Small to large strain mechanical behaviour of an alluvium stabilised with low carbon secondary minerals

P. Sargent* [1], M. Rouainia [2] A. Diambra [3], D. Nash [4] and P. N. Hughes [5]

* Corresponding author

1 Stephenson Building, School of Science, Engineering and Design, Stephenson Street, Teesside University, Middlesbrough, Tees Valley, TS1 3BA, UK.

2 Drummond Building, School of Engineering, Newcastle University, Newcastle upon Tyne, Tyne and Wear, NE1 7RU, UK.

3 Queens Building, Department of Civil Engineering, University of Bristol, University Walk, Clifton, Bristol, BS8 1TR, UK.

4 Posthumous. Previous affiliation is Queens Building, Department of Civil Engineering, University of Bristol, University Walk, Clifton, Bristol, BS8 1TR, UK.

5 Christopherson Building, Department of Engineering, Durham University, Lower Mountjoy, South Road, Durham, DH1 3LE, UK.

Paul Sargent MSci MSc PhD CGeol FGS

E-mail: p.sargent@tees.ac.uk

Tel: +44(0)7815 714237

Mohamed Rouainia Dipl. -Ing MSc PhD

E-mail: m.rouainia@newcastle.ac.uk

Tel: +44(0)191 208 3608

Andrea Diambra MEng PhD

E-mail: andrea.diambra@bristol.ac.uk

Tel: +44(0)117 331 5600

David Nash MA MSc DIC PhD MICE CEng

E-mail: please forward correspondence to andrea.diambra@bristol.ac.uk

Paul N. Hughes   BSc MSc PhD

E-mail: paul.hughes2@durham.ac.uk

Tel: +44(0)191 334 2450

Abstract

Deep dry soil mixing is a popular ground improvement technique used to strengthen soft compressible soils, with Portland cement being the most popular binder. However, its continued use is becoming less sustainable given the high CO_2 emissions associated with its manufacture. Alkali-activated cements are considered to be viable low carbon alternative binders, which use industrial waste products such as blast furnace slag. This study focusses on the stabilisation of a potentially liquefiable soft alluvial soil using a dry granulated binder comprising sodium hydroxide-activated blast furnace slag (GGBS-NaOH). This binder has previously been demonstrated by the authors to have potential as a replacement for Portland cement due to its excellent engineering performance, positive contributions towards the circular economy, reducing energy usage and CO_2 emissions in the construction sector. A detailed comparison in mechanical behaviour is presented between the soil in its reconstituted, undisturbed and cemented states after 28 days curing through the use of advanced monotonic triaxial testing techniques, including small strain measurements. Mechanical behaviour was specifically analysed regarding peak deviatoric strength, pore pressure response, stress – volumetric dilatancy, shear stiffness degradation over small to large strain ranges, critical state and failure surfaces. Using 7.5% GGBS-NaOH increased the stiffness and shear strength of the soil significantly, whereby the shear strains at which initial shear stiffness degrades is three times higher than the untreated undisturbed soil. As a result, larger amounts of dilation was observed during shearing of the material and resulted in an upward shift of the soil's original critical state line due to the creation of an artificially cemented soil matrix through the precipitation of C-(N)-A-S-H gels.

Keywords

Mechanical behaviour, low carbon, GGBS, alluvium, stiffness degradation, triaxial.

Highlights

- Using GGBS in geotechnics contributes towards lowering global CO_2 emissions.
- GGBS-NaOH stabilisation of a soft soil enhanced strength over various strain levels.
- Stabilisation successfully delayed the onset of stiffness degradation.
- The new cemented structure significantly improved the soil frictional behaviour.

What is already known in this area?

Extensive research has been undertaken focussing on basic mechanical performance (e.g. UCS) of soils stabilised with CEM-I binders, which have only been partially replaced by GGBS/PFA.

What does this study add to the literature?

Small strain mechanical behaviour of soils stabilised with an alkali-activated GGBS binder, which can completely replace CEM-I. This has sustainability benefits in terms of lowering CO_2 emissions.

1.0 Introduction

Alluvial soils are in great abundance in river flood plains, which are problematic in construction due to their low bearing capacities and high compressibilities. Deep dry soil mixing (DDSM) has become an increasingly popular technique for improving such ground conditions. DDSM involves injecting a cementitious binder into the ground via a rotating auger drill; thereby producing soil-cement columns or panels. Current trends in the European market also involve using DDSM as an alternative technique for creating deep pile foundations. DDSM is very versatile in that it may be used to treat a wide variety of soil types [1], is economical and produces considerably less waste and ground vibration compared with other ground improvement and piling techniques. Ordinary Portland cement (CEM-I) and lime have been traditionally used as the cementitious binders in DDSM since this technique was developed in Scandinavia during the 1960's, due to their favourable strengthening properties.

The mechanism through which strength improvements are generally achieved within cement-treated soils is via an increase in pH conditions and the hydration of calcium silicates / aluminates within the binder to form cementitious gels, producing a cemented soil matrix. Whilst the majority of strength development occurs during the first month of curing through hydration, strength continues to increase slowly with time through pozzolanic reactions if soil pH ≥10.5 [2]. To ensure DDSM is effective in enhancing a soil's engineering performance, it's physico-chemical properties such as particle size distribution, plasticity, pH, moisture content, cation exchange capacity (CEC), specific surface area, organic and sulphate contents must be characterised prior to selecting the most appropriate binder. Generally, soils suitable for DDSM treatment are characterised by low organic contents (<1%), low sulphate contents (<0.3%) and clay contents of 10–50% [3].

The continued use of CEM-I and lime in today's society is becoming less economically and environmentally sustainable. Cement manufacture is highly energy intensive, requiring 5000MJ per tonne of CEM-I [4] and contributes up to 7% of the world's CO_2 emissions [5]. To address this issue, efforts have been made to introduce industrial waste products (IWPs) as partial replacements for CEM-I, including ground granulated blast-furnace slag (GGBS), pulverised fly ash (PFA), red gypsum (RG) and rice husk ash (RHA). Alkali-activated cements (AACs) are considered to be popular and viable low carbon and economical alternatives to CEM-I and lime. These materials involve the sole use of pozzolanic alumino-silicate based IWPs in combination with alkalis (e.g. sodium hydroxide, NaOH) for raising soil pH to promote pozzolanic conditions and activate the hydraulicity of the IWP.

There is extensive literature demonstrating that AACs are capable of producing engineering performances that are either comparable to or exceed those of CEM-I and lime. Cristelo et al. [6] undertook laboratory and field studies using sodium silicate (Na_2SiO_3)-NaOH activated type F PFA for stabilising a low plasticity sandy clay. The binder produced higher strengths more rapidly compared with CEM-I stabilised samples. Sargent et al. [7] assessed the UCS, compressibility, durability and pH performance of NaOH pellet–Na_2SiO_3 solution activated GGBS, PFA and RG binders at a dosage of 10% by dry weight to stabilise an artificial low plasticity alluvial silty sand. Results indicated that alkali-activated GGBS stabilised mixtures produced the best engineering performances; whereby 28 day UCS of 6MPa was achieved in comparison to 3MPa achieved by CEM-I stabilised samples. Whilst the activator used promoted pozzolanic conditions, there are practicality issues associated with using Na_2SiO_3 solution in a binder for DDSM. Habert et al. [8] determined that Na_2SiO_3 production is also more expensive and has a higher environmental impact than NaOH. Bernal [9] determined that Na_2SiO_3 has a higher accelerated carbonation depth over NaOH. These findings informed Sargent et al.'s [10] study for treating a soft alluvium from Northumberland (UK), whereby the binder used was GGBS-NaOH, with a GGBS-NaOH ratio of 2:1 at dosages of 0 – 10% by dry weight. Whilst a 10% dosage produced the best engineering performances, using a 7.5% dosage was sufficient for achieving EuroSoilStab (2002) [12] 28 day undrained shear strength requirement of 150kPa. The 7.5% dosage also produced strengths that were superior than using 10% CEM-I and was deemed more economically and environmentally sustainable.

With the progressive closure of coal fired power stations and extensive use of PFA in concretes and grouts over the past few decades as a partial replacement for CEM-I, PFA supplies are rapidly disappearing – especially in the UK. Hence, the future development of AACs needs to utilise IWP waste streams which have longevity in supply. Whilst the UK steel manufacturing industry is declining, other global economies (e.g. China, USA) continue to grow – particularly in the construction sector. Hence, the demand for steel continues to rise meaning that slag waste will continue to be produced worldwide for the foreseeable future. Slags produced from steel manufacture represent approximately 15% by mass of the steel produced [12].

This highlights the continued need to recycle slag to make positive contributions towards the circular economy and reducing global CO_2 emissions.

Other recent studies that have investigated alternative waste streams for developing AACs in soil stabilisation include NaOH-activated volcanic ash (VA) to stabilise a low plasticity clay [4]. Whilst VA is naturally pozzolanic, represents a vast worldwide resource and when NaOH-activated produces strengths 200% higher than CEM-I stabilised samples [4], there are potential environmental implications. These include the need to quarry out ash deposits and that VA can contain elevated concentrations of Cl, S and F [14], all of which are water-soluble. Depending on the geological setting in which VA is produced, it may be highly acidic or alkaline and have knock-on effects on the pH of surface waters. Collectively, these could have negative impacts on soil fertility, groundwater resources and associated ecosystems if VA cements are used in DDSM.

Whilst there is extensive literature covering laboratory investigations into the performance of IWP, CEM-I and AAC concretes that focus on unconfined compressive strength, there is relatively little material which investigates their mechanical behaviour in the context of geotechnical materials over small to large strain ranges using triaxial equipment. Ahnberg [15] undertook triaxial tests on two Swedish clays stabilised with lime, CEM-I and GGBS at a dosage of 100kg/m^3. Whilst Ahnberg observed variations in shear strengths due to the soil type, binder design and curing period, stress-strain behaviour was noted to be similar due to the degree of overconsolidation. Rios et al. [16] investigated the shearing behaviour of a CEM-I stabilised silty sand, derived from weathered Porto granite. Triaxial testing was undertaken over a range of confining pressures (30 – 20,000kPa), whereby samples were prepared with controlled binder dosages of 2 – 7%. In both effective deviatoric and volumetric stress spaces, Rios et al. [16] identified a possible critical state line (CSL) for the cemented soil at large strains – independent of the binder dosage or porosity/cement index. Additionally, Rios et al. [16] concluded that normal compression lines (NCL), CSL and state boundary surface for the cemented soil may be independent of binder dosage but may depend only on binder dosage in terms of porosity/cement ratio.

Knowledge of how soft soils and cement stabilised soils behave at small strain levels is of key interest in geotechnical design when considering soil–structure interactions for foundations and retaining walls. Presented in this paper are results from a suite of drained and undrained isotropically consolidated triaxial compression tests undertaken on an alluvium in its natural undisturbed state, reconstituted state and when treated with a GGBS-NaOH binder after 28 days curing. Tests were undertaken over a range of effective stress conditions to understand the improvement in mechanical behaviour achieved when using an optimum binder dosage. Testing data is presented in terms of small strain shear stiffness degradation, critical state and stress dilatancy.

2.0 Materials and Testing Methodologies

2.1 Soil and Binder Materials

2.1.1 Lanton Alluvium

The alluvial soil considered in this study was sourced from the River Glen flood plain in Lanton, approximately 4km North West of Wooler in Northumberland, UK. Disturbed and undisturbed thin-walled U100 samples were taken from the depth range of 1.5–2.4m. The local superficial geology is characterised by Holocene alluvium deposits along the course of the River Glen, bounded by river terrace sand and gravel deposits, Devensian glacial till and fluvioglacial deposits. The index properties of the soil were determined by Sargent et al. [10], which are summarised in Table 1. A soil grading curve and compaction curves for the Lanton alluvium soil are provided in Figures 1 and 2 respectively. The particle size distribution and compaction curves for the soil were obtained using methods in accordance with BS1377 [11].

Table 1: Summary of the Lanton alluvium's index properties. Sourced from Sargent et al. (2016).

Property	Unit	Value
In-situ moisture content	%	25
Plasticity Index	%	14.95
Liquid Limit	%	35.66
Saturated unit weight	kN/m^3	18.44
Bulk Density	Mg/m^3	2.0
Dry density	Mg/m^3	1.74
Cation exchange capacity	cmol/kg	11.45
Specific surface area	m^2/g	6.45
Total organic content	%	0.76
Sulphate content	mg/kg soil	49
BS 5930 (BSI, 1990) classification	-	Silty SAND

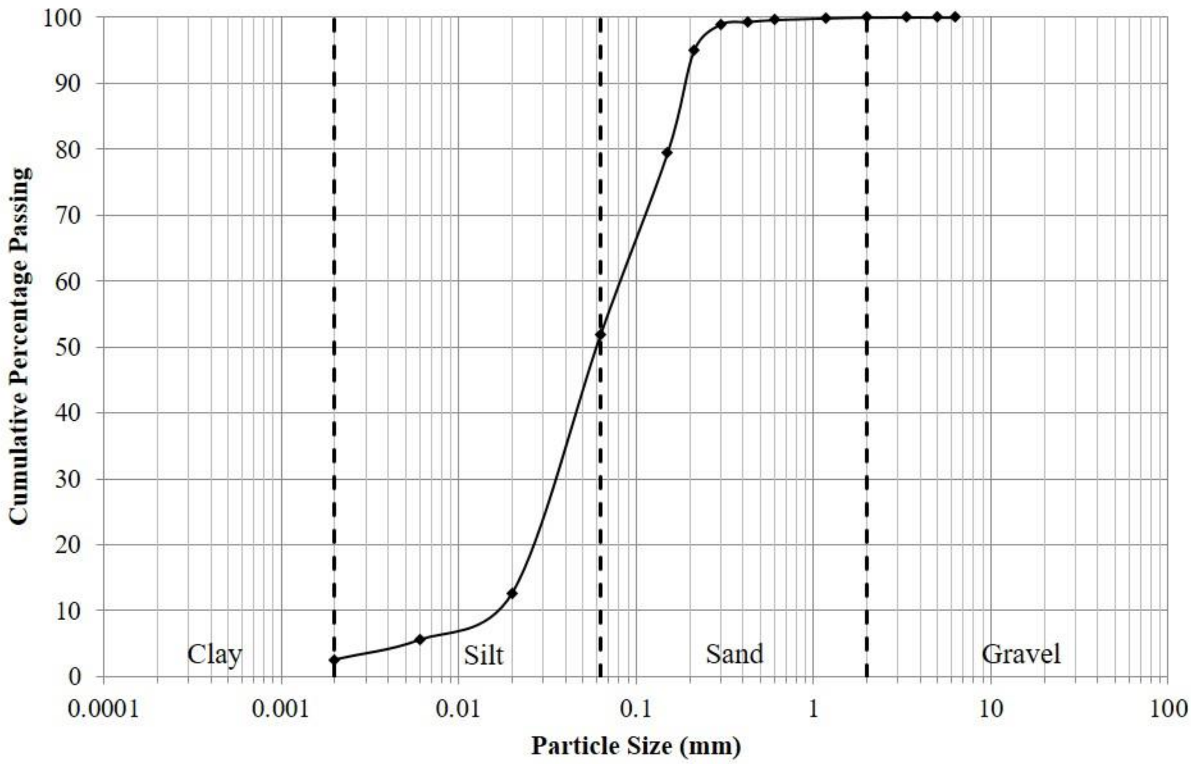


Figure 1: Soil grading curve for Lanton alluvium.

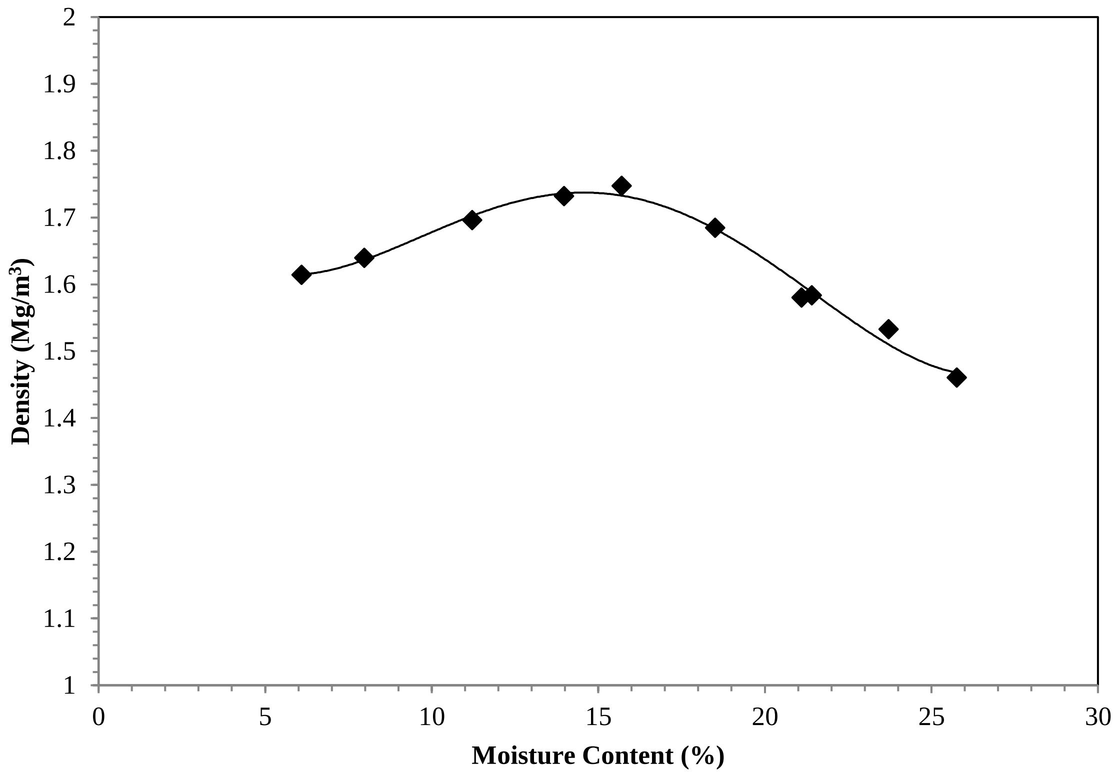


Figure 2: Dry density compaction curve for Lanton alluvium.

2.1.2 Cementitious Binder

The dry binder and dosage used was based on the results from Sargent et al. [10]. The IWP used was GGBS as supplied by Hanson Cements Ltd., which was mixed with NaOH in dry pellet form (supplied by Fisher

Scientific UK Ltd) as an alkali activator. The molarity of the NaOH was 39.997g/mol. Sargent et al. [10] revealed that an optimum dosage of 7.5% by dry weight is appropriate for stabilising Lanton alluvium, with a view to achieving engineering performances which are comparable or exceed those of Lanton alluvium stabilised with lime or CEM-I. Additionally, the 7.5% dosage of GGBS-NaOH ensured that the stabilised alluvium met the minimum 28 day undrained shear strength requirement of 150kPa defined by EuroSoilStab [12]. Hence, this is the binder design selected for this study.

2.2 Testing Methodologies

To gain a comprehensive understanding of the short and long term mechanical behaviour of reconstituted, undisturbed and GGBS-NaOH treated Lanton alluvium, particularly in defining their respective critical state lines, yield surfaces and effective shear strength properties; monotonic consolidated drained and undrained triaxial tests were undertaken under a minimum of four effective confining stress conditions. Bender elements and local instrumentation were employed to characterise the initial shear stiffness of the material over a range of stress level and its degradation during shearing.

2.2.1 Sample Preparation

2.2.1.1 Reconstituted samples

Cylindrical samples 100mm in diameter and 200mm long were prepared by initially drying the disturbed soil in an oven at 110^{o}C for 24 hours. Once dry, the soil was milled into a fine powder (particle size ≤1mm) for ease of sample mixing and then mixed with water in a Hobart rotary mixer for 10 minutes to achieve a gravitational moisture content of 25% (per the soil's in-situ moisture content) and homogeneity. Once mixed, the soil was compacted into three layers within a standard 100mm diameter compaction mould in accordance with BS 1377 [11]. All samples were prepared with a bulk density of 1.75Mg/m^3, based on optimum compaction criteria. Once compacted, the sample was extruded and trimmed in preparation for triaxial testing according to BS1377 [11]. Prior to placing on the triaxial base pedestal and encapsulation within a latex membrane, sample masses and dimensions were measured. Due to the soil's high silt content, low cohesion, relatively high moisture content and absence of natural internal structure, reconstituted samples displayed some slumping upon extrusion.

2.2.1.2 Undisturbed Thin-walled (UT) samples

100mm diameter UT100 sized samples were obtained from the field, wax sealed and wrapped in bubble wrap to prevent loss of moisture and minimise sample disturbance during transport to the laboratory. Prior to mounting on the triaxial base pedestal for testing, samples were extruded and trimmed to the appropriate dimensions, weighed on a mass balance and dimensions measured. Undisturbed samples exhibited little slumping upon extrusion compared with reconstituted samples, thereby providing an indication of the soil's initial sedimentation structure.

2.2.1.3 Alkali activated cemented samples

For practicality purposes in the laboratory, maximising sample homogeneity and the number of reactive sites for cementitious bond formation, samples were prepared in accordance with the methodology adopted by Sargent et al. [10] by first mixing the oven dried soil powder with the GGBS-NaOH binder at a dosage of 7.5% by dry weight in a rotary mixer for 10 minutes. The GGBS – NaOH ratio used was one part NaOH to two parts GGBS [10]. Once the required quantities of soil and binder had been mixed, water was incrementally added to the mixture to achieve the pre-treatment (in-situ) soil optimum moisture content of 25%. Samples were tamped and compressed into a split sample mould, inserted into a hydraulic press to form samples of various sizes (200mm long – 100mm diameter; 100mm long – 50mm diameter; 76mm long – 38mm diameter) and achieve a target density of 1.9Mg/m^3 based on optimum compaction criterion. Samples were then cured within wax-sealed PVC tubes for 28 days [17] and stored within a temperature-controlled room with a relative humidity of 55% and ambient air temperature of 20^{o}C [7] [10]. Once cured and extruded, sample ends were carefully trimmed using a surface grinder within a tolerance of 25µm according to ASTM D4543-08 [18]. This ensured sample ends were smooth, parallel with each other and reduced bedding errors during testing.

2.2.2 Apparatus and Testing procedures

Triplicate samples of each material were tested for each effective confining stress to remove bias, identify anomalies and maximise data reliability. Table 2 summarises the triaxial testing programme undertaken. A total number of 53 triaxial tests were performed.

Table 2: Experimental programme for triaxial testing

Sample ID Convention	Sample type	Drainage condition	GGBS-NaOH content (%)	Effective mean confining stress, p'$_0$ (kPa)
ReconCU X [1]_Y [2]	Reconstituted	Undrained	0	50, 100, 150, 250
ReconCD X_Y	Reconstituted	Drained	0	50, 150, 200
UndisCU X_Y	Undisturbed	Undrained	0	50, 150, 250
UndisCD X_Y	Undisturbed	Drained	0	50, 150, 200
CemCU X_Y	28 day Cemented	Undrained	7.5	50, 100, 200, 300, 400, 600
CemCD X_Y	28 day Cemented	Drained	7.5	50, 100, 200, 400, 600

Notes: [1] – Value for 'X' denotes the p'$_0$ value used for the test. [2] – Value for 'Y' denotes the test sample number.

2.2.2.1 Apparatus

The equipment used comprised a 2MPa capacity perspex cell mounted on an electro-mechanical advanced digital triaxial system. A 64kN capacity submersible load cell was mounted on the end of the loading ram within the triaxial cell. Cell pressures were controlled and regulated via a 3MPa digital pressure controller, whereas a 1MPa digital pressure controller was used to supply and control back pressures and monitoring sample volume changes. To accurately measure the cell pressure, back pressure and pore pressures at the top and base of samples during tests, calibrated 15 bar capacity pressure transducers were fitted to the base of the triaxial cell. An external LVDT was mounted on the exterior of the triaxial cell to measure external ram displacements. For measuring local axial strains (ε_a) on samples, two LVDT local strain gauges were mounted diametrically opposite each other to the middle third section of samples which is considerably less restrained compared with the sample ends. A further LVDT local strain gauge was used to measure radial strains at mid-sample height, which was mounted via a radial caliper. To accurately measure changing shear stiffnesses during shearing over the small to large strain range, bender elements were installed within the sample top cap and triaxial frame base pedestal.

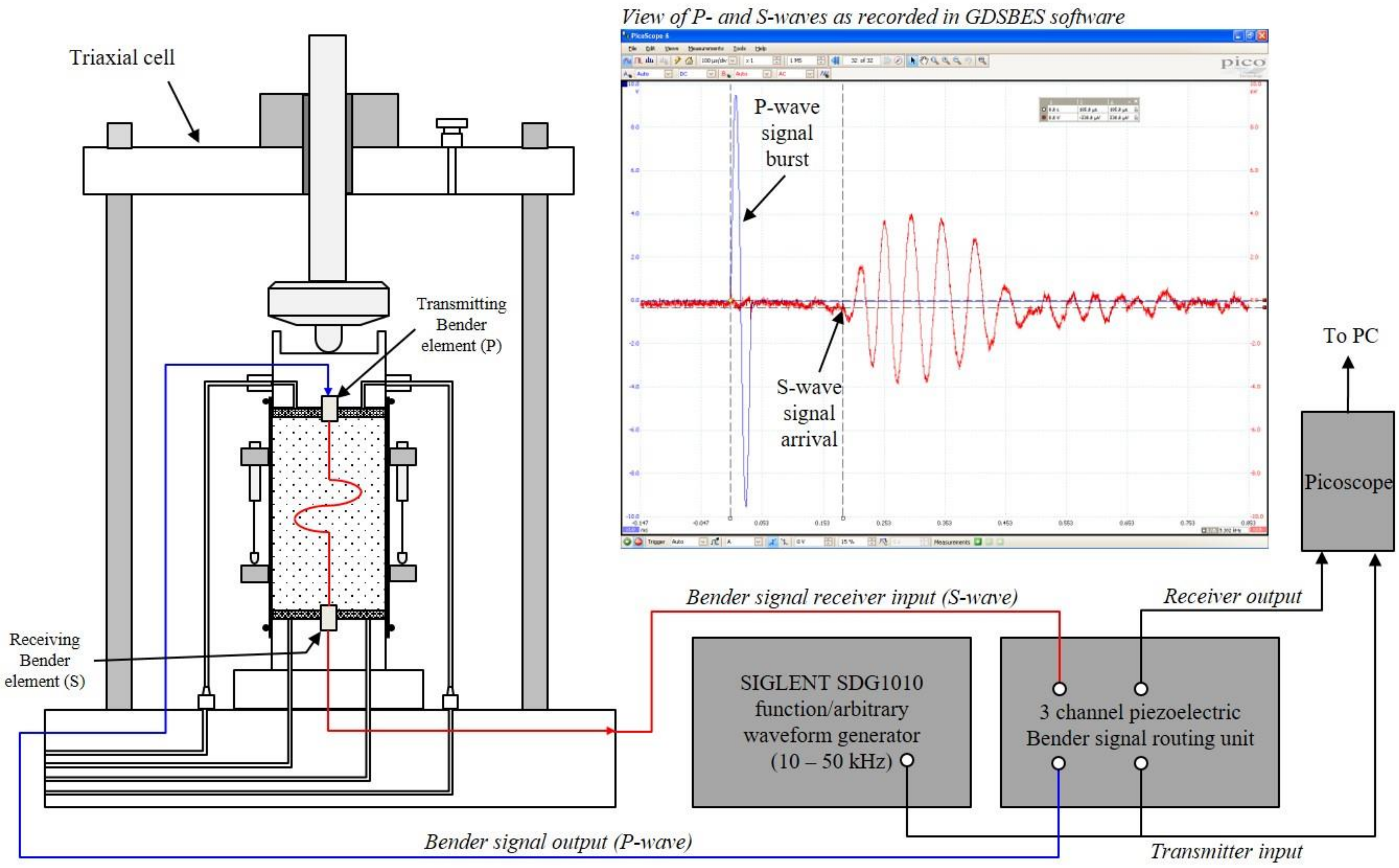
2.2.2.2 Triaxial Testing Procedures

When mounting untreated soil samples within the triaxial cell for testing, bender elements installed within the top cap and base pedestal protruded by 10mm, which ensured they were of sufficient length to penetrate through the porous discs and then pressed by hand into the sample ends by 3–5 mm – ensuring a good contact. However, for cemented samples the bender elements could not simply be pushed in by hand. Instead, 5mm deep slots were formed in the sample ends by using a Stanley knife. To ensure good coupling between the cemented samples and the bender elements, any gaps between the slot walls and the bender elements were infilled by using a filler made from the Lanton alluvium – GGBS-NaOH mixture. The top cap was then directly connected to the submersible load cell to reduce bedding errors. After filling the triaxial cell with de-aired water, samples were saturated in general accordance with BS1377 [11]. Cell and pore pressures were incrementally increased by 50-100kPa per day until a minimum back/pore pressure of 350kPa was achieved to ensure dissolution of air bubbles within the back pressure system. An effective confining stress of 5-10kPa was maintained throughout saturation. Samples were considered saturated once a minimum Skempton's B value of 0.95 was recorded. Following saturation, samples were isotropically consolidated until the required effective confining stress (p'_0) had been reached and the volume change stabilised. The p'_0 conditions used for tests are summarised in Table 2. Throughout saturation and consolidation stages, the submersible load cell maintained a constant deviatoric stress (q) on the sample of 0kPa to reduce bedding errors and enabled the constant measurement of changing sample height throughout tests. After consolidation, samples were compressed at constant ε_a rates of 0.01mm/minute and 0.05mm/minute for undrained and drained tests respectively, based on BS1377 [11] calculations. The failure criteria selected for this work were peak effective stress ratio and ultimate state (i.e. reaching ε_a of 15%) to capture the residual (i.e. critical state) mechanical behaviour of samples.

At the end of saturation, consolidation and during compression stages of each test, bender element measurements were taken to determine degradations in sample shear stiffnesses. An S-wave was transmitted from the bender element housed within the sample top cap, through samples and received by the bender element within the base pedestal. A minimum of three wavelengths were required to pass through samples during bender element tests. The frequency used for the source bender element's signal bursts was based on prior knowledge of the stiffness of the Lanton alluvium in its untreated and cemented states, in addition to a sensitivity analysis whereby frequency was varied between 1 and 50kHz to identify which frequency produced the highest quality S-wave signal. It was determined that S-wave signals produced for bender element measurements using lower frequencies were characterised by higher degrees of noise, due to the influence of the near field effect [19]. The optimum frequency for bender element measurements taken for all sample types was found to be 20kHz. The time domain method for determining S-wave arrival times from bender element tests was adopted, which assumes no reflected/refracted waves are detected. The arrival of the received signal tends to be characterised by an initial downward deflection relative to the travel time (x) axis, which arises from the "near field effect". This effect is caused by wave front spreading and coupling between waves that are characterised by similar particle motions but propagating at different velocities [19] [20]. The GDS BEAT tool was predominantly used as a convenient method for analysing the bender element data sets, by using the time and frequency domain techniques. Based on the experience of the authors in the use and interpretation of bender elements, the first bump maximum was considered the most suitable point on the received signal as the first S-wave arrival. Figures 3 and 4 show the experimental apparatus used for taking bender element measurements and some representative S-wave signals observed for the untreated and GGBS-NaOH treated Lanton alluvium, respectively.

305

306



307 Figure 3: Bender element apparatus used within triaxial apparatus.

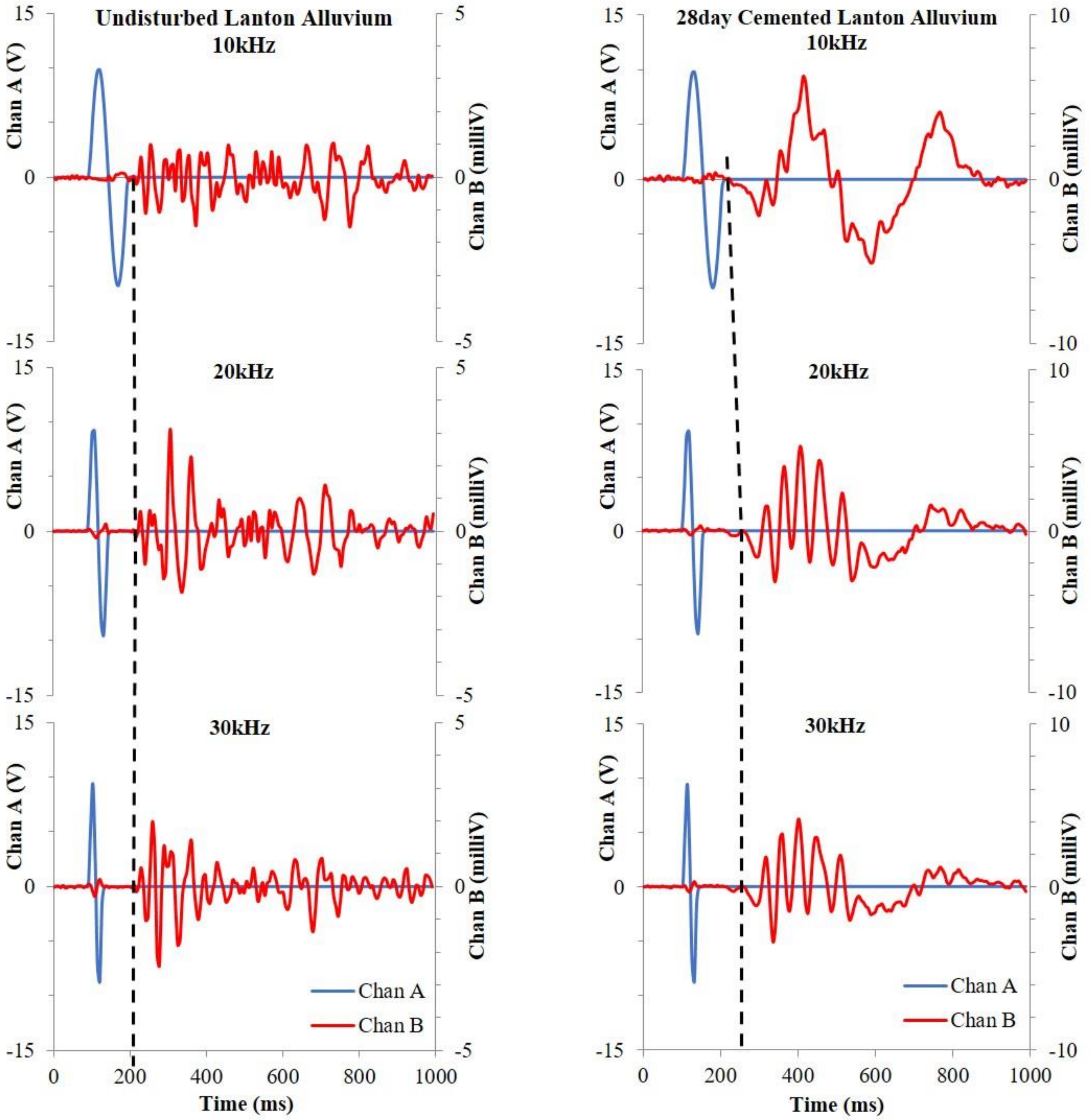


Figure 4: Source and received signals recorded for bender element measurements taken at frequencies of 10-30kHz during triaxial tests on samples of undisturbed Lanton alluvium (left) and 28 day GGBS-NaOH stabilised Lanton alluvium (right).

3.0 Results

3.1 General stress-strain behaviour

3.1.1 Reconstituted Lanton alluvium

The mechanical response under consolidated undrained triaxial shearing of the reconstituted samples are displayed in Figure 5 in terms of q-ε_a response (Figure 5a), excess pore pressure (U) - deviatoric strain (ε_q)

response (Figure 5b) and q versus mean isotropic stress (Figure 5c). The undrained q-ε_a behaviour of the reconstituted alluvium all exhibited work hardening up to peak strength at deviatoric strains of 5–6%. Irrespective of the confining stress level, ranging from 50–250kPa, U shows a pronounced tendency to contraction with increases in the applied confining stress. For p'$_0$=50kPa, pressures increased up to 20–40 kPa at ε_q of 1%; where pressures equalised with further straining until failure. In contrast, U values within undrained samples tested at p'$_0$=100–250kPa reached peak values of 50–90kPa at 2% ε_q. With increasing ε_a, a gradual reduction in U of 30–50kPa was then observed until failure.

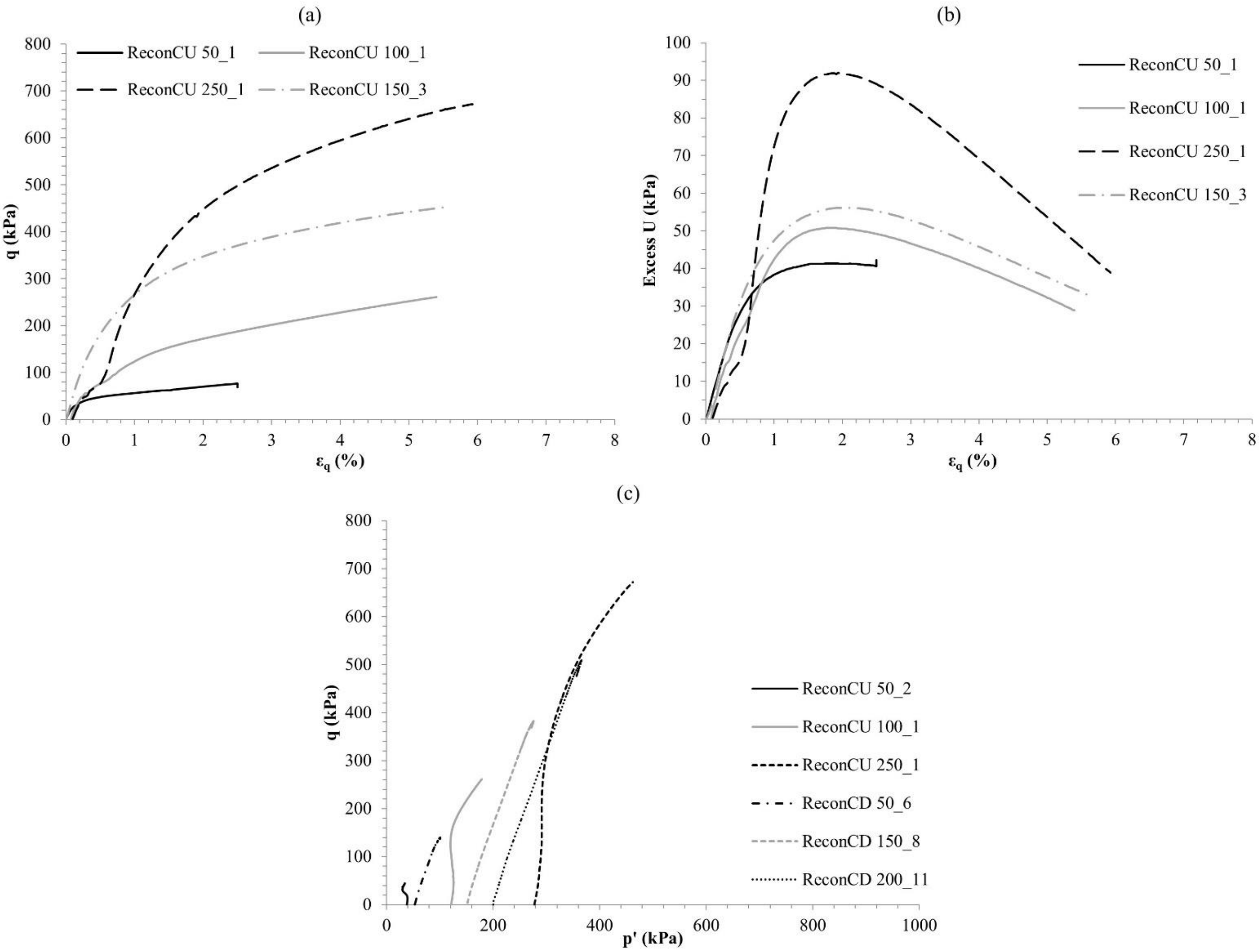


Figure 5: (a) Undrained deviatoric stress – strain response; (b) excess pore pressure – deviatoric strain response and (c) drained and undrained effective stress paths in the deviatoric stress – mean isotropic stress plane for reconstituted Lanton alluvium during shear tests.

The q and volumetric response of reconstituted samples under drained conditions with increasing ε_q is presented in Figure 6. Most samples sheared under initial effective stresses of 50 and 150kPa displayed work hardening between small and large strains up to failure. However, at p'$_0$ values of 200–250kPa samples exhibited some limited evidence of strain softening at larger strains (>10% for drained tests). Along with the contractional behaviour typical of normally consolidated samples, the volumetric response shows an overall compressive behaviour as observed in Figure 6b. Compressive behaviour increased with confining cell pressure and very limited dilation of 0.2% at large strains only.

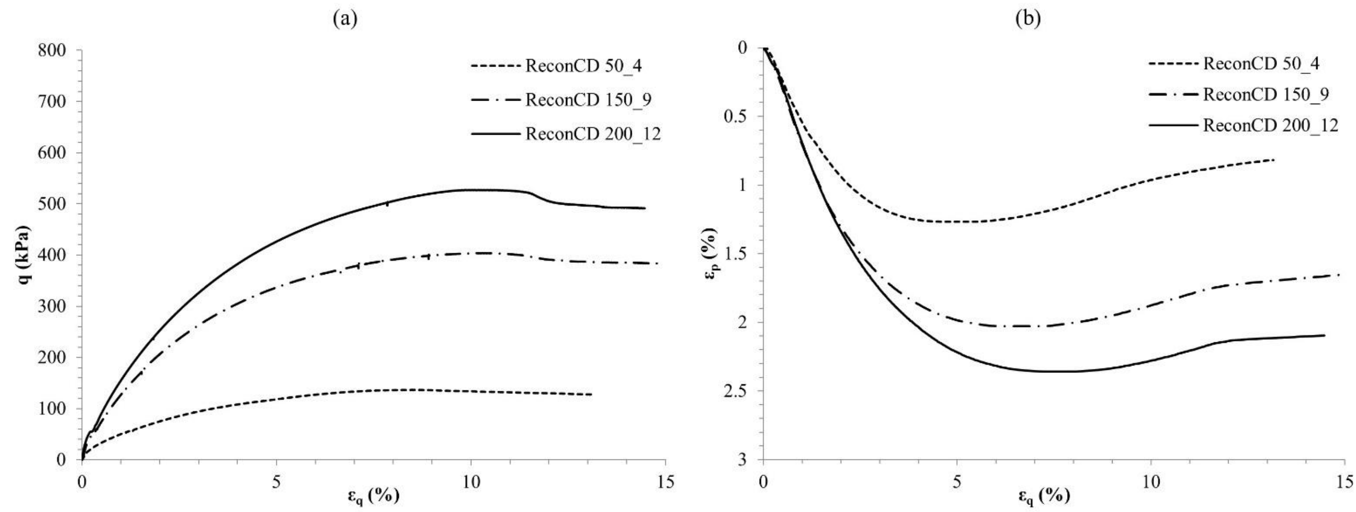


Figure 6: (a) Drained deviatoric stress – shear strain and (b) dilational behaviour of reconstituted Lanton alluvium during shear tests.

For drained tests undertaken at $p'_0 = 50$, 150 and 200kPa, peak dilation occurred at ε_q values corresponding to peak strength. Upon reaching ε_q of 12–15% where most samples reached their final q value, dilation became suppressed, suggesting that most samples had reached their critical state. Volumetric trends for the samples tested are seen tending towards zero volumetric strain (ε_p). Strain localisation or shear plane development were generally not observed for reconstituted samples.

Using the slope (M*) from the dilation segments of the ε_p–ε_q curves in Figure 6b, dilation angles (ψ) values for the reconstituted material generally ranged between 1.37–1.81^o. Higher ψ values of up to 2.3^o were recorded in some samples, which may be attributable to slight density variations between samples.

3.1.2 Undisturbed Lanton alluvium

The typical mechanical responses for undisturbed Lanton alluvium samples are reported in Figure 7 in terms of undrained q versus ε_q (Figure 7a), U versus ε_q (Figure 7b) and q versus mean isotropic stress (p') (Figure 7c). The undrained q-ε_q behaviour of undisturbed samples consolidated to p'_0=50 and 150kPa displayed work hardening up to failure. Whereas samples consolidated to p'_0=250kPa experienced strain softening at strains >7%. In general, it can be seen from Figures 5a and 7a that the peak q achieved for reconstituted samples were markedly higher than those for undisturbed samples. This apparent higher strength is likely due to a degree of over-compaction during preparation of reconstituted samples.

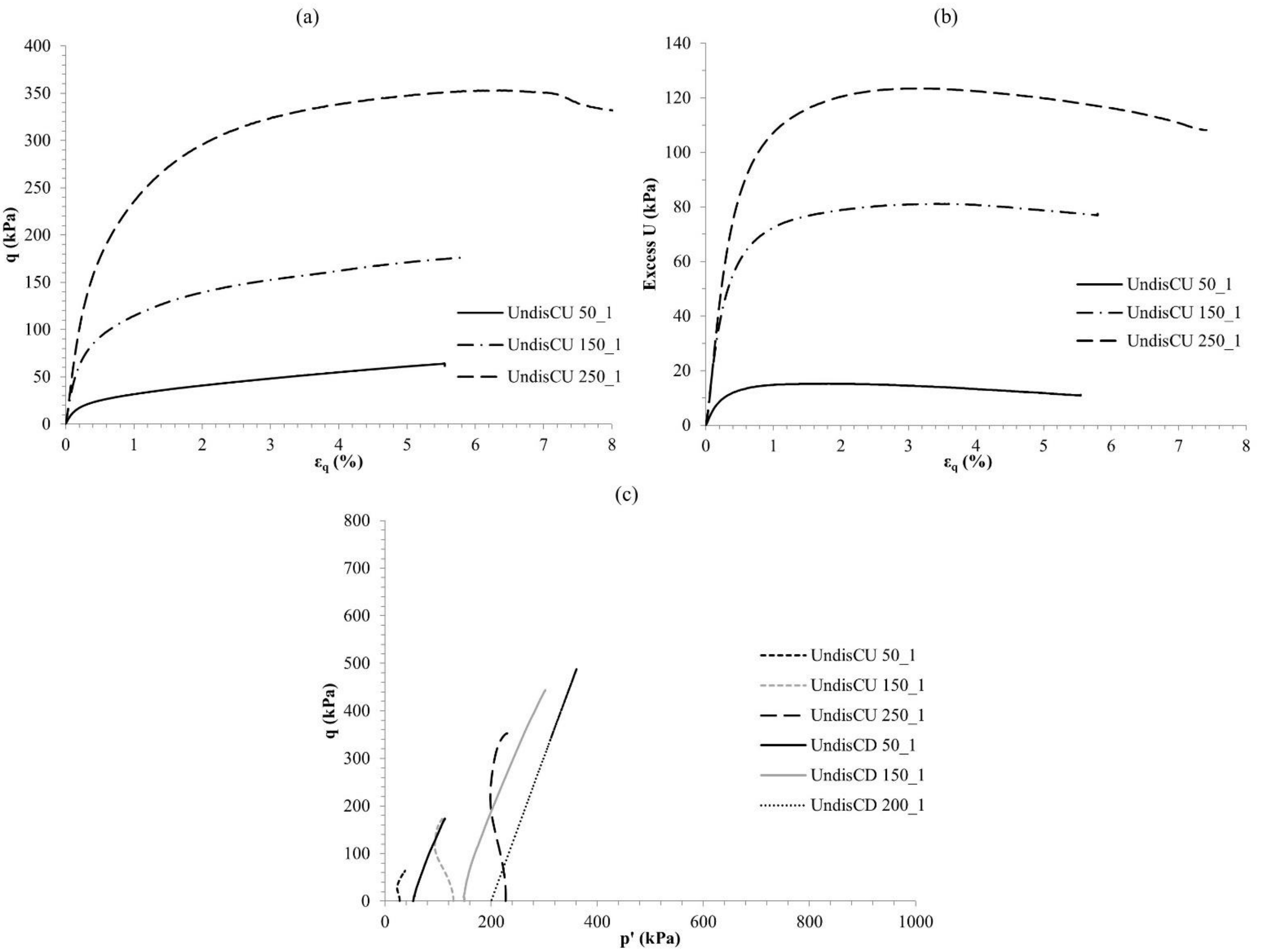


Figure 7: (a) Undrained deviatoric stress – strain response; (b) excess pore pressure – deviatoric strain response and (c) drained and undrained effective stress paths in the deviatoric stress – mean isotropic stress plane for undisturbed Lanton alluvium during shear tests.

The U trends show a general increase with strain, with a peak reached between 1.5 and 3% ε_q, followed by a slow reduction under further straining. However, with continued shearing post-peak, U slightly increased before equalising. This behaviour coincides with the onset of strain localisation and softening. Compared with reconstituted samples, it appears that slightly larger build ups were observed for undisturbed samples.

During the consolidated drained triaxial tests, as reported in Figure 8, work hardening is observed up to approximately 2–2.5% ε_q for samples consolidated and tested at p'_0 of 50–150kPa. This was followed by a pronounced period of strain softening, most likely resulting from the damage of the natural soil structure during shearing. The sample UndisCD 200_1 tested at a higher confining stress exhibited a much more limited softening response, showing work hardening behaviour up to approximately 5% ε_q.

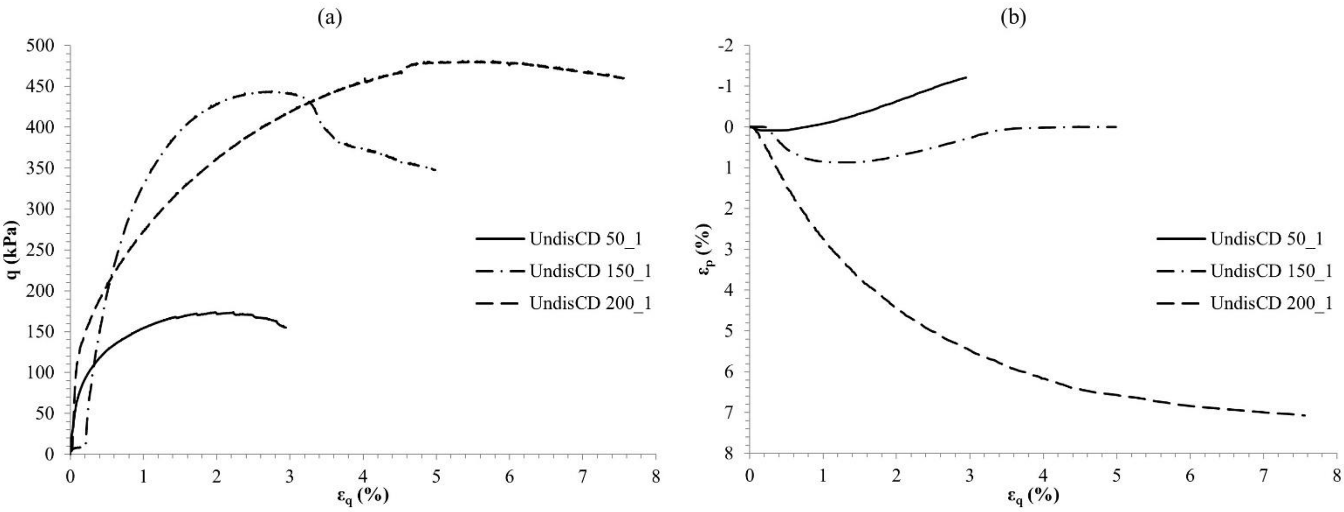


Figure 8: (a) Drained deviatoric stress – shear strain and (b) dilational behaviour of reconstituted Lanton alluvium during shear tests.

Strain localisation and shear plane development were observed within all undisturbed samples. Based on the soil's OCR of 1 obtained from oedometer testing [21], Lanton alluvium is considered normally consolidated. p'_0 normalised undrained elastic stiffness ($E_{u\,p'0}$) and drained elastic stiffness ($E'_{p'0}$) for the undisturbed soil were recorded as $E_{u\,p'0}$ = 63–139MPa and $E'_{p'0}$ = 166–600MPa. The undrained stiffnesses were lower and drained stiffnesses were higher compared with those recorded for the reconstituted soil; namely $E_{u\,p'0}$ = 223–297MPa and $E'_{p'0}$ = 150–215MPa. The slightly higher stiffness values recorded for the undrained reconstituted samples are likely to be attributed to considered to a degree of over-compaction during sample preparation. From these figures, it is clear that p'_0 conditions used during testing influences the elastic moduli. Furthermore, considering the higher yield stresses of undisturbed samples over reconstituted samples, the post-yield softening behaviour can be attributed to the degradation of the bonding based sedimentation structure within the soil. During drained tests, higher peak q values were noted compared with reconstituted samples due to their internal structure which would have caused dilation. Once the peak q had been reached, softening was observed.

The radial stiffness of some undisturbed samples as measured from local strain gauges appeared higher than their axial stiffness, which in turn produced a Poisson's ratio (ν) of approximately 0.33. Such a value is typical for a soft silty alluvial soil, whereby Bowles [22] stated that ν values of 0.3–0.35 may typically be expected. This observation may indicate a degree of stiffness anisotropy within the soil. Further testing with horizontally mounted bender elements at the centre of samples would be required to investigate this postulate.

As for reconstituted samples, higher degrees of dilation were observed within samples which had been consolidated to lower p'_0 values. Figure 6b shows that for tests conducted at p'_0=50 and 150kPa, dilation initiated at much smaller shear strains of 0.3 and 1.2% respectively, compared with reconstituted samples. At these strain levels, samples were generally within 1% ε_q of reaching their yielding points, indicating that samples stress paths were close to the CSL. The sample tested at p'_0=50kPa reached maximum effective stress ratio and therefore failure at ε_q = 3%.

The contraction experienced by sample UndisCD 200_1 became negligible with further straining at ε_q >6%. The dilation coupled with loss of structure may explain the observed post-yielding softening behaviour for samples UndisCD 50_1 and UndisCD 150_1 [23].

Ψ values were calculated to range between 4.2–13.6^o, whereby such variation confirms that higher p'_0 values inhibit soil particle rearrangement. Based on the typical friction angle value of 33^o for loose silty sand as given by Carter and Bentley [24] and the formula of $\psi = \varphi - 30$, the ψ values measured for Lanton alluvium are considered high, regardless of any natural bonding structure and the associated peak and softening behaviour.

3.1.3 28 day cured stabilised alluvium

The effect of stabilisation was observed to produce shear strengths up to four times higher than those measured for reconstituted Lanton alluvium, which was accompanied by brittle behaviour upon failure due to strain localisation and shear plane development. The overall undrained response for the stabilised alluvium is shown in Figure 9a and 9b. The q-ε_a behaviour is generally characterised by work hardening up to peak conditions, followed by strain softening. Increasing confining stress results in peak q values being reached at lower ε_q values; whereby peak values were recorded at approximately 6% strain for samples tested at p'$_0$<100kPa and 2–3% strain for samples tested at p'$_0$>200kPa. The U trends show an initial contraction, proportional with the applied confining stress. A peak in U appears to be coincident with the onset of yielding followed by a reduction in U. A stable U value is reached beyond ε_q = 4–5%. This behaviour is typical of densely cemented soils; whereby for less dense cemented soils, U would be expected to continue increasing towards a steady state [16].

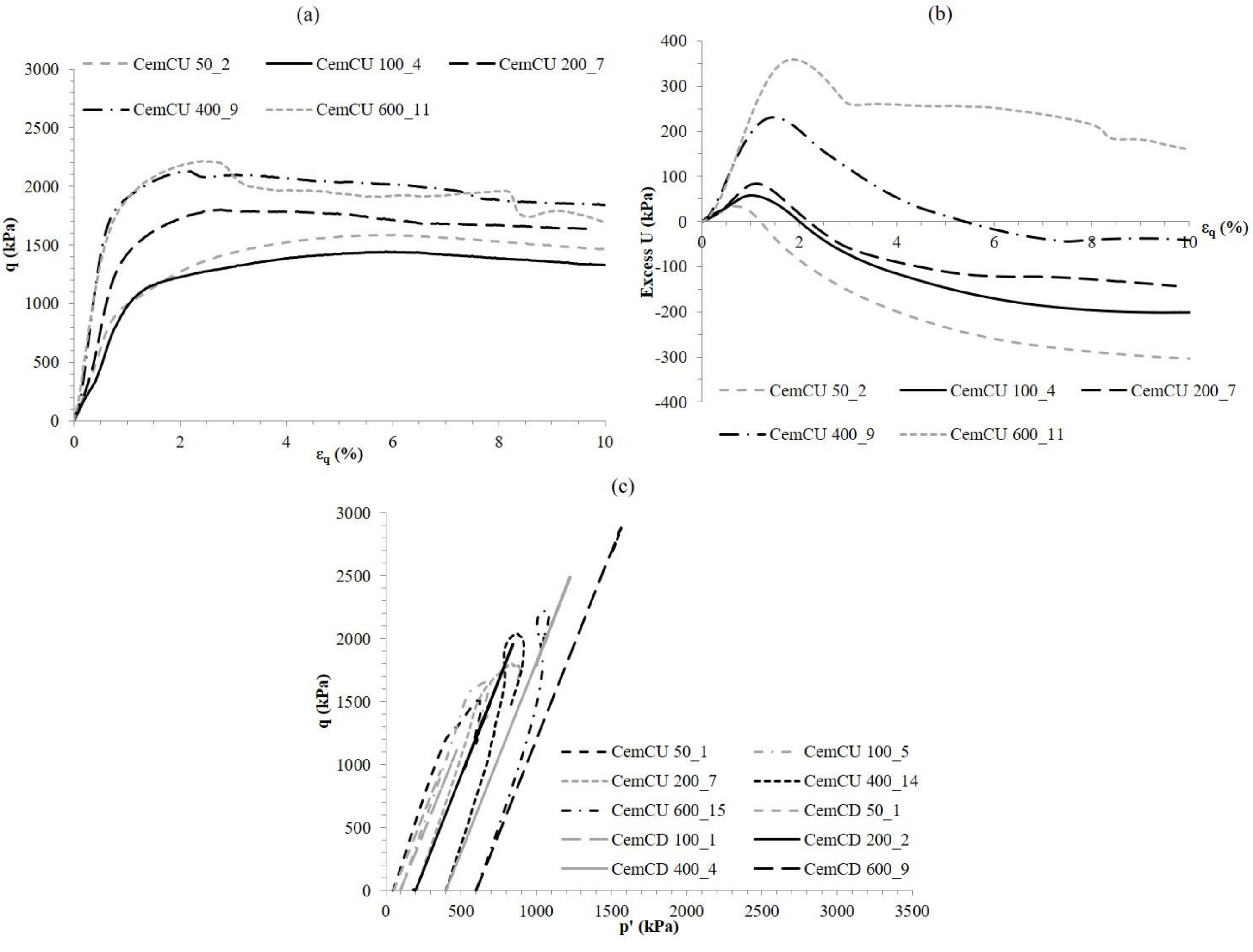


Figure 9: (a) Undrained deviatoric stress – strain response; (b) excess pore pressure – deviatoric strain response and (c) drained and undrained effective stress paths in the deviatoric stress – mean isotropic stress plane for 28 day cured stabilised Lanton alluvium during shear tests.

For samples tested at p'$_0$<200kPa, once U had peaked and started decreasing, they decreased towards negative values. Although this behaviour was not observed by [16] on an artificially cemented silty sand, it was recorded by Ahnberg [15] for cemented post-glacial Swedish clays. This behaviour is commonly observed within dense rocks, whereby the generation of suction occurs close to failure when the material starts to dilate [25] with strain localisation. This U reduction ultimately increased the effective confining stress and therefore strength.

The drained q–ε_q response of the 28 day cured GGBS-NaOH cemented alluvium is displayed in Figure 10a. All samples exhibited non-linear elastic work hardening behaviour up to peak q, followed by strain softening. This behaviour became less pronounced with increasing p'_0; particularly for ≥400kPa. The magnitude of strains at which yielding occurred also increased with increasing p'_0.

The p'_0 applied to samples during testing appears to influence the q-ε_q behaviour of the material. For drained samples tested at $p'_0 \leq 200$kPa, the peak q and corresponding strains were similar – resembling overconsolidated behaviour. However, for drained samples tested for $p'_0 > 400$kPa the maximum q values recorded by samples and their corresponding ε_q increased and resembled normally consolidated behaviour. This was similarly observed by Ahnberg [15] for lime and cement stabilised clays from Sweden, which can be attributed to the material's vertical yield stress (σ'_{qp}) (aka quasi preconsolidation pressure) as derived from one-dimensional consolidation oedometer tests. Ahnberg [15] indicated that σ'_{qp} for cemented soils depends on the level of cementation within the soil matrix and the magnitude of stress applied to the material during curing. A value for σ'_{qp} can be estimated based on an empirical correlation with unconfined compressive strength (UCS), namely σ'_{qp} = 1.3UCS. Sargent et al. [10] undertook UCS and oedometer tests in accordance with BS1377 [11] on the Lanton soil stabilised with 7.5% GGBS-NaOH after 28 days curing. UCS values of approximately 1500kPa were recorded, indicating σ'_{qp} = 430kPa. Hence, for samples tested in this study which had been consolidated to effective stresses $\geq \sigma'_{qp}$, a transition in mechanical behaviour from overconsolidated to normally consolidated behaviour occurred.

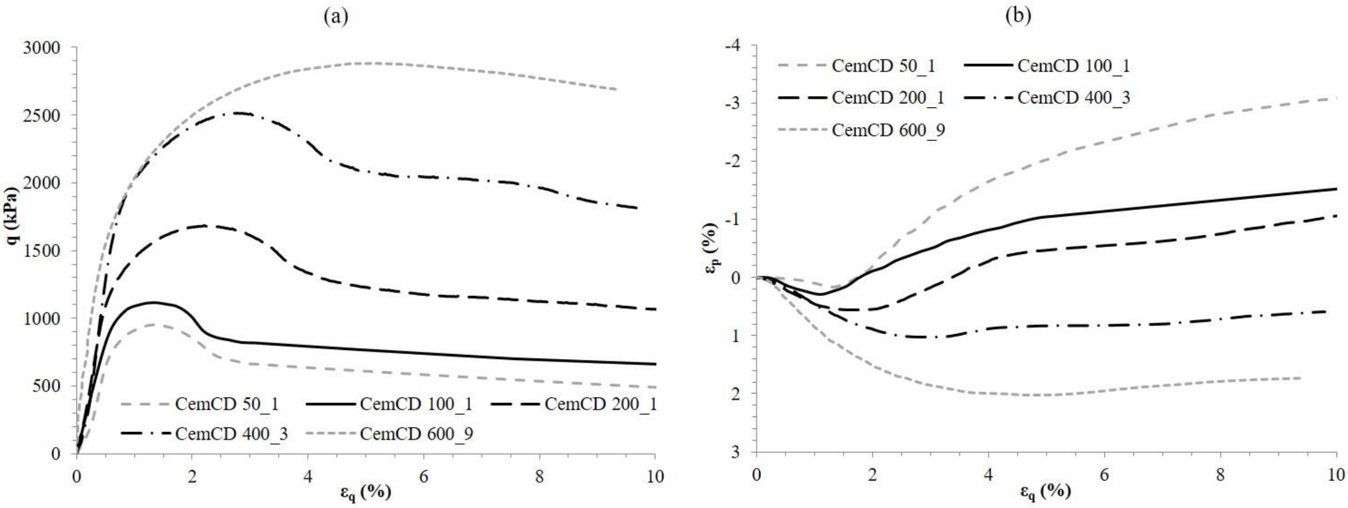


Figure 10: (a) Drained deviatoric stress – shear strain and (b) volumetric – shear strain behaviour of 28 day cured stabilised alluvium samples during shear tests.

$E_{u\ p'0}$ and $E'_{p'0}$ value ranges for the stabilised Lanton alluvium were recorded as 552–2593MPa and 531–3750MPa, respectively. Evidence for stabilised Lanton alluvium's dilative behaviour during shearing is shown in Figure 10b. Volumetrically, samples experienced a combination of contractional and dilational behaviour. Samples characterised by more dilation were tested at p'_0=200kPa, whereby the onset of dilation occurred at ε_q of 1.7%. Negative ε_p was observed over the ε_q range of 3–3.5%. Such pronounced dilation corresponds with the softening, due to the breakdown of the newly formed cementitious bonding structure.

For samples consolidated to p'_0>400kPa, their behaviour was predominantly contractional; thereby demonstrating the influence of p'_0 on shearing behaviour. However, at larger ε_p of 3–5%, samples experienced varying degrees of dilation; albeit no negative ε_p. This complements the suppressed peak and softening behaviour observed in Figure 10a; whereby work hardening dominates particularly for samples tested at p'_0=600kPa. Under such stress conditions, particle rearrangement within samples was less permissible compared with at lower p'_0 of 200kPa. Once samples had failed and reached their critical state, no further ε_p was anticipated as they continued to be sheared. Zero ε_p was not encountered by samples during testing.

Ψ values for the material ranged between 1.2–3.5^{o}, with higher values obtained for samples tested at lower effective stress conditions. Samples achieving their peak effective stress ratios marked the onset of dilational behaviour. Particularly for samples tested at p'_0=200kPa, once the dilation rate peaked at shear strains of approximately 5%, the degree of dilation and the effective stress ratio reduced.

3.2 Failure Envelopes

Using the Mohr's circle at peak strength conditions, the failure envelopes for the three materials have been determined and provided in Figure 11. The reconstituted alluvium was characterised by average values for average effective cohesion (c') of 2.35kPa and effective friction angle (φ') of 34.1^o, whereas the undisturbed Lanton alluvium exhibited a rather similar average φ'of 32^o but higher values of c' equal to 12kPa as a result of the internal natural structure. Theoretically, the undisturbed soil ought to possess higher shear strength properties compared with its reconstituted state, due to the presence of inter-particle bonding within the soil structure. However, at higher stress levels (i.e. σ'_n>200 kPa) the opposite was observed whereby that the Mohr-Coulomb failure envelope for the undisturbed soil is positioned beneath that for the reconstituted soil. This appears emphasised for undrained tests. Considering peak q values recorded during undrained tests were generally lower compared with drained tests under identical p'_0 conditions, the apparent higher strength of reconstituted Lanton alluvium may be attributed to a degree of sample over-compaction. Some of the soil's bonding-based structure may also have collapsed during transport from the field to the laboratory.

Whilst the untreated alluvium behaved as a typical frictional granular material, the mechanical behaviour of the cemented alluvium is largely controlled by its cement content. As previously mentioned, strain localisation and the development of shear failure planes occurred during triaxial compression when stress conditions closely approached their peak.

The applied GGBS-NaOH stabilisation process had a significant impact on the peak strength of the alluvium, whereby the failure envelope was characterised by an average c' of 360kPa as a result of cementation within the soil matrix, and an average φ' of 37^o. This suggests that the applied stabilisation procedure improved not only the cohesive components of the shear strength, but also its frictional components.

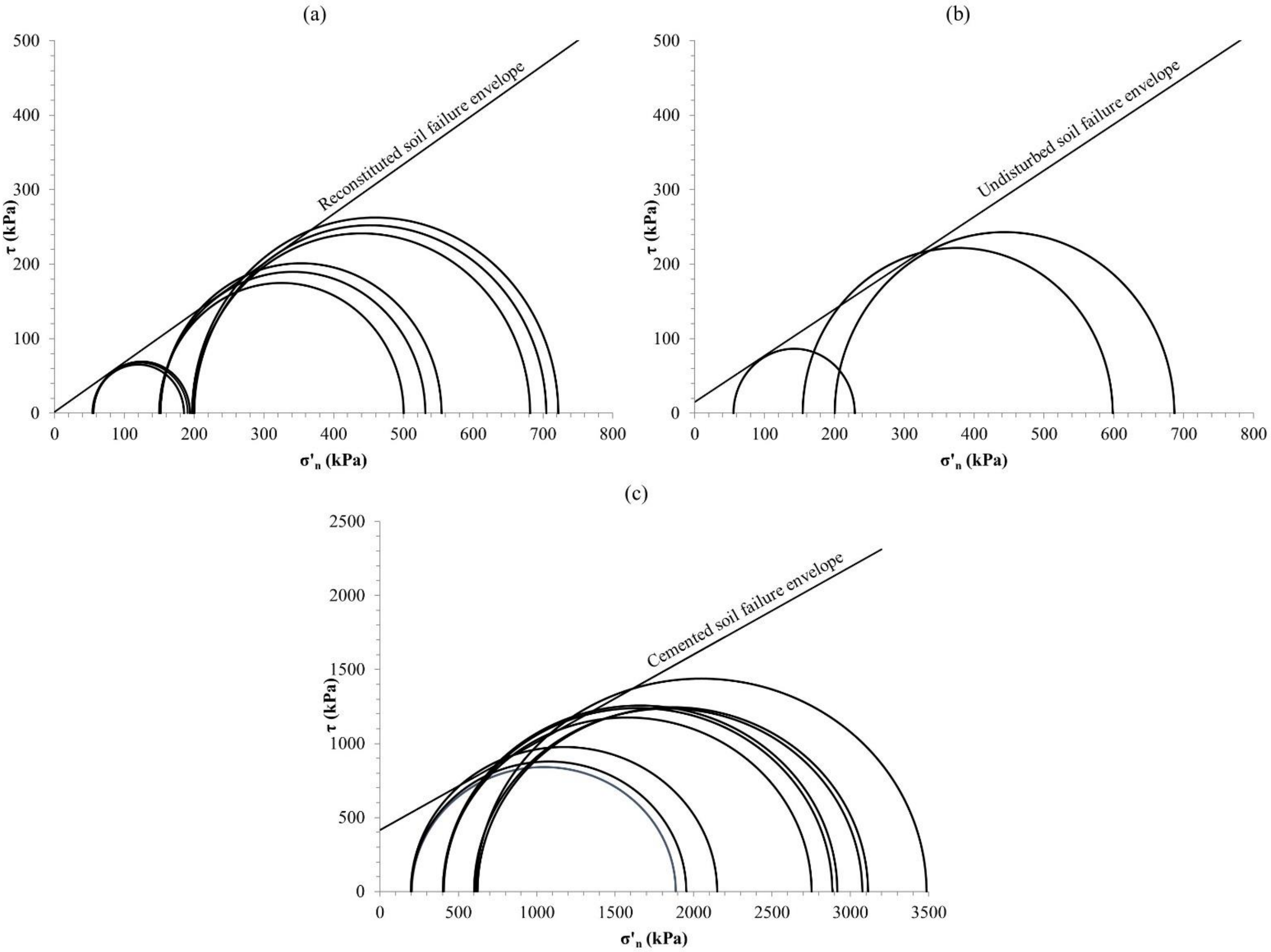


Figure 11: Mohr-Coulomb failure envelopes at peak strength conditions for: (a) reconstituted, (b) undisturbed and (c) 28 day cured stabilised Lanton alluvium.

The effective stress paths followed by the three different materials during both undrained and drained triaxial shearing tests are reported in Figure 12. q and p’ have been normalised by p'_0 to bring the stress paths together for the purpose of defining a single locus within the $q / p'_0 - p' / p'_0$ plane. Both reconstituted and undisturbed Lanton alluvium undrained stress paths within q-p’ stress space demonstrate strain hardening with little softening. Drained stress paths followed by all three materials exhibited the 1:3 slope typically expected for drained shear tests. It is possible that the occurrence of localisation within samples during shear testing may have affected the location of the CSL’s for the three materials. Therefore, this study provides an estimation of the CSL locations to enable comparisons with different soils.

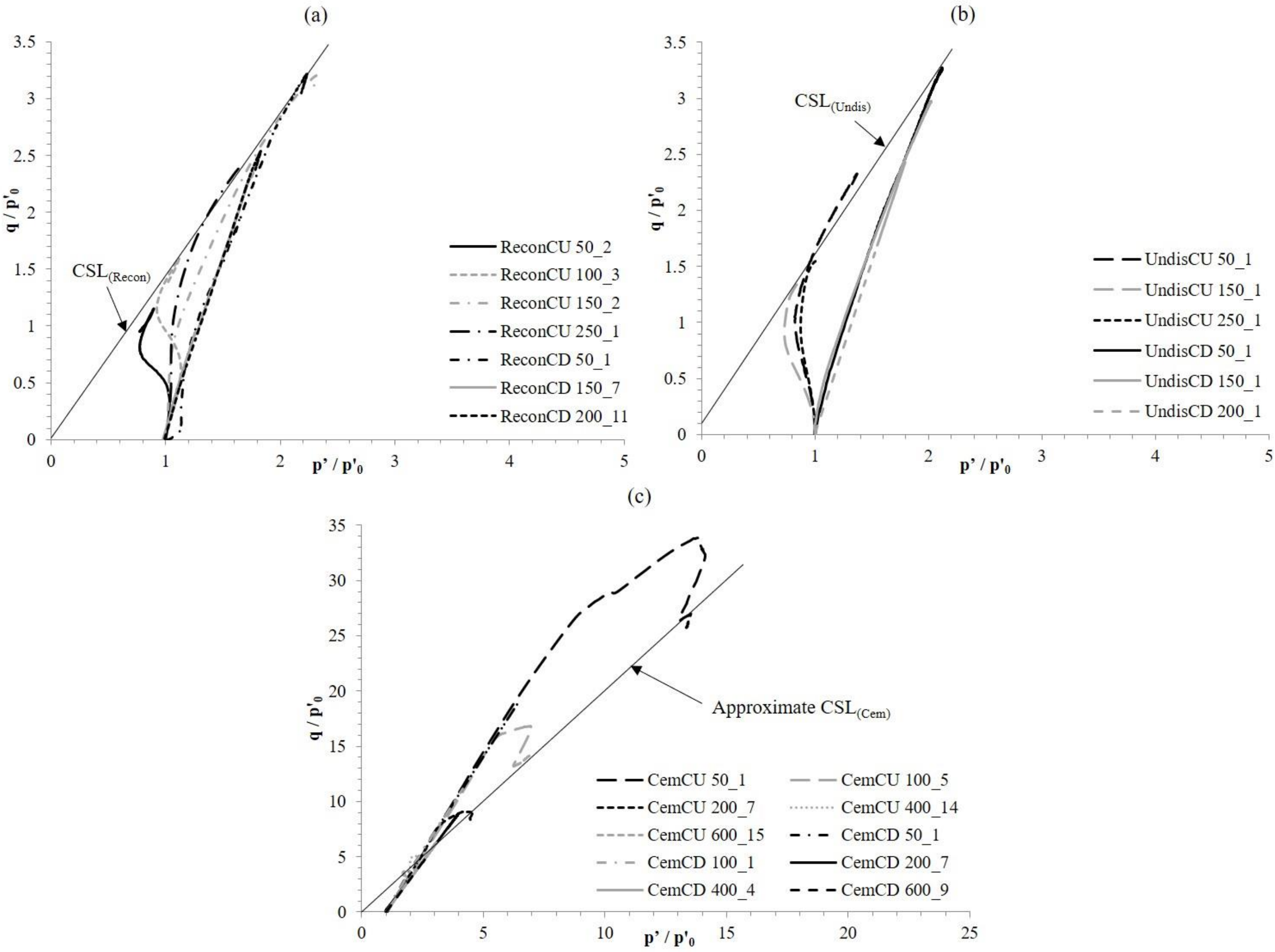


Figure 12: p'_0 normalised drained and undrained effective stress paths for: (a) reconstituted, (b) undisturbed and (c) 28 day cured stabilised Lanton alluvium.

The CSL's for all three materials pass through the origin, as typically expected for frictional granular soils. Per Figure 12a, the average gradient (M) measured for the estimated location of $CSL_{(Recon)}$ was 1.42, deriving a φ' = 35°. Whereas in Figure 12b for the undisturbed alluvium, an M value of 1.37 was measured for the estimated location of the $CSL_{(Undis)}$, deriving φ' = 34°. By examining the stress path data for the stabilised Lanton alluvium in Figure 12c, the estimated $CSL_{(Cem)}$ has a steeper gradient compared with the reconstituted and undisturbed Lanton alluvium, thereby giving a higher M value of 1.88. This provides an estimate of φ' = 40°. Whilst the CSL estimates for effective friction angle are similar to those measured from peak Mohr circles for the reconstituted and undisturbed Lanton alluvium, there seems to be some disparity regarding the friction angle estimates for the stabilised alluvium. This is largely attributed to sample variability regarding level of cementation.

Whilst p' generically influences the shear stiffness of soils, it appears less significant for the stabilised alluvium at p' ≤200kPa. Peak q values for undrained samples were similar at p'_0 values of 50 and 100kPa. For undrained tests undertaken at p'_0>400kPa, increases in peak q and a change in shape of the effective stress paths in q-p' stress space were observed. The normalisation of effective stress paths in Figure 11 shows that samples consolidated to higher p'_0 achieved higher peak q. Per Muir Wood [26], this proved useful in defining a single locus for the stabilised material within the $q / p'_0 – p' / p'_0$ plane.

The shapes of undrained effective stress paths varied according to the level of effective confining stress. For samples tested at p'_0 ≤200kPa, stress paths followed a 1:3 slope similar to drained tests until they reached their yielding point when a sharp phase transformation occurred. The stress paths then followed the failure envelope until sample rupture and failure occurred. For undrained stress paths taken by samples tested at confining stresses ≥400kPa, their shapes resemble those typically expected for soils which predominantly experience work hardening with limited softening. Once these samples started to reach advanced stages of yielding and their corresponding stress paths had reached their failure envelopes, a phase transformation occurred. p' increased whilst q stabilised. Once stress paths reached their peak strength (i.e. defining the failure envelope), q started to decrease. It is thought that this resulted in the stress paths coming down to meet the CSL. However, the stress paths only followed the CSL briefly, as q often suddenly decreased due to brittle rupturing and volumetric increase within the samples.

3.3 Ultimate State Locus in the Volumetric Stress Plane

The end points of stress paths followed by all samples tested in the typical specific volume versus logarithm of mean effective stress (ν – ln p') are reported in Figure 13. The end points are highlighted to determine the likely location of the CSL's for the reconstituted, undisturbed and stabilised Lanton alluvium. It should be noted that the occurrence of strain localisation or non-homogeneity of deformation within the tested samples can prevent the determination of the exact location of the critical state line. Nevertheless, a general approximation of the position of the ultimate state locus for the three materials can still be obtained. The data plotted in Figure 13 shows how the compression curve end points and the CSL's for the GGBS-NaOH stabilised alluvium samples plot well above the respective data for the reconstituted and undisturbed samples.

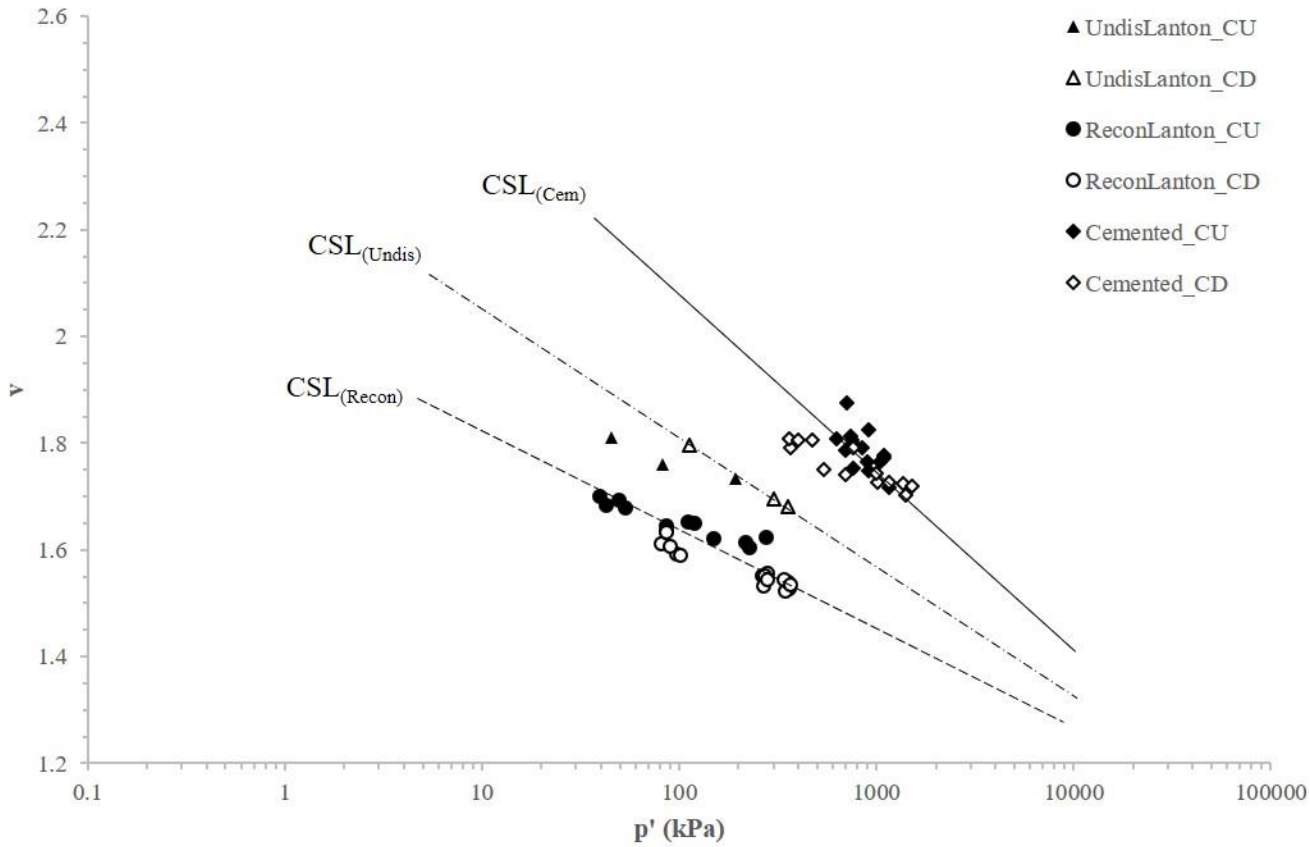


Figure 13: Possible CSL surfaces for reconstituted, undisturbed and stabilised Lanton alluvium in the ν – ln p' stress plane.

It is clear that the presence of internal structure, either in the form of natural post-sedimentation structure or artificial cementation, results in an upward movement of the CSL. Such movement is greater for the stronger structure provided by the GGBS-NaOH stabilisation. However, it is difficult to comment on the relationship between the slope of the CSL's, due to the relatively limited range of mean effective stress and the uncertainty relating to strain localisation and inhomogeneity of deformation. According to critical state soil mechanics theory, it may be assumed that the ultimate compression locus for the undisturbed and stabilised Lanton alluvium will converge with the intrinsic compression line for the Lanton alluvium at very high stress levels. The data trends shown in Figure 13 appears to generally corroborate such an assumption, based on the mean effective stresses experienced by the samples tested. However, Todisco and Coop [27] determined that soils characterised by more complex particle size distributions and mineralogies may not exhibit convergence behaviour. Given the cement bonding-based structure and thus more complex mineralogy of the stabilised Lanton alluvium due to the inclusion of the GGBS-NaOH binder and cementitious gels, further triaxial testing involving higher strains and effective confining stresses would be required to determine whether convergence will occur.

3.4 Stress – Dilatancy Relationship

The stress-dilatancy relationships for the three investigated materials are reported in Figure 14 to allow a direct comparison among the materials and understand the influence of the presence of both natural and artificial cemented structure. The stress-dilatancy behaviour of reconstituted samples is displayed in Figure 14b, whereby all samples display frictional behaviour. They compress to ultimately reach similar critical state M values of 1.3–1.4, with some limited dilation associated with reaching the peak strength ratio. The data seems to follow the typical linear trend between stress ratio and incremental strain ratio, after the elastic component of deformation becomes negligible. The undisturbed Lanton alluvium samples also reached critical state ratio (M) values (i.e. q / p') of 1.3–1.4, although they experienced larger dilation during testing (Figure 14a). The relationship between stress ratio and strain ratio seems to be governed by a much flatter line if compared with the reconstituted Lanton alluvium. This is thought to be a consequence of internal structure and some natural bonding between the soil particles.

The presence of strong cementitious bonding within the stabilised Lanton alluvium results in a unique critical state stress ratio, which appears to decrease with the applied stress level as shown in Figure 14c. The strength contribution provided by the cementitious bonding is generally stress dependent and becomes less significant as the testing confining stress increases. Interestingly, for all of the samples tested, the dilatancy stress- relationship is initially very flat (gradient close to zero), which confirms that the peak strength of the stabilised alluvium samples is not related to a frictional mechanism, but largely governed by the artificial cementation. After a peak rate, the dilation reduces together with the stress ratio which agrees with the observations of Rios et al. [16] on artificially cemented soils and Coop and Wilson [28] on sandstones. For the stabilised soil, the ultimate value of the peak strength ratio is considerably higher than that of the reconstituted and undisturbed alluvium. This suggests that during shearing, the stabilised material does not evolve in the original non-cemented reconstituted alluvium.

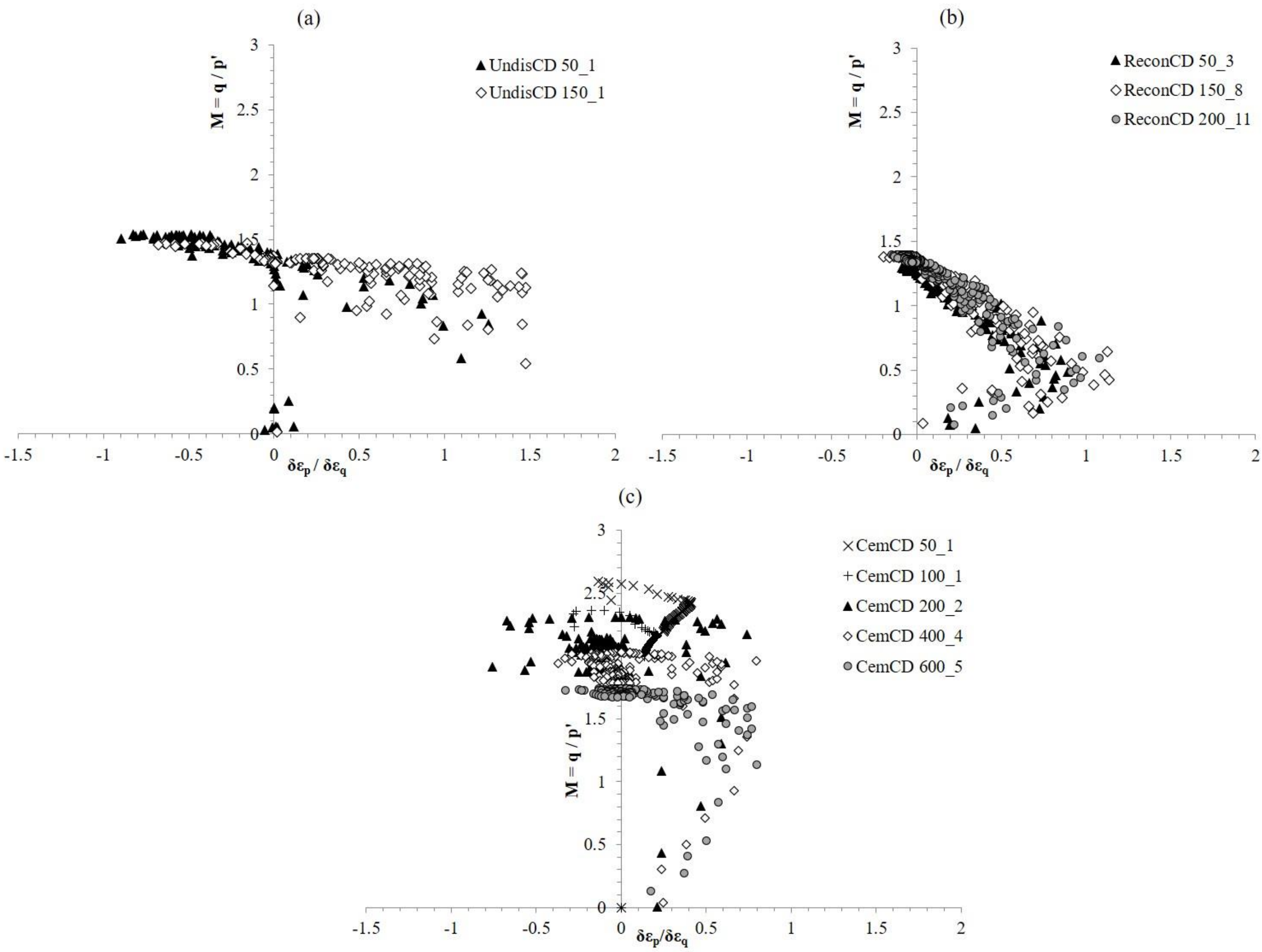


Figure 14: Stress-dilatancy relationship for (a) reconstituted, (b) undisturbed and (c) 28 day cured stabilised Lanton alluvium.

3.5 Shear stiffness

3.5.1 Characterisation of the Small Strain Stiffness

Bender elements and measurements from local instrumentation attached to the middle third of samples were used to assess the relationship between the initial shear stiffness (G_{max}) and the applied p'_0. An average G_{max} value of 89MPa was obtained from bender element measurements performed on samples consolidated to a p'_0 of 50kPa prior to shearing. For the stabilised alluvium, bender element and local strain measurements indicated G_{max} values to range between 1053 and 1813MPa for the p'_0 range of 50–600kPa.

The relationship between G_{max} and the applied confining stress for the three investigated materials are reported in Figure 15. While the derivation of such a relationship for the reconstituted and undisturbed Lanton alluvium is beyond the scope of this paper, the results of Figure 15 demonstrate that the proposed stabilisation method provides an approximate 13 fold increase in initial shear stiffness.

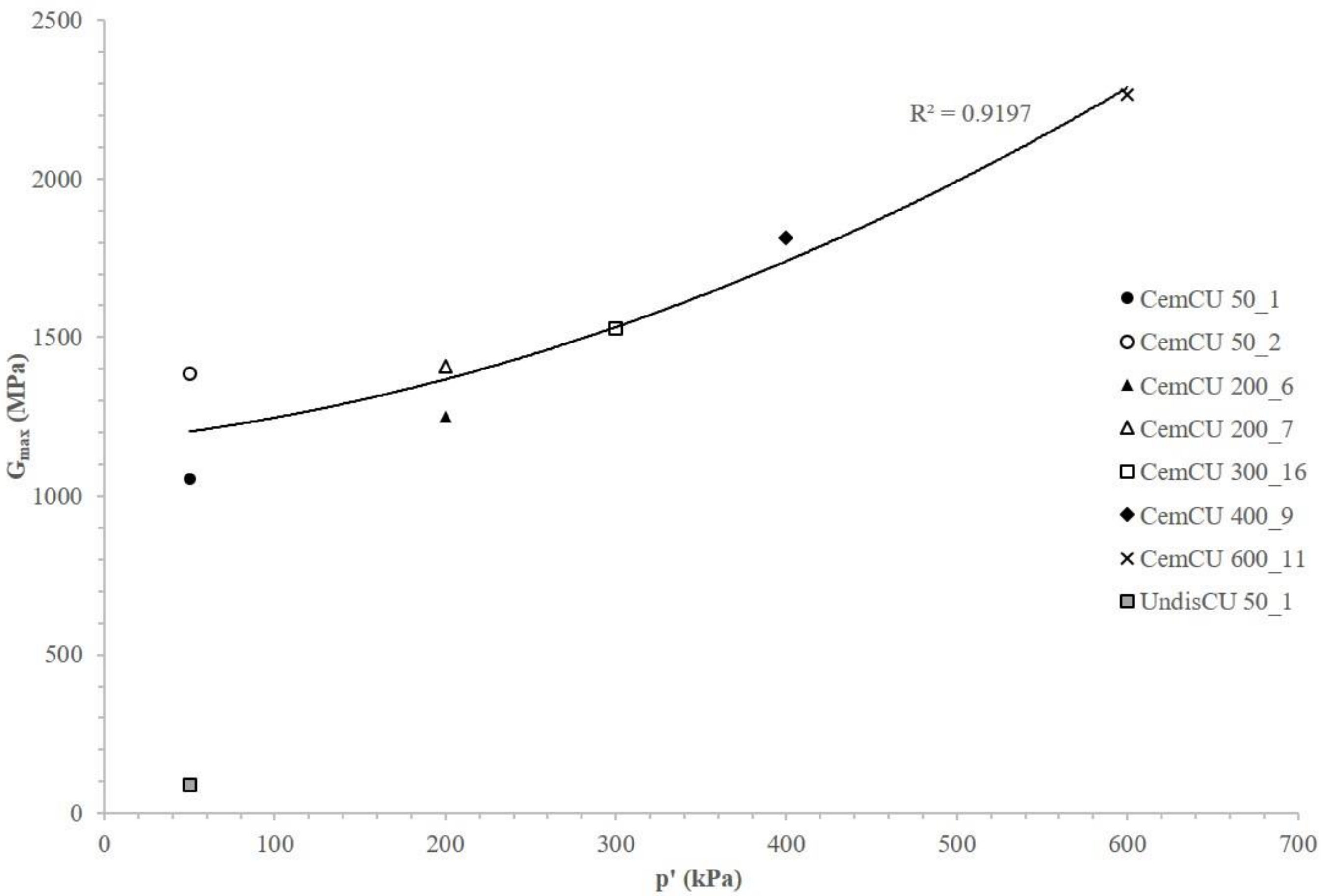


Figure 15: Relationship between G_{max} and initial mean effective stress for undisturbed and 28 day cured stabilised Lanton alluvium.

Figure 15 shows that the value of G_{max} for the stabilised Lanton alluvium is not proportional to the applied confining stress. Variability of the data does not assist in this situation, but it may appear that, for lower values of mean isotropic stress (i.e. <200kPa), the G_{max} is constant or may even decrease with increasing mean isotropic stress. This was similarly observed by Verastegui-Flores and Van Impe [29] for a cemented kaolin soil. In this stress range, the structure of the stabilised material is entirely governed by the cementation and an eventual reduction in shear stiffness (G) with stress level may be associated with a collapse of the cemented soil structure. For isotropic stress levels greater than 200kPa, the G_{max} value increases as it would be expected for non-cemented frictional soils. This suggests that the frictional nature of the soil matrix is activated for these stress levels.

3.5.2 Degradation of shear stiffness during shearing

The measured G degradation behaviour for the undisturbed and stabilised Lanton alluvium is presented in Figure 16, whereby G values were normalised with respect to the measured G_{max} value. Data scatter is observed within the small strain range, which may be due to the slightly lower resolution of stiffnesses calculated from the axial and radial LVDT's local strain gauges compared with bender element measurements. For the undisturbed Lanton alluvium, G degrades significantly at small strains. The degradation is much more rapid if compared with highly structured cohesive soils such as London Clay, as tested by Gasparre [19] whose initial structure and highest stiffnesses are retained up to shear strains of approximately 0.01%.

For the majority of stabilised alluvium samples, the onset of G degradation during compression occurs at approximately 1% shear strain. The shear strain levels at which the shear stiffness of the cemented samples starts to degrade is approximately three orders of magnitude higher than those measured for the undisturbed alluvium. This highlights the level of improvement in mechanical behaviour provided by the addition of the GGBS-NaOH binder and 28 days curing.

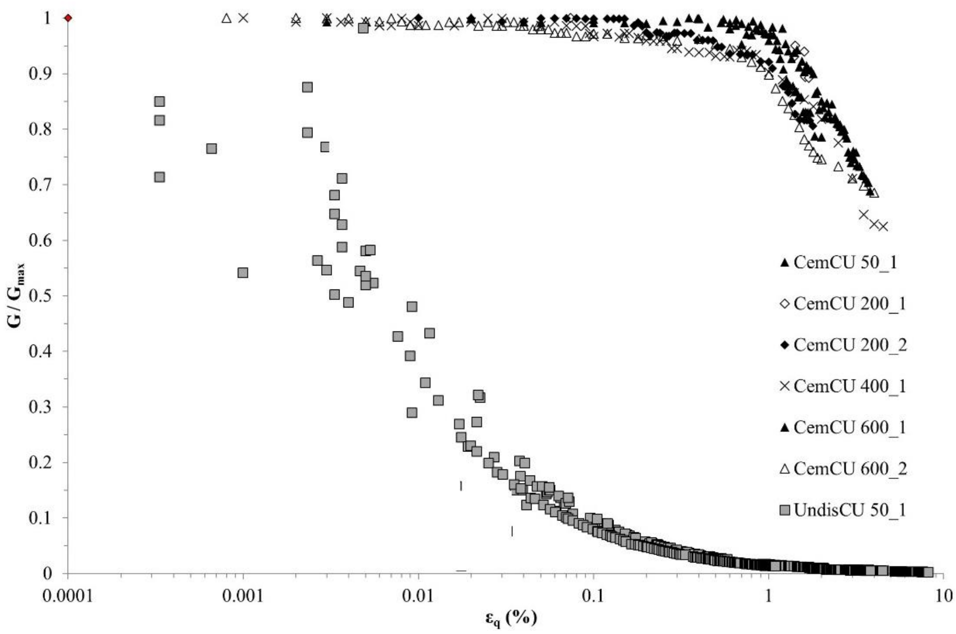


Figure 16: Normalised shear stiffness degradation curves for the undisturbed and 28 day cured stabilised Lanton alluvium.

4.0 Conclusions

An extensive experimental triaxial testing programme was undertaken on reconstituted, undisturbed and stabilised Lanton alluvium samples to characterise the improvement in mechanical behaviour provided by the precipitation of cementitious gels (most likely C-(N)-A-S-H) derived from a new low carbon GGBS-NaOH binder [21]. Findings presented by Sargent et al. [10] informed the work presented in this paper, which revealed that the engineering performance of the GGBS-NaOH binder at a dosage of 7.5% was comparable to using a 10% dosage of CEM-I, thereby meeting minimum strength requirements defined by the EuroSoilStab [12] standard. All cemented samples had a controlled binder dosage of 7.5% (i.e. 107kg m^{-3}). The analysis of the experimental results has revealed the following insights:

- The use of the GGBS-NaOH binder proved successful in significantly increasing the shear strength of the initially soft and sensitive Lanton alluvium soil by a factor of 13 after 28 days curing. The relationship between initial shear stiffness and effective stress level is non-linear: the stiffness appears to be chiefly governed by the cemented structure of the material for low stress levels up to 200kPa. The initial stiffness of the cemented soil was also significantly improved, which only started to degrade at shear strain levels of 1% - approximately three orders of magnitude higher than the untreated undisturbed alluvium.

- GGBS-NaOH stabilised samples exhibited higher values of maximum dilation for the same confining stresses. Such pronounced dilation corresponds with the onset of softening, due to the breakdown of the newly formed internal cementitious bonding structure.
- The peak strength of the stabilised Lanton alluvium was less influenced by the frictional mechanism but largely governed by the artificial cementation. Only after the peak rate is reached, the dilation reduces together with the stress ratio to eventually reach their ultimate values.
- Possible locations for the untreated and GGBS-NaOH cemented alluvium's critical state lines have been defined in the mean effective stress and v – ln p' stress planes. The CSL surfaces for the reconstituted and undisturbed soil are indicated to be approximately in parallel with each other in the v – ln p' stress plane. The stress paths for the cemented alluvium were akin to heavily over-consolidated soils or soft rocks, defining a new CSL surface located above those for the natural soil.

Further research is required for characterising the mechanical behaviour of the GGBS-NaOH cemented soft alluvial soils under higher effective confining pressures, with focus on fatigue, creep and response to dynamic and cyclic loading conditions associated with earthquake phenomena and modern engineering infrastructure such as high-speed railways.

For soils that are softer and more problematic compared with Lanton alluvium, higher binder dosages (i.e. ≥10%) may be required to achieve high strengths. Additionally, the GGBS-NaOH ratio will require careful customisation for individual projects, whereby higher NaOH concentrations would be required to effectively stabilise soils characterised by a low pH. However, using high concentrations of NaOH will result in the binder becoming less environmentally and financially sustainable. The mineralogy / chemistry of a soil (i.e. organic and sulphate contents) being considered for stabilisation must be investigated prior to selecting the binder to be used on site. Failure to do so may result in ineffective stabilisation and potentially worsen ground conditions in the long term due to the formation of structurally unfavourable minerals such as ettringite.

References

[1] Sargent, P., 2014. Alkali activated mixtures for soil stabilisation, in Pacheco-Torgal, F., Labrincha, J., Leonelli, C., Palomo, A. and Chindaprasit, P. (Eds.), Handbook of alkali activated cements, mortars and concretes. Woodhead Publishing.

[2] Davidson, L. K., Demirel, T. and Handy, R. I., 1965. Soil Pulverization and Lime Migration in Soil Lime Stabilisation. Highway Research Record. 92, 103-126.

[3] Tutumluer, E., 2012. Short course notes for Geotechnical Aspects of Pavement Design and Construction – ASCE GeoCongress 2012 "State of the Art and Practice in Geotechnical Engineering", 25 – 29th March 2012, Oakland, CA, USA.

[4] Ghadir, P. and Ranjbar, N., 2018. Clayey soil stabilization using geopolymer and Portland cement. Construction and Building Materials. 188, 361-371.

[5] McLellan, B. C., Williams, R. P., Lay, J., van Riessen, A. and Corder, G. D., 2011. Costs and carbon emissions for geopolymer pastes in comparison to ordinary portland cement. Journal of Cleaner Production. 19, 1080-1090. Doi: 10.1016/j.jclepro.2011.02.010.

[6] Cristelo, N., Glendinning, S. and Teixeira Pinto, A., 2011. Deep soft soil improvement by alkaline activation. Proceedings of the Institution of Civil Engineers, Ground Improvement. 164 (2), 73-82. Doi: 10.1680/grim.900032.

[7] Sargent, P., Hughes, P. N., Rouainia, M. and White, M., 2013. The use of alkali activated waste binders in enhancing the mechanical properties and durability of soft alluvial soils. Engineering Geology. 152, 96-108. Doi: 10.1016/j.enggeo.2012.10.013.

[8] Habert, G., d'Espinose de Lacaillerie, J. B. and Roussel, N., 2011. An environmental evaluation of geopolymer based concrete production: reviewing current research trends. Journal of Cleaner Production. 19, 1229-1238. Doi: 10.1016/j.clepro.2011.03.012.

[9] Bernal, S. A., 2014. The resistance of alkali-activated cement-based binders to carbonation. In: Pacheco-Torgal, F., Labrincha, J., Leonelli, C., Palomo, A. and Chindaprasit, P. (Eds.), Handbook of Alkali-activated Cements, Mortars and Concretes. Woodhead Publishing. ISBN 978-1-78242-276-1.

[10] Sargent, P., Hughes, P. N. and Rouainia, M., 2016. A new low carbon cementitious binder for stabilising weak ground conditions through deep soil mixing. Soils and Foundations (The Japanese Geotechnical Society). 56 (6), 1021-1034. Doi: 10.1016/j.sandf.2016.11.007.

[11] BSI, 1990. BS: 1377 Incorporating Amendment No. 1, Methods of test for Soils for Civil Engineering Purposes. British Standards Institution, Milton Keynes.

[12] EuroSoilStab, 2002. Development of design and construction methods to stabilise soft organic soils: Design guide soft soil stabilisation. CT97-0351. Project No.: BE 96-

[13] Ge, L., Wang, C-C., Hung, C-W., Liao, W-C. and Zhao, H., 2018. Assessment of strength development of slag cement stabilized kaolinite. Construction and Building Materials. 184, 492-501.

[14] USGS, 2015. Volcanic Ash Impacts & Mitigation – Water Supply. [online]. Available at: <https://volcanoes.usgs.gov/volcanic_ash/water_supply.html>. Last accessed: 7th September 2018.

[15] Ahnberg, H., 2007. On yield stresses and the influence of curing stresses on stress paths and strength measured in triaxial testing of stabilized soils. Canadian Geotechnical Journal. 44, 54-66. Doi: 10.1139/T06-096.

[16] Rios, S., Viana da Fonseca, A. and Baudet, B. A., 2014. On the shearing behaviour of an artificially cemented soil. Acta Geotechnica. 9, 215-226. Doi: 10.1007/s11440-013-0242-7.

[17] Hansson, T., Parry, L., Graham, M., Troughton, V. and Eriksson, H., 2001. Limix: a deep dry mixing system used at Channel Tunnel Rail Contract 440. Proceedings of Underground Construction 2001 Symposium and Exhibition, London. Institute of Materials, Minerals and Mining, London.

[18] American Society for Testing and Materials (ASTM), 2008. Designation: D 4543-08 Standard Practices for Preparing Rock Core as Cylindrical Test Specimens and Verifying Conformance to Dimensional and Shape Tolerances. Annual Book of ASTM Standards. 4.08, West Conshocken: Pa.

[19] Gasparre, A., 2005. Advanced laboratory characterisation of London Clay. PhD thesis. Department of Civil and Environmental Engineering, Imperial College London.

[20] Rees, S., Le Compte, A. and Snelling, K., 2013. A new tool for the automated travel time analyses of bender element tests. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris 2013. Technical Committee 212. 2843-2846.

[21] Sargent, P., 2015. Secondary minerals to replace cement in stabilising an alluvium. Ph.D. thesis, Newcastle University, UK.

[22] Bowles, J. E., 1977. Foundation Analysis and Design. 2nd edition, McGraw-Hill, New York. ISBN 10: 0070067503.

[23] Obrzud, R. F., 2010. On the use of the Hardening Soil Small Strain model in geotechnical practice, in Numerics in Geotechnics and Structures. Eds Zimmerman et al., Elmepress, Lausanne.

[24] Carter, M. and Bentley, S. P., 1991. Correlations of soil properties. Pentech Press ltd., ISBN 0-7273-0317-1.

[25] Mesri, G., Jones, R. A. and Adachi, K., 1972. Annual report on Influence of Pore Water Pressure on the Engineering Properties of Rock. Department of Civil Engineering, University of Illinois,

Urbana, Illinois, USA. [online]. Available at: <http://www.dtic.mil/dtic/tr/fulltext/u2/740169.pdf>. Last accessed: 10th June 2014.

[26] Muir Wood, D., 1990. Soil Behaviour and Critical State Soil Mechanics. Cambridge University Press. ISBN: 0521332494.

[27] Todisco, M. C. and Coop, M. R., 2016. Normalisation techniques for slowly-converging soils. Procedia Engineering. 158, 110-115. Doi: 10.1016/j.proeng.2016.08.414.

[28] Coop, M. R. and Wilson, S. M., 2003. On the behaviour of Hydrocarbon Reservoir Sands and Sandstones. Journal of Geotechnical Engineering, ASCE. 129 (11), 1010-1019.

[29] Verastegui Flores, R. D. and Van Impe, W. F., 2009. Stress-strain behaviour of artificially cemented Kaolin clay. Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering, Hanza, M. et al. (Eds). 283-286.