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Change of Mechanical behavior of a compacted well-graded granular material with and without cement

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ABSTRACT: *Concrete increases improve the presentation of granular soils. Be that as it may, most writing instances of concrete augmentations are in inadequately evaluated sands, either to imitate the conduct of sandstones or to complement the mechanical differences among established and uncemented soils. In this article, the conduct of an all around evaluated granular soil, utilized for base and sub-base of streets, was concentrated by doing triaxial tests on solidified and uncemented tests. Tests were compacted to accomplish a thick texture and tried at stresses ordinarily utilized in prac-tice. Sieving was utilized to comprehend if breakage is significant and to decide the grain size disseminations of the examples after compaction and shearing. The outcomes show that the expansion of little rates of concrete extraordinarily increment stiffness and widening. Along these lines, creating bigger qualities; this is especially significant at low confining stresses in streets and stopping regions, where this material is usually utilized. Everywhere strains, the outcomes show that different Critical State Lines exist for both the uncemented and solidified soils. Each line has a different slant, which is accepted to be the aftereffect of the development of the grain size appropriation of the established soil. The standardized information demonstrate that an extraordinary state limit surface can be resolved for every one of the three tried soils.*

Keywords*: Cemented soil; Compacted soil; Triaxial test; Critical state; Base and sub base; Granular soil*

INTRODUCTION

Research in uncemented granular material has highlighted the importance of breakage, where the onset of breakage in the NCL is a function of the mineralogy of the grains [12]. Breakage is important as it also marks the location of the CSL, and many researchers have shown that by changing the grain size distribution, the CSL will also change (Thevanayagam et al., 2002, Carrera et al., 2011; Xiaoet al., 2016). In structured sands, only a few researchers considered the changes in particle size distribution (PSD) and its eff ect on altering the location of the CSLs and NCLs, when compared to the uncemented samples (Cuccovillo and Coop, 1997a; Marri et al., 2012). Certain results have shown that due to cementation, the resultant CSL would have a reduced gradient (Cuccovillo andCoop, 1997a), whilst others have shown that the cementa- tion increases the gradient of CSL (Schnaid et al., 2001). In the aforementioned research, it is not clear if the alterations of the CSL gradient are due to particle breakage, bond degradation or a combination of both. Diff erent critical state lines for the same samples with diff erent cement con- tents are also reported by Cruz et al. (2011). The DEM results have shown that alterations in the CSL are due to the breakage of the bonds and the generation of a diff erent grain size distribution, as some of the particles are still cemented together (Yu et al., 2014; Yu et al., 2015). The alteration of the CSL due to breakage was also investigated by Ghafghazi et al. (2014), where they claimed that break- age causes a downward parallel shift in the CSL, and according to Bandini and Coop (2011), large amounts of breakageareneededforsignificantchangestooccur.

In the majority of the research encountered so far, the samples were prepared using poorly graded granular mate- rials (sands of aeolian origin) or with lower densities; this was done in order to accentuate the breakage or the improvement caused by the binding agent added. In a cou- ple of articles (Rios et al., 2014; Consoli et al., 2014) well graded residual soils are reinforced with cement, however some of them have fines and there is no attempt to measure or determine the breakage.

When well graded soils are used the research tends to concentrate on the mechanical properties of the material at small strains i.e. stiff ness and strength up to peak, using multiple-step loading triaxial tests (Kongsukprasert andTatsuoka, 2007and Taheri et al., 2012). These tests have the advantage of allowing the use of a single sample to cover a large range of stresses, however it is unclear what the eff ect of damage to the cement bonds and particle breakage is from the previous loading steps. Hence, the eff ect of the addition of cement on manmade materials used forengineeringpurposesisnotverywellunderstood.

The purpose of this paper is then to study the eff ect of small levels of cementation on a very dense fabric, created by compaction of a well graded granular material, under monotonic loading, on commonly used soils. The improve- ment of the mechanical properties and examines the eff ects of cementation within the Critical State framework is also explored.

1. Materialtested

The soil used in this research was a crushed limestone with 88% CaCo3, collected from a depot in South London and is currently used commercially for the bases of roads in Southern England. The soil was wet sieved and each parti- cle size range was stored in separate bags. The main prop- erties of this material are summarised in Table 1, with the particle size distribution (PSD) shown in Fig. 1, together with the range defined by the UK Highways Agency (2016)forabaseandsub-basetype.

The idealised grading curve proposed by Fuller andThompson (1907) is based on the idea that when larger par- ticles are in contact with each other, larger voids are gener- ated and occupied by intermediate particles; this procedure is then followed to the smallest size available. Theidealised curve generates dense fabrics and was then used to correct the initial grading curve of the soil, given that the particles are not spherical, it is argued that it then does not generate the densest possible fabric. Given the sizes of the particles available, the PSD named ‘‘Adjusted grading” (Fig. 1) was used for all the tests. This curve follows the Fuller curvefor the largest sizes, and below the size 3.35 mm it was trans- lated downwards as not enough material was available. For the same reason, sizes below 0.425 were chosen to make sure that all samples would have the same grain size distribution and the grading within the Type 1, as defined by the UK Highway Agency (2016)for a base and sub- base. As the triaxial equipment used is capable to test sam- ples up to 100 mm diameter and 200 mm high, the grain size distribution was truncated at 20mm.

Cemented samples were created by adding Portland cement classified as CEM1, in accordance to the British Standards (BSEN 1197-1:2011). Given the high strength of the compacted samples, only small percentages of cement (1 and 2%) were used to generate modest changes instrengththatcouldbetestedinatriaxialequipment.

2. Apparatus and samplepreparation

A computer-controlled triaxial apparatus, with a local strain measurement system capable of measuring 10^{-6} strain, similar to Cuccovillo and Coop (1997b), was used for the conventional triaxial tests (Fig. 2). The system uses RDP electronics LVDTs (model D6/05000) attached to a modulator/demodulator (model S7DC) that allow the full configuration of the output electric signal. At the beginning of the shearing, the local instruments are reset to zero to take advantage of the 16-bit auto scale of the data logger. The volumetric strain was measured using the volume gauge and the localinstrumentation.

The desired amount of each fraction of soil was thor- oughly mixed in a tray, with diff erent moisture contents, before being compacted in 5 layers, using 27 blows of a 5 kg hammer falling from a height of 450 mm (BS1377-4, 1990). A compaction curve for the uncemented soil wasdeterminedinordertodefinetheoptimummoisture

Table 1
The index properties of research material from dry sieving.

Description	Crushedlimestone
Max density-vibratinghammer(g/cm^3)	2.24
Maxvoidratio	0.83
Mindensity(g/cm^3)	1.51
Minvoidratio	0.23
Particledensity(gr/cm^3)	2.76
Max dry density-automatic heavy compaction (g/cm^3)	2.24
Optimum water content (modifiedproctor)	6%
Typeofsoil	GW
D_{10}(mm)	0.2
D_{30}(mm)	1.5
D_{50}(mm)	3
D_{60}(mm)	4
Uniformity coefficient $C_u = \frac{D_{60}}{D_{10}}$	20
Curvature coefficient- $C_c = \frac{D_{30}^2}{D_{10} \times D_{60}}$	2.8

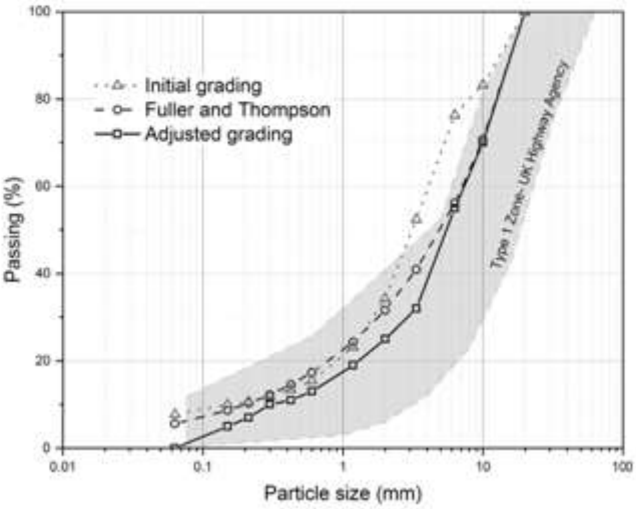

Fig. 1. Particle size distributions: Initial grading; Fuller and Thompson(1907)andtheAdjustedgrading.Thegreybanddefinestherangeoftype1(UK Highway Agency,2016).

content. Given that the cemented and uncemented sampleswere tested, a decision was made to compact all sampleswith a moisture content of around 10%, providing enoughwater for cement hydration at the cost of a lower dry den- sity. Table 2 contains the properties of every sample tested.After compaction, the uncemented samples were trans-ported to the pedestal of the triaxial equipment for testing. Suction maintained that the sample was intact before theconfining pressure was applied and the percolation proce-dure started.

The cemented samples were prepared with two diff erent percentages of cement (45 g for 1% and 90 g for 2% cement); where an equivalent mass was removed from the smallest grading of the samples' PSD. The sample prepara- tion followed a similar procedure to the one described above, except that the cement and dry soil were thoroughly

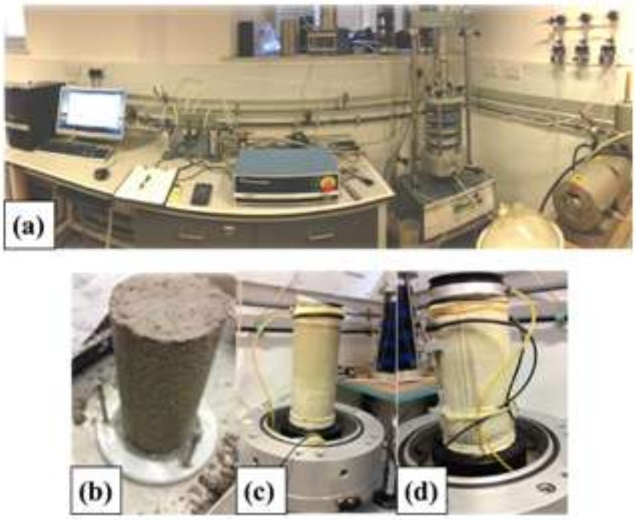

Fig. 2. Picture of the equipment and samples: (a) triaxial equipment in the lab; (b) compacted sample in the pedestal and sample with local instrumentation before closing the triaxial chamber.

mixed together before adding the same wateramount,mix-ingwas thencontinueduntilhomogeneitywasachieved.After compaction the sample and mould wereputinsideof a plastic bag and allowed to cure for24 h. Thesamplewasthenplacedinsideofatankwithwaterat22°Candallowedtocure,submerged,foranother4days.Atday5the sample was removed from the tank, mounted onthetri-axial pedestal and prepared for testing. This procedure was followed to guarantee that the cement couldhyd ratefully.Avolumeinexcessof2000ccofwaterwaspercolatedthroughthesampletoremoveairbubbles.Thepressu reapplied was around 18 kPa, caused by thediff erencein height between a water containerlocatedapproximately2 m above the sample and the outlet from thepedestal.The watercomingfromthesamplewasclearandtheauthors believe that no cement particleswereremovedfromthesampleduringpercolation.Thesamplewasthensaturated, maintaining an eff ective stress of 15 kPa.AB-parametertestwasperformedatdiff erentbackpressures,from 100 to 350 kPa on the first sample. Whilstitincreasedslightlyupto250kPa,theincreasesat300and350kPawhere negligible (values measured were of the orderof0.86to0.92).Itisimportanttomentionthatthevolumegauge only changed slightly during the increase in pres- sures, being fairly constant once the required pressures were maintained. This indicated that the sample was satu- rated and the B-parameters would not reach the required value. Therefore, a minimum back pressure of 250 kPa was used in all drained tests and monitored by another transduceratthetopofthesample.

In the consolidation stage the eff ective stress was raised to the value used in the test. This procedure would take around 2 to 3 days before shearing; however, the cemented samples were all sheared in a drained way at the end of day

Table 2

Properties of the samples tested: the name indicates the cement percentage and the confining stress used during shearing.

Test Name	W_0 (%)	* e_0	** e_{con}	c_{Dry} (g/cm^3)	q_{max}(kPa)
M-0%-020	9.61	0.379	0.378	1.952	301
M-0%-050	8.63	0.376	0.370	1.961	482
M-0%-100	9.66	0.417	0.405	1.938	804
M-0%-200	10.03	0.404	0.388	1.978	1177
M-0%-400	9.71	0.400	0.374	1.973	2067
M-1%-050	9.59	0.397	0.392	1.958	941
M-1%-100	8.44	0.367	0.357	1.974	1219
M-1%-200	8.72	0.392	0.364	1.968	1910
M-1%-300	9.83	0.410	0.381	1.943	2247
M-1%-400	9.03	0.393	0.350	1.965	2755
M-2%-020	9.21	0.380	0.380	1.961	809
M-2%-050	8.74	0.388	0.383	1.960	1359
M-2%-100	8.61	0.371	0.361	1.972	1656
M-2%-200	9.04	0.396	0.380	1.951	2157
M-2%-300	8.67	0.388	0.360	1.973	2718

* e_0 – initial void ratio.

** e_{con} is the void ratio after consolidation, before shearing.

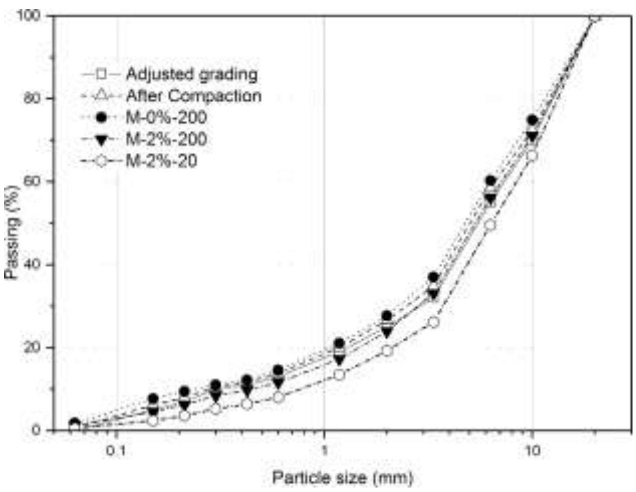

Fig. 3. Particle size distributions of the samples: original and after the compaction and monotonic shearing.

7. Therefore, the consolidation stage was extended even if the volume change wasnegligible.

During drained shearing a constant confining pressure was maintained throughout the test. The sample was sheared at a constant rate of strain of 0.016 mm/min, this was determined based on the capacity of the data acquisi- tion system to interpret strains of 10^{-6}. Well after peak strength, the speed was doubled until the test was termi- nated. Tests were terminated either by achieving a constant strength and a constant volume, or by reaching an axial strain of30%.

The moisture content was determined by using left over soil from the tray, and the initial void ratio of each sample was calculated in five diﬀ erent ways: an average of the ini- tial dimensions, the volume of voids and solids, the dry unit weight and the final water content. Outliers were removedand an average of the values deemed acceptable was used. Although the samples were carefully prepared, a variation in the initial void ratio was unavoidable (Table 2).

4. Breakage

The idea of having a balance between particle breakage and particle rearrangement at critical state has been reported by many authors [(Chandler 1985, Daouadjiet al. 2001, Coop et al. 2004, Salim and Indraratna 2004,Muir Wood and Maeda 2008; Rubin and Einav 2011). The literature review has also shown that the level of breakage in dense granular materials, at lower pressures, is rarely investigated. It is often disregarded and assumed not to aﬀ ect critical state, particularly when a large number of contacts is expected. Therefore, samples were sieved to evaluate breakage after shearing, whilst extra samples were prepared to determine if compaction caused particle break- age (Fig.3).

The results showed that after compaction small amounts of breakage can be seen in all sizes. Breakage was alsoseen after shearing, where the largest changes in PSD were seen in sizes ranging from 1 to 7 mm, where the increase inpass- ing percentage is in the order of 8%. The smaller sizes have also increased, perhaps indicating the shearing of the asper- itiesatthesmallconfiningstressesused.

An attempt to determine the PSD for the cemented sam- ples was carried out by breaking the cement bonds before sieving, samples were put on individual sealed bags andthe bonding destroyed by hand. Larger pieces that were kept intact were removed by hand, and only the soil that seemed not to have bonds was used. The results showed that the PSD curve of sample M 2%-200 is slightly above the original curve for sizes above 3 mm and below the orig- inal curve for sizes below 3 mm; whilst the M 2%-20 is below the original size tested (Fig. 3). Thisdemonstrates

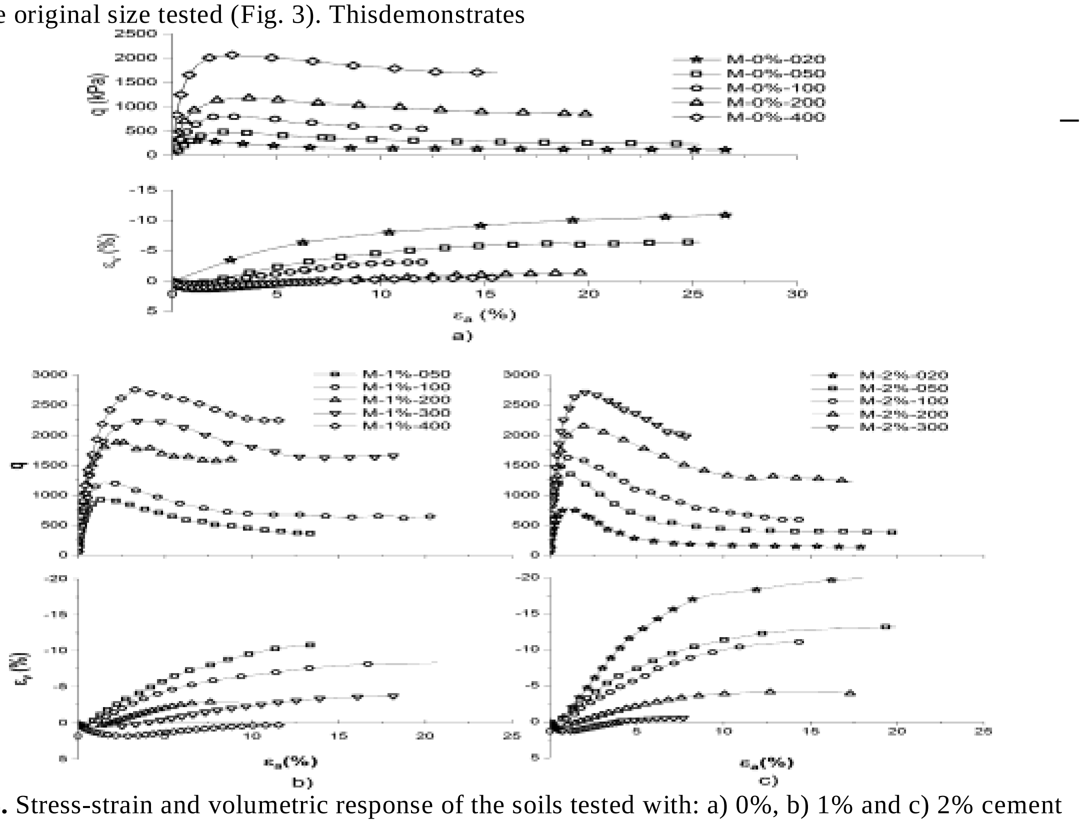

Fig. 4. Stress-strain and volumetric response of the soils tested with: a) 0%, b) 1% and c) 2% cement content.

the effect of the confining stress on the destruction of the bonds and the difficulty to destroy by hand, the bonds on the smallest sizes, even after a monotonic shearing has taken place.

5. Triaxial tests (stress-strainbehaviour)

The triaxial test results are shown on Fig. 4, where the stress-strain and the volumetric curves of 15 tests are plot- ted. As expected, all samples showed a strain-softening behaviour towards a constant strength after peak stress. The volumetric behaviour is similar; after a large dilation that reduces with the confining stress, it is possible to visu- alise a steady state, where no change in volume and strength is seen with the increase in shear strain. Large vol- umetric strains are also seen particularly at low stresses, where there are sharp changes in the volumetric behaviour, possibly indicating the occurrence of localisation. How- ever, all samples have failed in barrelling and only in a few samples signs of localisation were noticed. A couple of tests were terminated earlier, due to small punctures on the membrane given the large strains. The axial strain, e_a, was measured by two local displacement transducers up to a certain point (usually peak stress) switching to the external transducerafterwards.

The effect of the addition of cement in the strength is clear, as the peak values increase with the addition of cement for all confining stresses tested. Simultaneously, there is also an increase in the brittleness index (the ratio between the peak shear strength and the shear strength at large strains) of the samples. Fig. 5shows the brittleness index calculated for all of the samples. The samples with 2% cement have a much larger brittleness index when com- pared to the other samples. It is also clear that the values of brittlenessindexcalculatedforthe1%cementsampleshave little deviation from the 0% samples; i.e., the addition of 1%cementcausessmallchangestothisparameter.

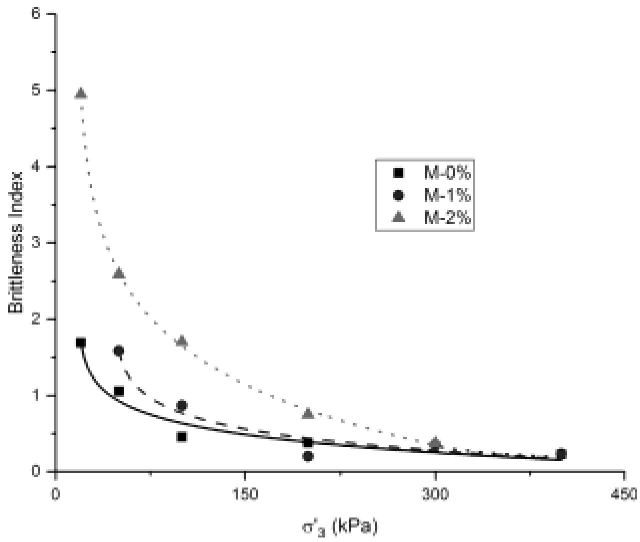

Fig. 5. The relationship between brittleness index and confining pressure.

The results also show that increasing thecementcontentreduces the level of strain required to reach thepeakstress.Thisistrueforeveryconfiningstresstested,confirmingthatthereisanincreaseinstiffnesswiththeadditionof cement. Fig. 6shows a direct comparisonbetweenthecemented and uncemented samples forcertainconfiningstresses, to better show the effect of cementation inthepeakshearstrengthandthevolumetricbehaviour,wherehigher cement content generates larger dilativevolumetricstrains.For the uncemented soil, the area correspondenttothemaximum rate of dilation directlycorresponds to thepeakstrength,whilstthecementedsamplesexperiencepeakslightly before the maximum rate of dilation.Thechangeseen is not large, but enough to demonstratethatsmalladditions of cement can generate a structure that affectthestrengthofevenverydensefabrics,asshowninFig.7.Fig.8showsthetangentstiffnesscurves(slopeofthestress-strain curves) with arrows indicating thepointswereachangeinshearingratewasperformedtoacceleratethetests. The gross yield points, regarded as the onsetofbond-ing degradation and the locus wheresignificantplasticdeformations start to occur, are also indicated inthecurveswith the use of a black square. Thesewere determinedusing the method proposed by Malandraki and Toll(1996) and Alvarado et al. (2012b)and are marked bythe start of the change in direction of the stiffnesscurve.Itisclearthattheadditionofcementincreasesthetangentstiffness;thetangentstiffnessoftheuncem entedsamplesstartatvalueslowerthan1GPa,whilstthesampleswith1%cementstartatvalueslowerthan2to3GPa,a ndthesampleswith2%showvalueslowerthan7or8GPa.Itcanalsobeseenthatincreasingthestrainreducesthetan- gent stiff ness and that the rate of reductionin stiff nessisrelatedtothepercentageofcement(i.e.lowercementper- centages lower reductions and highercementpercentageshigher reductions). Comparing tests in each group, itis also seenthatincreasingtheconfiningpressurealsoresultsin higherstiff nesses,despitethefactthatincertainsamplesthisisnotveryclearanditislikelytobetheeff ectofthe scales used in the graph.

5.1. Stress-Dilatancy

A stress-dilatancy analysis was performed and Fig. 9contains the plots of all 15 uncemented and cemented sam- ples, shown with the respective cement percentage. Oneach graph, the peak strength, the gross yield and last test point are represented by diff erent symbols. The uncemented sam- plesshowanincreaseintheratioq/p^{0}withdilation,uptoapeak, reached at the same time as the highest dilation rate. From that point onwards, dilation rate reduces together withtheq/p^{0}ratio.Asthevolumestopschanging,auniquevalue of M = 1.76 can be determined, corresponding to a friction angle at critical state $u'_{cs} = 43^{\circ}$.
Theeff ectofthecementpercentagecanbeseenbythe initial change in the shape of the curve. At the start of shearing,asdilationdevelops,thesamplesquicklyreach

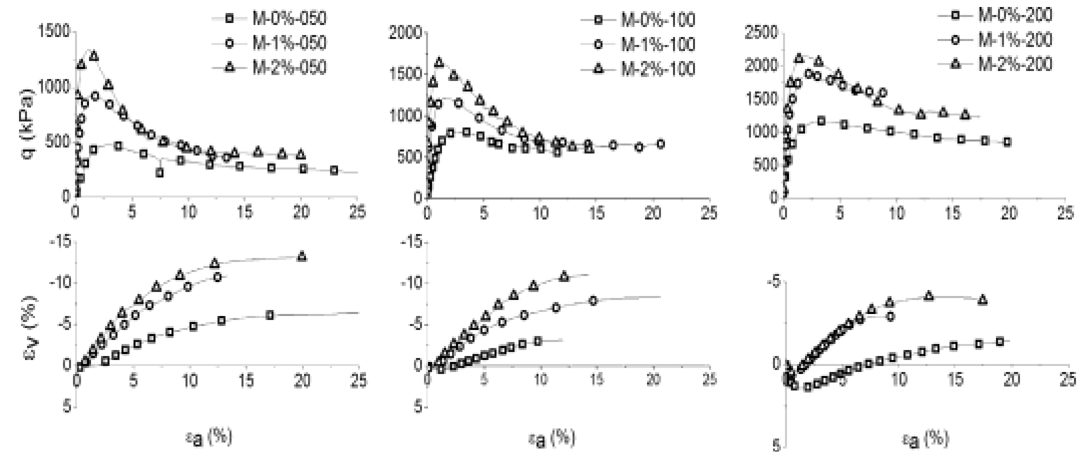

Fig. 6. Comparison of the stress-strain and volumetric responses of the samples with 0%, 1% and 2% cement, under diff erent confining pressures.

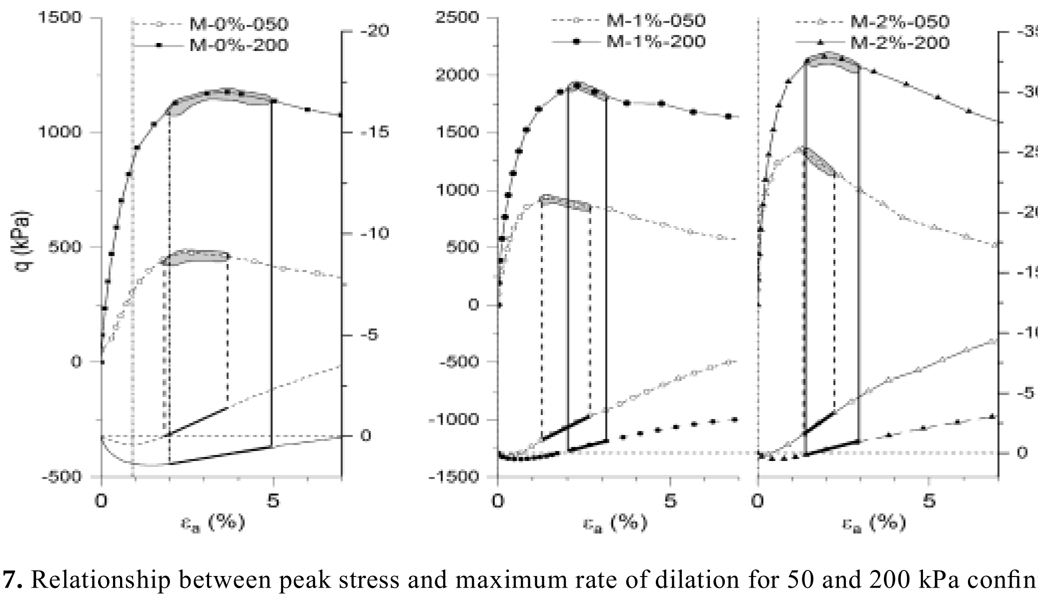

Fig. 7. Relationship between peak stress and maximum rate of dilation for 50 and 200 kPa confining stress: 0% cement on the left; 1% in the middle and 2% on the right.

higherratiosofq/p^{0},indicatingthatthecementationisnow active and allowing a stiff er response from the sample. This diff erence is proportional to the cement percentage; i.e., the highercementpercentage,thehighertheratioq/p^{0}.Theeff ects of cementation are also visible in the location of the peak stress, as it occurs before the maximum rate of dilation; this is similar to what was observed previously in Fig. 7and described by Leroueil and Vaughan (1990). However,thediff erencebetweentheratioq/p^{0},measuredat the peak stress and at the maximum rate of dilation is verysmall,indicatingthatthepeakislargelygovernedby
dilation rather than by cement content. The samples with 1% cement do not show this as clearly as the samples with 2% cement. Another expected behaviour is the reduction in dilation rate with the confining stress, seen in all cement and uncemented samples, similar to the behaviour of cemented sands, demonstrated by Coop and Willson(2003),Consolietal.(2012)andAlvaradoetal.(2012b).
The same authors have pointed out that after the max- imum dilation rate the samples seem to follow a linear fric- tional trend, however, as the stress ratio reduces, the dilationrateseemstoreducemuchquicker,i.e.volumetric

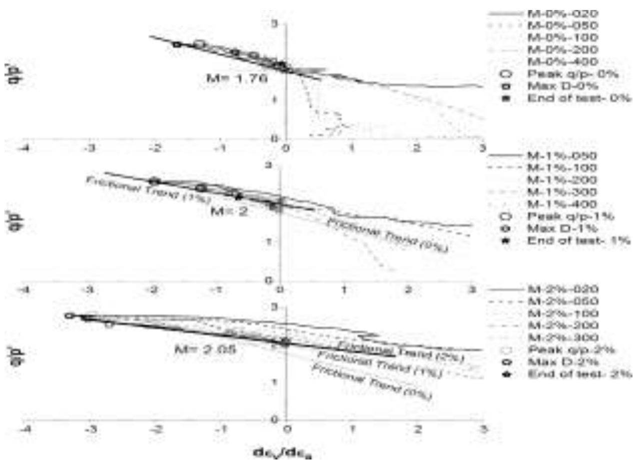

Fig. 8. Tangent stiff ness against axial strain in log scale, together with the gross yield points for: (a) 0%, (b) 1% and (c) 2% cement.

strainschangeatalargerrate,andthepathmovesinwardsand away from the frictional trend. Theauthorsattributethisbehaviourtotheoccurrenceoflocalisationandtherapidreductionofvolumetricstrains.Inthecaseofthesamplestestedhere,asimilarbehaviourwasobservedafterpeak, however, as the shearing continues
thistrendisreversedandthesamplesseemtoconvergetoauniquevalueofM:M=2.00for1%cementandM=2.05for2%cement.AscanbeseeninFig.9band9c,ifthelineartrend line is followed, a higher value of Misdeterminedfor the same percentages of cement, whilst in thecaseofCoopandWillson(2003),alowervaluewouldbedefined.TheworkdonebyMühlhausandVardoulakis(1987)andFinnoetal(1997),showthatthethicknessoftheshearband is proportional to the particle sizedistribution.

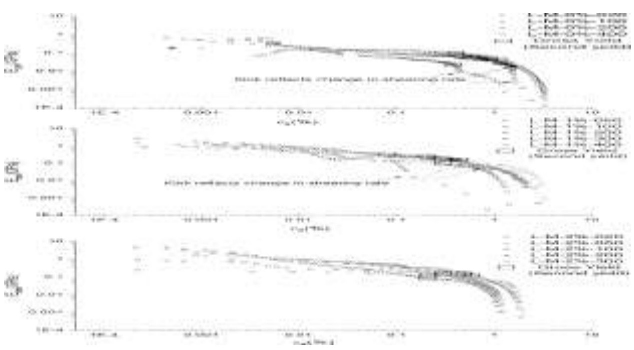

Fig. 9. Stress-dilatancy analysis: (a) 0%, (b) 1% and (c) and 2% cement content.

Authors mention values of 16 and 10 to 25 times d_{50} respectively. Given that the triaxial sample has a finite vol- ume this implies that the volumetric strains measured in a triaxial sample are a function of the grain size distribution. Therefore, variations in the grain size distribution during shear, will cause large changes in the dilation ratio. The samples tested are lightly cemented and the grain size dis- tribution curve obtained after shearing, shows that the final grading has larger intermediate particles than the original grading. This indicates that there is an evolution of the par- ticle size distribution during the shearing process. The DEM work done by Wang and Leung (2008a, 2008b), clearly shows that despite the shearing, there are still clus- ters of particles that remain intact within the sample. The authors, therefore, believe that as bonds degrade due to shearing there is a constant change in the particle size dis- tribution. The consequence are seen as diff erent volumetric strain rates that bring the dilation path inwards. As shear- ing continues, a more stable grading is achieved and a dif- ferentvalueofMisreachedatcriticalstate.

6. Criticalstate

The points correspondent to the peak strength and the end of tests were plotted in Fig. 10, together with the results obtained from the Indirect Tensile tests (ITS) (BSEN13286-42:2003), as well as Unconfined Compressive tests (UCS) (BSEN 13286-41:2003), on samples of the same size, prepared using the same methodology. These samples are not shown on Table 2 but fall within the same

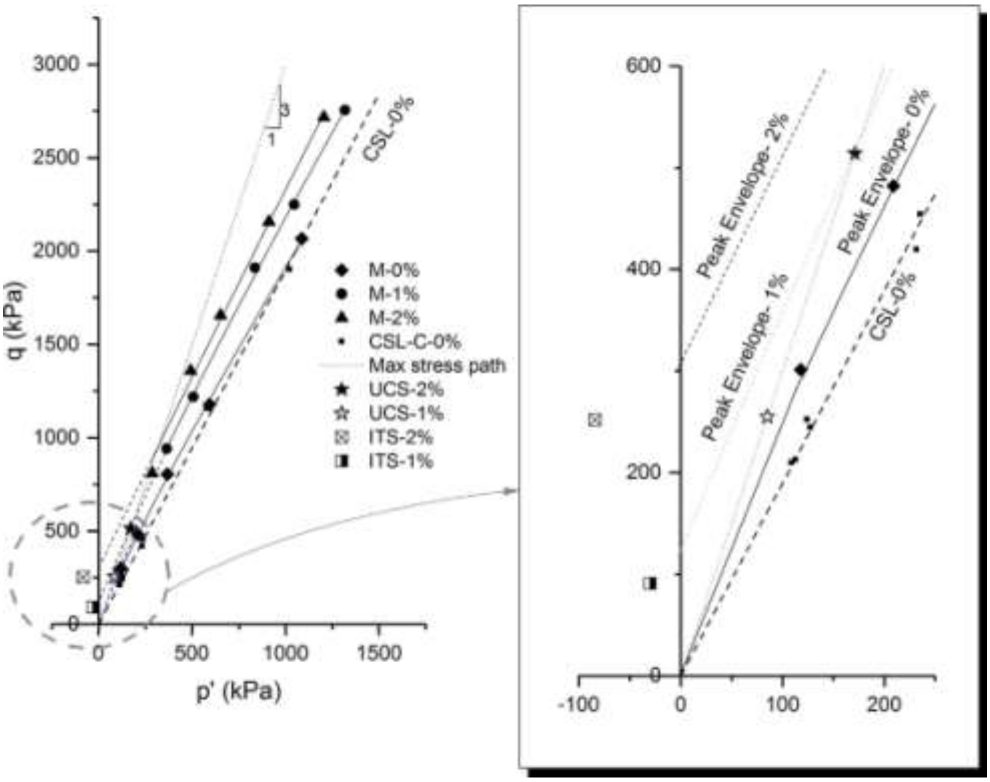

Fig. 10. Peak envelop on q versus p´ diagram; the inset shows the small stresses region.

Table 3

Specification of peak and critical state line.

Type of crushed limestone	Critical state k	C	M	ϕ'_{cs}(°)	Peak q/p'	q Intercept	ϕ'_{p}(°)
Uncemented	0.053	1.747	1.77	43.1	1.81	0	46.6°
1% cement	0.101	2.054	2.00	48.6	1.90	265	45.9°
2% cement	0.122	2.218	2.05	49.8	2.03	299	46.7°

average values. These results served to plot the peak envel- opes, as the cemented samples have a small tensile strength anditmustbeconsideredwhendefiningthepeakenvelopes of the cemented soils. The failure envelopes plottedsuggest values of M that are very similar to the values determined in the dilation plots above. Fig. 10b shows the small stress area with more detail. It is also important to point out that the peak envelope for the uncemented soil is curved and seemstojointheCSLatp'ofaround1100kPa.Theprop- erties of the strength envelopes are shown on Table 3, where the peak friction angle for the uncemented soil was calculated assuming nocohesion.

Been et al. (1991)have shown that within the normal range of engineering stresses, sands show a steady stateline at small stresses that is much shallower than at high stres- ses. Therefore, the paths followed by the tested samples whereplottedonthespecific volume,lnp'spaceonFig. 11, with the final pointof each test indicated by a sym- bol. Although the arrows indicate the direction thetestswere following when they were terminated, Fig. 4shows that the magnitude of this movement was very small for most of the tests. The results show that there is no unique CSL for the cemented and uncemented soils. Instead, the results point clearly to the location of three distinct CSLs, one for each type of soil tested. The results also show that the addition of cement increases the slope of the CSL when compared to the uncemented soil. The larger dilative beha- viour seen in the cemented samples is responsible for the steeper curve gradient shown by the cemented samples. A summary of the parameters obtained for the steady state lines in Fig. 11 is shown in Table 3.

Thesteadystatelinesdeterminedforthecementedsoils seem to reach a common point at stresses of around p'=1000kPa.Thisiscompatiblewiththeresultsshown by the grain size distribution curves, where the PSD of cementedsamplesshearedatlargestressesisverysimilar to the PSD of the uncementedororiginalgrainsizedistri- bution. As the percentages of cement content used inthis

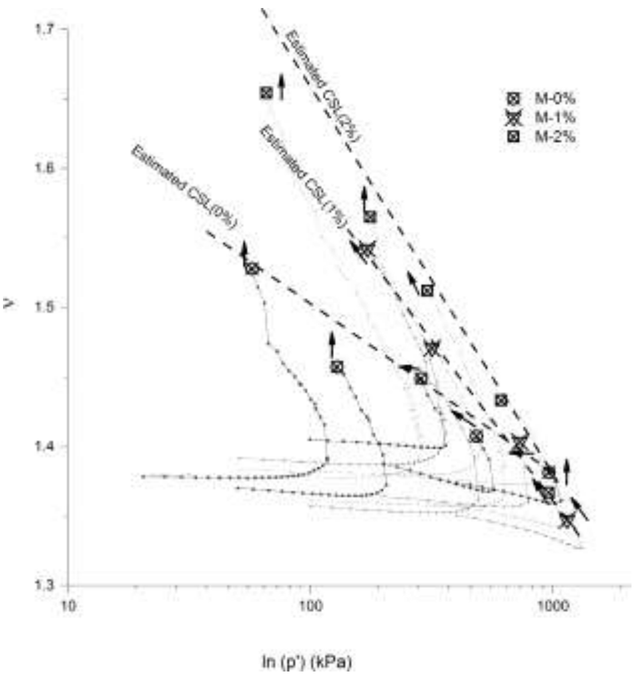

Fig. 11. Location of critical state lines for samples with 0%, 1% and 2% cementcontentonthespecificvolume,m,versusthelogarithmofthemean eff ectivestress,ln(p').

research were very small, the eff ect on the strength of the samples is likely to be felt only at small stresses. At larger confining stresses, the resultant frictional strength mobi- lised is much larger than the contribution of the cement. At that point the changes in the critical state lines caused by the cementation are verysmall.

6.1. Normalisedbehaviour

Given that the stresses used to consolidate the samples were very low and it was not possible to determine a Nor- mal Compression Line (NCL), each set of tests was nor- malised with respect to M and the equivalent pressure on the CSL, by using Eq. (1) (k is the slope of the CSL, C is thespecificvolumeatp'=1kPaandmisthespecificvol- ume, on the CSL and p'_{cs} is the mean eff ective stress on theCSL):

$$p'_{cs} = \exp\left(\frac{\Gamma - \nu}{\lambda}\right) \quad (1)$$

The normalised stress paths show that it is possibleto determine a state boundary surface for the peak states for each set of test up to the critical state line (broken line on Fig. 12). The normalised gross yield points determined from the stiﬀ ness curves for each set of tests was also plot- ted in Fig. 11. The gross yield envelopes are fully enclosed within the respective SBSs indicating that the cement per- centages generated a very weak bonding. At low stresses, the yield surface seems to coincide with the SBS, however, as the stresses increase the yield surface moves inside and awayfromtheSBS.Theeﬀ ectofthecementpercentage

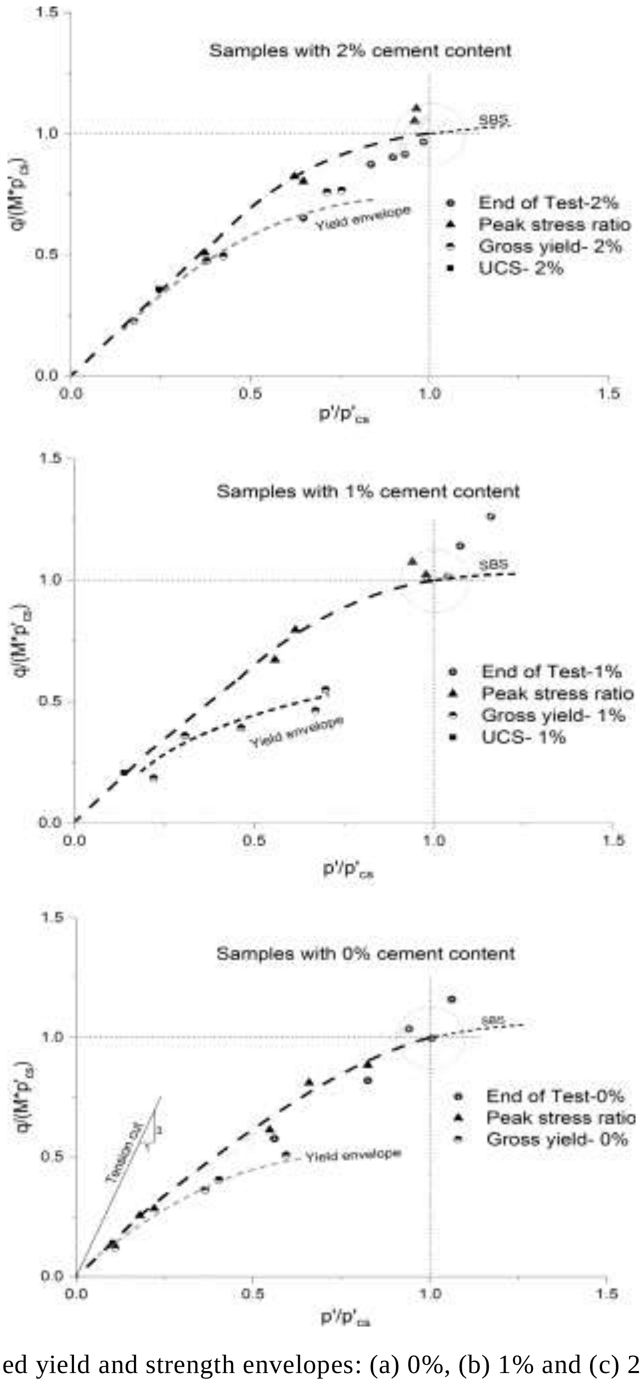

Fig. 12. Normalised yield and strength envelopes: (a) 0%, (b) 1% and (c) 2% cement.

is seen in the proximity between the gross yielding surface and the SBS, as the larger cement content keeps the yield surface closer to the SBS.
In Fig. 13all the state boundaries and yield surfaces wereplottedtogether.Theresultsshowthattheseare very

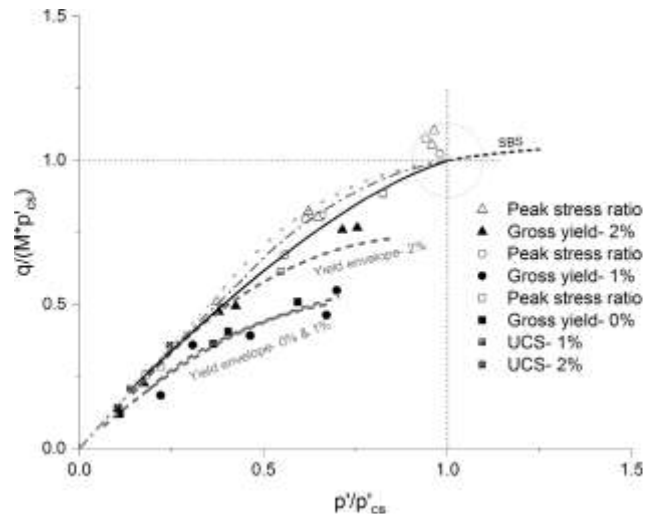

Fig. 13. Comparison of the normalized data for the uncemented and cemented soils.

similar, and a unique SBS could be used to represent the eff ect of the cemented and uncemented tests, when nor- malised by the CSL and the value of M. The gross yield surface of the uncemented and the 1% cement are coinci- dent, however, a unique surface cannot be assumed as the 2% cement results have shown a significantly higher gross yieldsurface.

CONCLUSIONS

This work presents the findings of a study conducted in cemented and uncemented samples of a well graded com- pacted granular material, used for base and sub-base con- struction in the UK. The following conclusions can be drawn from thiswork:

- The mechanical properties of a well graded compacted granular material traditionally used in construction, can be further improved with the addition of small per- centages ofcement.
- The addition of cement also increases the dilative ten- dency of these soils, providing better results particularly when small confining stresses are used as is the case of base and sub-base for road construction.
- The results show that it is possible to determine a unique Critical state line for the uncemented material and that the addition of cement will increase the slope of this line within the range of stresses commonly observed in engi- neering practice.
- When the data is normalised by the equivalent pressure on the CSL and the value of M, it is possible to deter- mine a unique state boundary surface for the cemented and uncemented soils used in this research. The gross yield surface, however, is not unique and will depend on the cementpercentage.

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