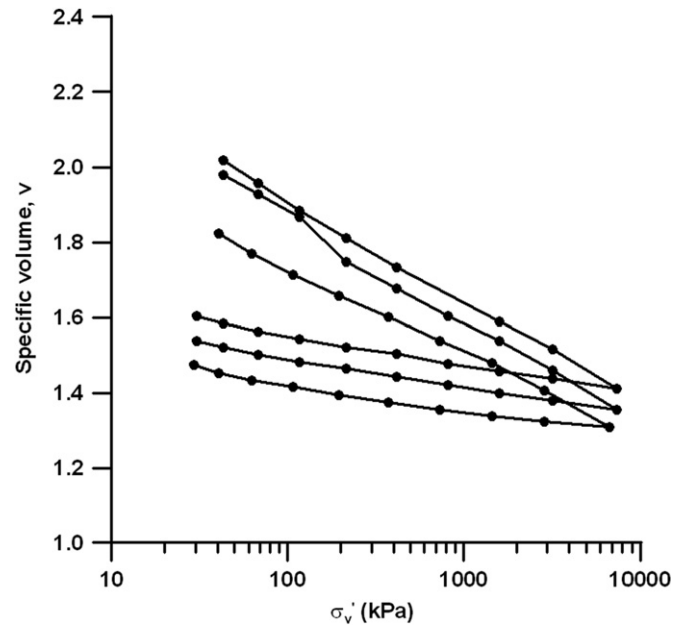

Fig. 8. Oedometer test data for quartz sand: quartz silt: kaolin mixture.

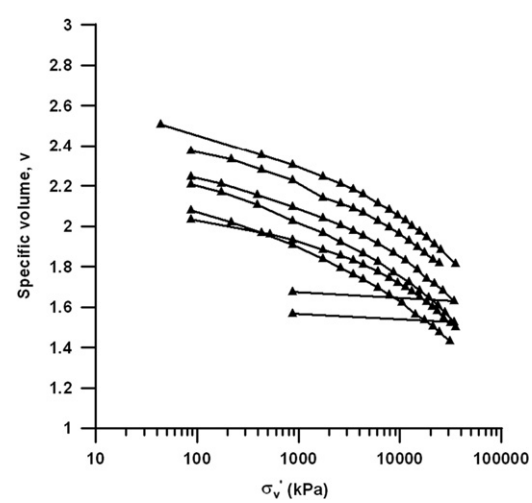

Fig. 9. Oedometer tests on reconstituted high fines content Thanet sand (data from Ventouras, 2009).

Fig. 8 indicate that this well graded artificial soil is clearly has non-convergent compression behaviour, as Nocilla et al. (2006) had found for controlled gradings of a naturally well-graded silty sediment.

Ventouras and Coop (2009) examined the behaviour of the two distinct sub-strata that are found within the Thanet sand; a low fines content stratum (5–10% < 63 μm) at shallower depths with a higher fines content stratum below (15–30% < 63 μm); the gradings curve for the high fines content stratum is given on Fig. 2. The compression behaviour has been reanalysed from the original data and oedometer compression curves for the high fines content sub-stratum, which may be described as a clayey-silty-sand, are shown in Fig. 9. The reconstituted samples were made by dry compaction and they have an estimated accuracy of

the specific volumes of ± 0.02. They therefore indicate an apparently non-convergent compression behaviour with no clear knee and unique NCL being identified even for stresses up to 30 MPa. It is however possible, that if still higher stresses were reached, some convergence might be seen. In contrast the soil from the low fines stratum gave convergent compression curves, by similar stresses.

5.6. Clays

Initially it was assumed that non-convergent compression and transitional behaviour would be limited to intermediate graded or gap-graded soils and that unique intrinsic properties would be seen for slurried samples of clays. Laboratory tests were therefore carried out on three reconstituted clays with different plasticities, London clay (from Unit B2) (LL=69% PL=27%), Oxford clay (LL=70% PL=25%) and Speswhite kaolin (LL=62% PL=32%). In each case samples were made either as slurries at different water contents or by dry compaction at different initial densities, although no difference in behaviour could be found from the preparation method, and so this has been omitted from the graphs for clarity. The compression data for the kaolin are shown in Fig. 10. The behaviour of the other two clays was similar. The data are slightly scattered but they seem to indicate some non-convergence. To check in more detail to what extent the curves are converging, the relationship between the initial specific volume, v_i, and the value of v at σ_v' values of 0.1 and 1 MPa are plotted and are shown in Fig. 11 for each clay. In this plot a horizontal line would indicate full convergence and a line at 45° would indicate completely parallel compression paths. It was found to be much more difficult to achieve accurate specific volumes in the very much more compressible clays, and the mean accuracy of the specific volumes for these tests was only around ± 0.07, although, as discussed above this is very likely to be an overestimate. Even allowing for the poorer accuracy though, it is clear that at 100 kPa the compression

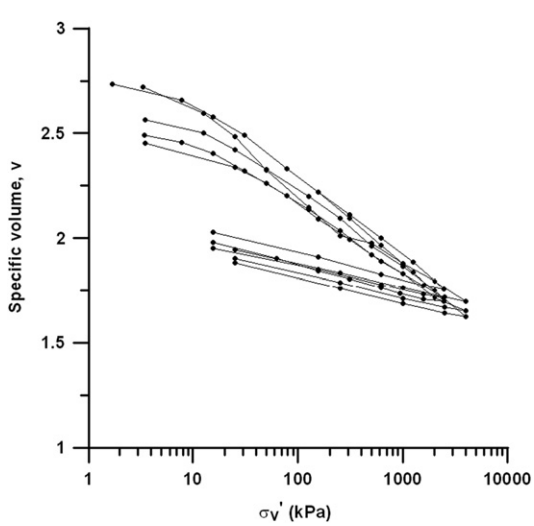

Fig. 10. One-dimensional compression behaviour of reconstituted Kaolin.

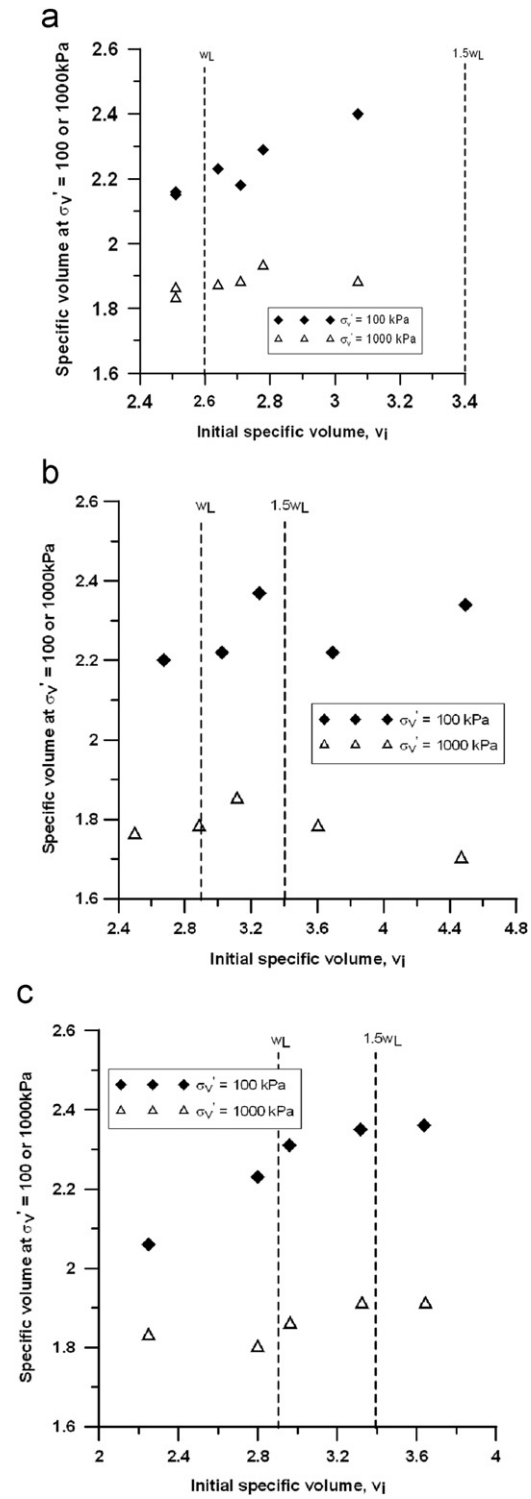

Fig. 11. Variation of specific volumes at 100 and 1000 kPa with initial specific volume for three clays. (a) Kaolin. (b) London clay. (c) Oxford clay.

paths for the Oxford clay and kaolin have not converged, and even for the London clay the differences measured are at the limit of what could be explained by the accuracy of the measurements. At 1 MPa the differences are much smaller and more accurate tests would be needed to confirm whether there is complete convergence or otherwise.

Burland (1990) recommended that in order to define unique properties for the compression of a reconstituted clay, slurried samples should be made at water contents between 1 and 1.5 e_{LL} (the void ratio at the Liquid Limit). His normalised void index is defined in terms of the void ratio at 100 kPa and 1 MPa for reconstituted samples created in this range, assuming that the clays will have reached a standard fabric and hence unique void ratio at 100 kPa. In each case, there is little evidence that the trends between 1 and 1.5 e_{LL} are different to those outside the range. It can therefore be tentatively concluded that even clays may show slight evidence of non-convergent compression behaviour with the implication, as Ferreira and Bica (2006) have identified, that if a unique intrinsic compression line cannot be identified, it is not possible to quantify the effects of structure in a natural soil objectively.

6. Conclusions

The factors that control whether non-convergent behaviour is seen in compression are clearly complex. Shipton (2010) collated all of the gradings curves for soils showing non-convergent compression and full transitional behaviour, as shown in Fig. 12, including data from the literature, the tests data from this paper and other unpublished tests carried out at Imperial College. They cover a wide range of gradings from clayey-silts to well-graded sands and gap-graded soils, so obviously grading alone is no indicator of whether transitional or non-convergent compression behaviour will be seen. Referring to the summary of the existing data from this paper and the literature in Table 1, it is clear that relatively small changes to the nature or quantity of the constituent particles of a soil can change the behaviour quite dramatically. It also seems that non-convergent compression behaviour tends to occur in soils of mixed grading, whether gap or well-graded, and soils with mixed mineralogy or particle types. Even the identification of unique NCLs in reconstituted clays may not be as simple as previously thought. Particle breakage can occur in soils with non-convergent compression behaviour but it is not a prerequisite.

Much early research on the application of the critical state framework to sands tended to focus on poorly graded sands of single mineralogy, often for simplicity in creating repeatable samples or identifying the effects of particle breakage. Although the behaviour observed for clean sands had some differences with that of clays, especially in the effects of the particle breakage, the applicability of a critical state type of framework to both has perhaps tended to obscure the fact that there may be many soils of intermediate grading and/or mixed mineralogy that have transitional or at least non-convergent compression behaviour. Since it appears that there are no simple means to predict whether transitional or non-convergent compression behaviour will occur or not, each soil must be assessed individually.

While for some of the soils investigated the pattern of convergent or non-convergent behaviour is clear even at modest stresses, in other cases very high stresses have been used in order to confirm whether convergence occurs or not. This may seem to be an academic exercise, but the compression behaviour at lower stresses will be influenced by whether compression curves from different initial densities are tending to converge to a unique line or not. Many constitutive models are also based on volumetric hardening which is linked to the isotropic Normal Compression Line, to which that in oedometric compression is also linked.

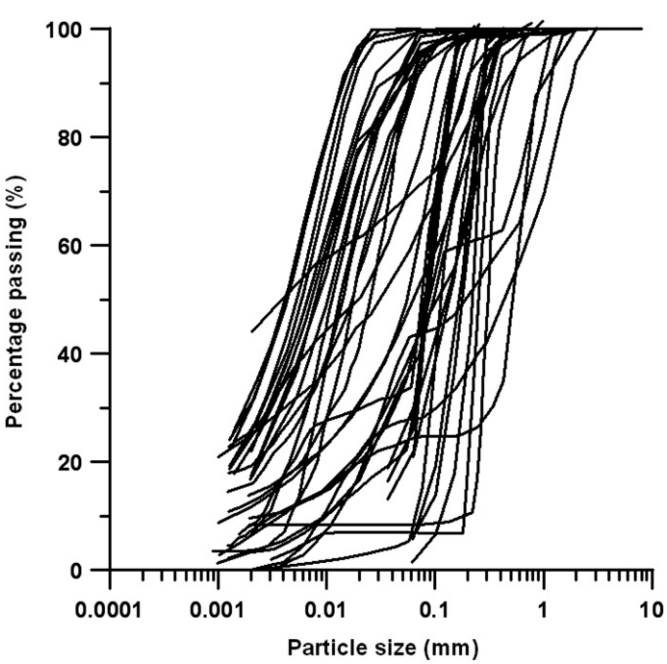

Fig. 12. Grading curves of soils which show transitional and non-convergent compression behaviour from Shipton (2010).

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