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Shear Response of Lime/GGBS-Stabilised High-Sulphate-Bearing Clay under Consolidated-Undrained Conditions

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Abstract: This study investigated the consolidated undrained shear behaviour of a stabilised high-sulphate soil system. Lime was used to stabilise the soil with the inclusion of ground granulated blast furnace slag (GGBS) as an ettringite suppressor. Both volumetric changes and shear strength responses of the stabilised soil containing various proportions (10%, 20%, and 30%) of sulphates were examined with corresponding pore pressure developments and stress path changes using a modern computer-controlled stress-path triaxial system. Results indicated greater volume change for the non-stabilised soils containing lower amounts of sulphates. This shows that calcium sulphate, which is a soluble salt with relatively less alkalinity, is capable of binding particles of soils together. The amount of volume change increased with the quantity of sulphates in the stabilised soil even though the quantity of GGBS utilised as an ettringite suppressor was twice more than that of the lime. This was attributed to the unreacted gypsum (calcium sulphate) used, which resulted in a decrease in the overall specific weight, thus affecting the texture of the stabilised mix and causing an increment in pore sizes. Generally, the stabilised sulphate soils showed some initial ductile responses with the yielding followed by an almost perfectly plastic behaviour up to about 6–8% of the strain before finally undergoing small amounts of strain-softening. Lastly, higher levels of plastic failure were achieved and at higher constant effective stress for the stabilised soils containing lower percentages of sulphates.

Keywords: lime; GGBS; gypsum; soil stabilisation; undrained shear strength; kaolinite; sulphate soil; gypseous soil

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1. Introduction

Sulphate-bearing soils are present in critical areas of the globe where the construction of civil engineering structures or similar land-based developments are unavoidable. Sulphates commonly found in soils such as sodium sulphate or Glauber's salt (Na_2SO_4), hydrated calcium sulphate or gypsum ($CaSO_4.2H_2O$), and magnesium sulphate or epsomite ($MgSO_4$) are either naturally occurring or imparted on the soil through some secondary sources [1,2].

Research has shown that the use of calcium oxide- (or hydroxide)-based agents (such as lime and cement) for the stabilisation of sulphate-bearing clays can lead to unwanted heaving (or sulphate-induced heaving) when inundated with water [3,4]. Researchers have also highlighted that sulphate-bearing soils may be prone to loss of strength, stability, and durability as a result of the generation and propagation of expansive reactions when stabilised with calcium oxide-based agents [5–9]. It is known that, within the mechanism of sulphate soil-binder reaction, a suitable pH condition (between 11 and 13) can develop and lead to the formation of an expansive mineral called "ettringite" ($(Ca_6[Al\ (OH)_6]_{20}\cdot 24H_2O\}\ (SO_4)_2\ H_2O$) or "thaumasite" $\{Ca_6\ [Si\ (OH)_6]_{20}\ (SO_4)\ (CO_3)_2\cdot 24H_2O\}$ in the hydrated system.

Studies on the suppression of the deleterious effects of ettringites and their formation mechanisms by the use of GGBS and other sustainable means in the stabilisation of sulphate soils rich in calcium oxide or (hydroxide) (such as cement and lime) are rife [4,6,10–23].

It has been reported that high proportions of GGBS (up to 80%) compared to another calcium oxide (or hydroxide) agent (e.g., cement and lime) are required in the stabilised soil mix to effectively prevent attacks by sulphates [11].

Although numerous investigations have been conducted in the past on the use of GGBS for the prevention of unwarranted linear or volumetric deformation of calcium oxide-based treated sulphate soils as noted above; however, studies that relate to the strength capacity of these stabilised sulphate-bearing soils are quite limited [5,6,15,16,21,24]. Earlier studies carried out by Wild [16] explored the effect of GGBS on the unconfined strength of lime-treated kaolinite in the presence of sulphates. Gypsum (4%) was used to mirror the influence of sulphate in the kaolinite while the minimum quantity of lime used was 4% with varying proportions relative to that of the GGBS utilised but maintaining an overall total of 10% of both (lime and GGBS) by weight of the soil. The samples were moist-cured for 7 and 28 days at 300 °C and 100% humidity. Results indicated an increase in strength corresponding to an increase in the GGBS-to-lime ratio in the absence of gypsum. However, with the sulphate added, an increase in the GGBS-to-lime ratio produced a "non-systematic" variation in strength. A further examination of the lime–gypseous soil–GGBS system was conducted by Wild et al. [24] but with the total percentage of GGBS and lime in the mix being 6% and gypsum varying between 1% and 2% by dry weight of the soil. Testing was carried out under the same conditions as Wild [16], with the results suggesting that the development of strength was dependent on the proportion of the additives used and curing. For instance, the most significant improvement was due to high lime/low GGBS in the presence of gypsum at 28 days. However, at 7 days, the greatest strength experienced by the treated sulphate soil was made possible by a high lime-to-GGBS ratio. Li et al. [15] recently studied the effect of sample soaking on the unconfined compressive strength of lime/GGBS-treated sulphate soil. Three weight ratios of lime and GGBS (1:9, 2:8, and 3:7) were used, and the treated soils compacted at optimum conditions. A duration of 35 days and 21 days of soaking were carried out on specimens cured (in dry and moist conditions) for 7 and 21 days, respectively. Results indicated that specimens cured in dry conditions prior to soaking had the best strength improvement but with the greatest strength produced for the mix having the lowest lime-to-GGBS ratio. Hence, strength was affected more by the conditions of curing rather than soaking. Adeleke et al. [6] investigated the effect of MgO waste and GGBS combinations (maintaining a total of 10% by dry weight of soil) on the unconfined compressive strength of a naturally occurring sulphate soil (containing 22% gypsum) stabilised by varying proportions (6%, 8%, and 10%) of Portland cement. The samples were compacted at optimal conditions and moist-cured for 7, 28, and 56 days. The compressive strength of the MgO/GGBS-stabilised sulphate soil was significantly improved in general but with the highest strength produced by using 10% of Portland cement.

The foregoing studies examined the unconfined compressive strength (mostly the peak stress at failure) of sulphate soils stabilised by agents rich in calcium oxide (or hydroxide) with a further incorporation of GGBS to prevent the harmful effects of sulphates. However, these studies may be quite insufficient to provide design guidelines for successful implementation of a stabilised sulphate soil project because of their limited and in some cases, their inclusive nature of outcomes. The results of unconfined compressive strength utilised to establish the general design parameters were presented in terms of the final yielding condition or peak strength from these studies. Hence, no account was taken of the sequence of stress changes throughout the test or the design problems for which the parameters are needed. Hence, in order to carry out design of complex problems, specific parameters with details of changes in stresses, volumetric strains, pore pressures, etc. are required. Therefore, the present research aims to investigate in further detail, the consolidated undrained shear strength (with an examination of pore pressure developments and stress paths) of a stabilised sulphate soil system consisting of gypsum, lime, and GGBS. Moreover, soils with very high sulphate contents have rarely been considered by previous studies. Hence, this research shall uniquely examine the effect of GGBS on the undrained strength capacity of lime-treated soils containing up to 30% of sulphates.

2. Materials and Methods

2.1. Clay

The soil material used in this study was kaolin clay (or kaolinite) obtained in a processed form. This choice was made to ensure that a clay material of known composition (i.e., "pure") was utilised given that some organics, sulphides or sulphates, and minor compounds would have been skimmed off during processing. This enables the physicochemical and micro-structural behaviour of the stabilised clay to be accorded the true composition of the actual mix. Moreover, as noted by Rollings et al. [25] and Knopp et al. [4] in their research, kaolinite could be regarded as being very chemically rich in the alumina compound when compared to some of the other expansive clay minerals because it is capable of giving up more alumina in a high pH environment to participate in the formation of ettringite minerals, thus becoming even more prone to sulphate attacks. The chemical composition of the kaolinite is given in Table 1. The basic properties of the soil used are given in Table 2.

Table 1. Chemical composition of materials.

Oxide Composition	**Kaolinite**	**Lime**	**GGBS**
	(%)	**(%)**	**(%)**
SiO_2	49.00	1.80	34.10
Al_2O_3	36.00	0.7	13.00
Fe_2O_3	0.75	0.3	0.51
CaO	0.06	66.7	39.00
MgO	0.30	0.60	9.50
K_2O	1.85	0.01	0.50
TiO_2	0.02	-	1.30
Na_2O	0.10	0.20	0.30
SO_3	–	0.30	0.30
Mn_2O_3	–	-	0.70
LOI	12.00	28.79	1.90

Table 2. Properties of soil.

Property	**LL**	**PL**	**PI**	**SC (%)**	**CC**	**SG**	**MDD**	**OMC**	**USCS**
				(%)	**(%)**		**(kN/m^3)**	**(%)**	
Value	58	30	28	74	26	2.6	15	17	CL
Standard	ASTM D4318-17 (2017)			ASTM D422-63 (2007)		ASTM D854-10 (2010)	ASTM D1557-12e1 (2012)		

LL = liquid limit; PL = plastic limit; PI = plasticity limit; SG = specific gravity; CC = clay content; SC = silt content; MDD = maximum dry density; OMC = optimum moisture content; USCS = unified soil classification system.

2.2. Sulphate Soil

To be able to simulate the presence of sulphate compounds, the kaolinite was doped with different predetermined quantities of processed gypsum or hydrated calcium sulphate ($CaSO_4{\cdot}2H_2O$) powder. The percentage purity of this brand of gypsum is 99%.

2.3. Binding Agents

Lime was used in this study as the primary stabilising agent. Lime (4% by dry weight of soil) was used to stabilise the soil. This quantity of lime is in keeping with the range of the optimum amount required at least for the development of the correct pH environment and that can lead to the replacement of any available exchangeable cations of clays by calcium, as well as the possible precipitation of ettringites [26,27]. A fixed amount (8% calculated by dry weight of soil) of GGBS was used in the stabilised soil mix. This proportion of GGBS is within the range suggested to eliminate the deleterious effects of ettringite in the case of deformation or swelling especially when compared to the quantity of lime used [11]. The chemical compositions of these binding agents are shown on Table 1. Particle size distribution of the materials used are given in Figure 1.

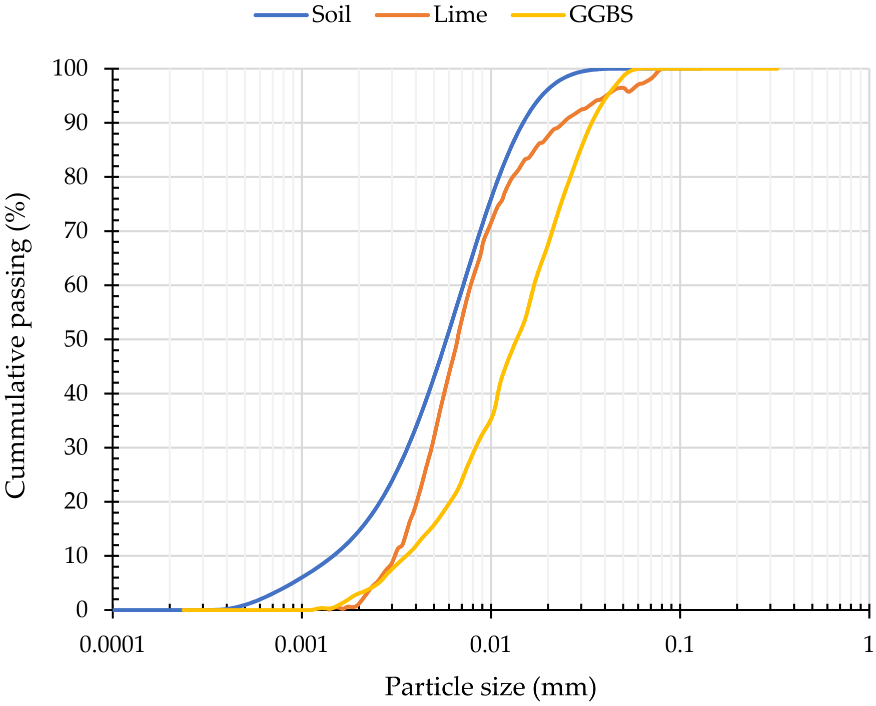


Figure 1. Particle size distribution of materials.

2.4. Material Preparation and Testing

2.4.1. Soil–Binder Preparation

To fulfil part of the aims of this study (simulating high-sulphate soils), the proportions of the gypsum used were 10%, 20%, and 30% calculated by dry weight of the soil (hereafter referred to as G10, G20, and G30). The quantities of the gypsum in the soil and binding agents used are summarised in Table 3. The materials were thoroughly mixed dry by hand for several minutes while also using a laboratory spatula to ensure that a proper homogeneous mix was achieved. Water was then added to the mix, the quantity of which was based on the optimum moisture (i.e., 20%) of the soil in its natural state as derived from the standard proctor compaction test (BS 1377-2:2022). It is also very necessary to state that the mixing was carried out on the "wet" side of the optimum moisture value with 2% of water added to ensure perfect or near-complete hydration due to the amount of the multiple binders used [24]. Further mixing was ensured for about 10 min with the water added to preclude the formation of "hotspots" and nonuniformity of migrated ions, while also increasing the chances of uniform distribution of ettringite nucleation sites [12]. Immediately after compaction of the mix at the optimum conditions, two each of the cylindrical samples (76 mm length and 38 mm diameter) were carefully extracted and wrapped securely using a cling film, sealed in Ziplock bags, and preserved in a curing tank for up to 28 days at a temperature of 20 ± 2 °C.

Table 3. Proportions of the materials (by dry weight of soil).

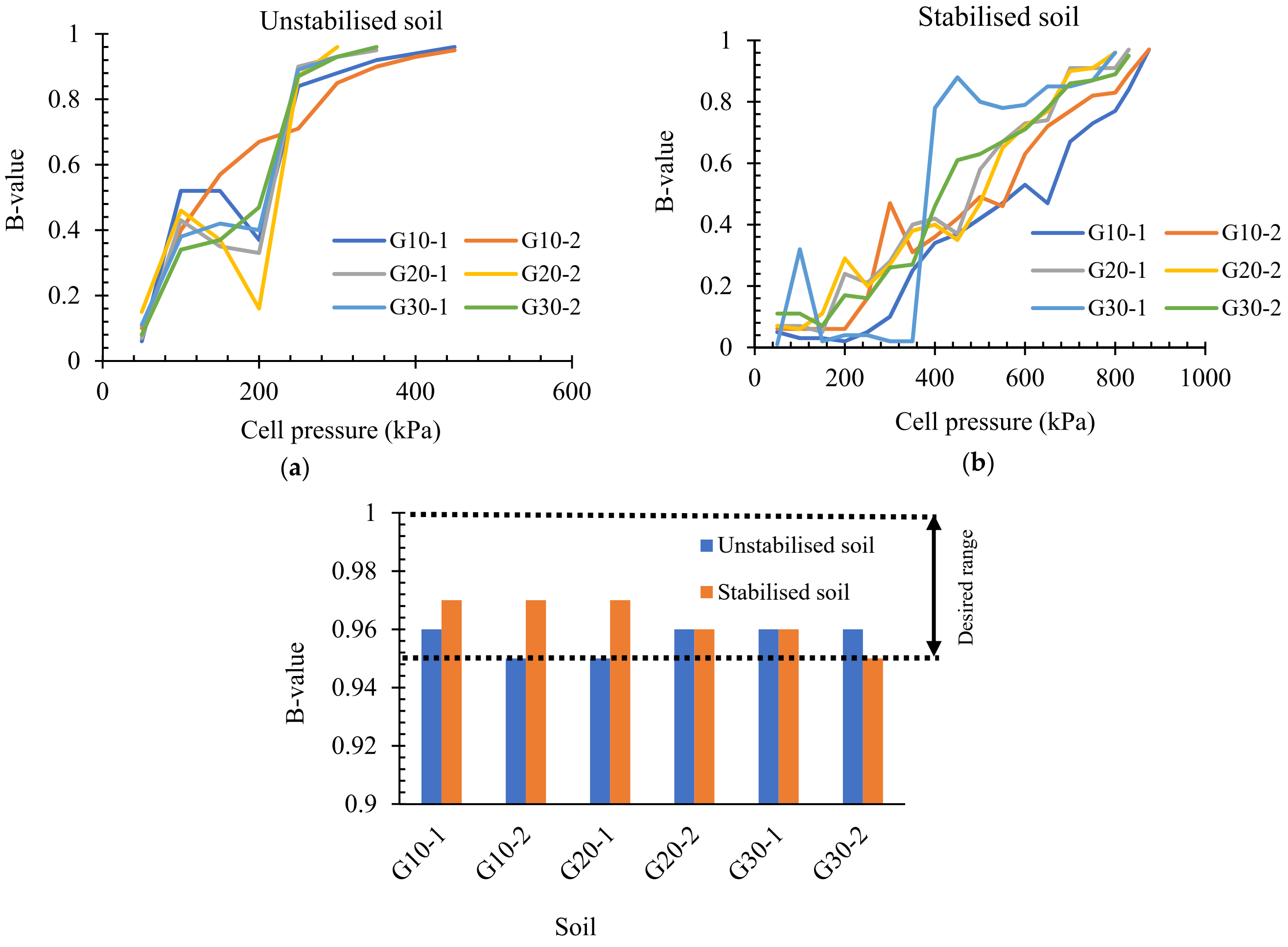
Gypsum (%)	**Lime (%)**	**GGBS (%)**
10	4	8
20	4	8
30	4	8

2.4.2. Consolidated (Isotropic) Undrained Triaxial Test (CIU)

Using a computer-controlled stress-path cell, consolidated (isotropic) undrained triaxial test (CIU) was performed in accordance with BS EN ISO 17892-9:2018 on the compacted and cured samples (natural and stabilised).

- Sample saturation

It should be noted that the compacted and cured samples were in a state of partial saturation. Hence, it is generally practical to use naturally saturated or soils which have been brought to a state of saturation before the commencement of testing. This is to prevent the effect of compressible pore fluid, which may distort the results of effective stresses [28]. The samples were subjected to values of effective pressures (i.e., cell pressure and back pressure) in multiple steps to ensure saturation in excess of 95%. A specimen can be considered as being saturated if the value of pore pressure coefficient B ($\Delta u_3/\Delta\sigma_3$) is at least 0.95 [29]. Figure 2a,b indicate the detailed plot of each "stepped" pressure inputs to achieve the desired saturation of the samples. The specimens were subjected to saturation with the value of B ranging between 0.95 and 0.98 (Figure 2c).



(**a**) (**b**)

(**c**)

Figure 2. Stepped input pressure: (**a**) unstabilised soil; (**b**) stabilised soil; (**c**) B-value range of unstabilised and stabilised soils.

- Sample consolidation and shearing

Following the saturation stage, two each of the triaxial specimens were subjected to constant effective consolidation pressures (i.e., cell pressure—back pressure) of 100 kPa and 200 kPa, representing values high enough to exceed any pre-existing preconsolidation pressures and the range commonly encountered in most earthwork applications [29,30]. Thereafter, shearing was conducted on the same constant effective consolidation pressures as in the consolidation stage.

2.4.3. Microstructural Examination

Microstructural analysis utilising the scanning electron micrograph (SEM) was performed to enable a description of the change mechanisms occurring in the untreated and stabilised soils' fabric. Small-sized chunks of the cured samples were utilised for the micrograph examination. Careful preparation of the samples for the SEM investigation are similar to those prescribed in [10]. The ZEISS EVO equipment was used with a minimum working distance (WD) of 7.7 mm, acceleration voltage (EHT) of 5.00 kV, and varying degrees of magnifications. Results obtained were also compared with those achieved in previous tests.

3. Results and Discussion

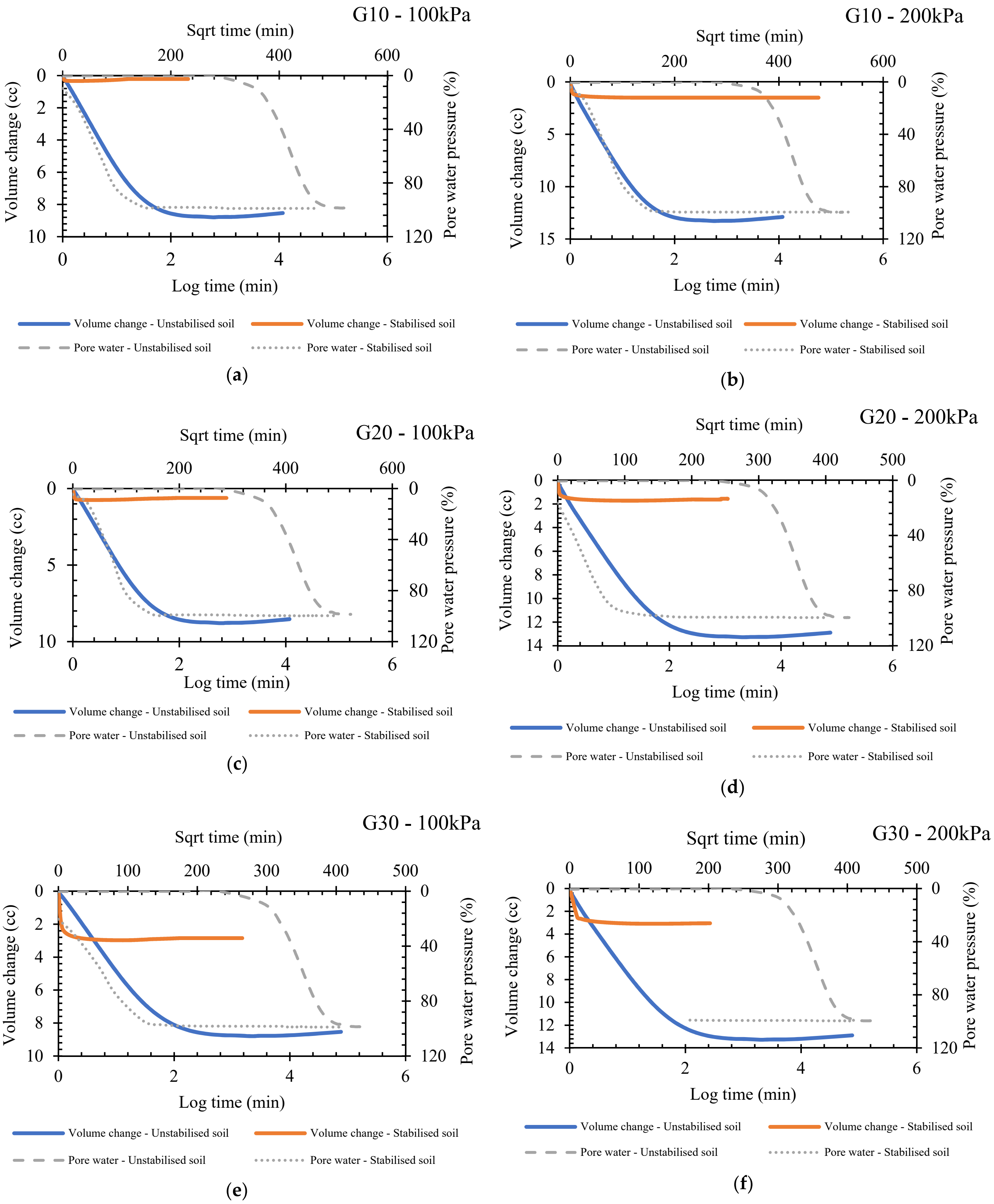
3.1. Isotropic Consolidation

Figure 3a–f indicate both the volumetric and pore pressure responses of the unstabilised (or non-stabilised) and stabilised soils with time during the consolidation stage. A decrease in volume as a result of the dissipation of excess pore water are observed. The volumetric reduction was greater for the unstabilised soils when compared to the stabilised soils. For the stabilised soil samples, the addition of calcium compound-rich agents clearly caused a hardening of the soils resulting in a decrease in the water content of these samples. As could be observed from the pore water pressure (pwp) curves, the time required for the expulsion of water from the non-stabilised soils was more than double those for the stabilised soils across all the samples. Figure 3a–f also generally indicate that consolidation under a higher pressure produced greater volumetric strain. Moreover, the volumetric strain curves were observed to have become asymptotic to the horizontal axis with the passage of time, thus indicating the end of primary consolidation under the given pressure.

In terms of the quantity of sulphates in the non-stabilised soils, there seems to be greater volume change observed for soils containing lower proportion of sulphates (Figure 3g,h). Although gypsum is a soluble salt with relatively less alkalinity when compared to other calcium oxide (or hydroxide) agents, they are still capable of binding particles of soils together [31,32]. It should also be noted that a higher solubility of gypsum can lead to greater deterioration of the bonding within the soil–gypsum mixture [33]. With regard to this study, the deformation of the soils was less with increasing sulphate content, as noted when the amount of gypsum was raised from 10–30% in the soil. Hence, it could be inferred that the surplus quantity of gypsum left without dissolution contributed to a reduction in the soil's volume.

On the other hand, it is interesting to also note that, for the stabilised soils, the amount of volume change seemingly increased with the quantity of sulphates in the soil (Figure 3i,j). As observed above, gypsum may have led to a reduction in volume by acting as a stabilising agent [34]. This phenomenon is mostly associated with short-term reaction and curing [35]. However, since gypsum is a lightweight material (relatively low specific gravity), any unreacted gypsum in the lime/GGBS-stabilised soils can result in a decrease in the overall specific weight, thus affecting the texture of the mix through an increment in pore sizes [36]. This can then lead to an increase in the volume of the stabilised soil. Although the quantity of GGBS required to prevent any possible attacks by sulphates should be twice the amount of lime in the mix [11], this may not have been sufficient to prevent volumetric increment as is the case in this study.

Overall, Figure 3g–j indicate that consolidation of the samples under a higher constant effective pressure did produce greater volumetric strain.

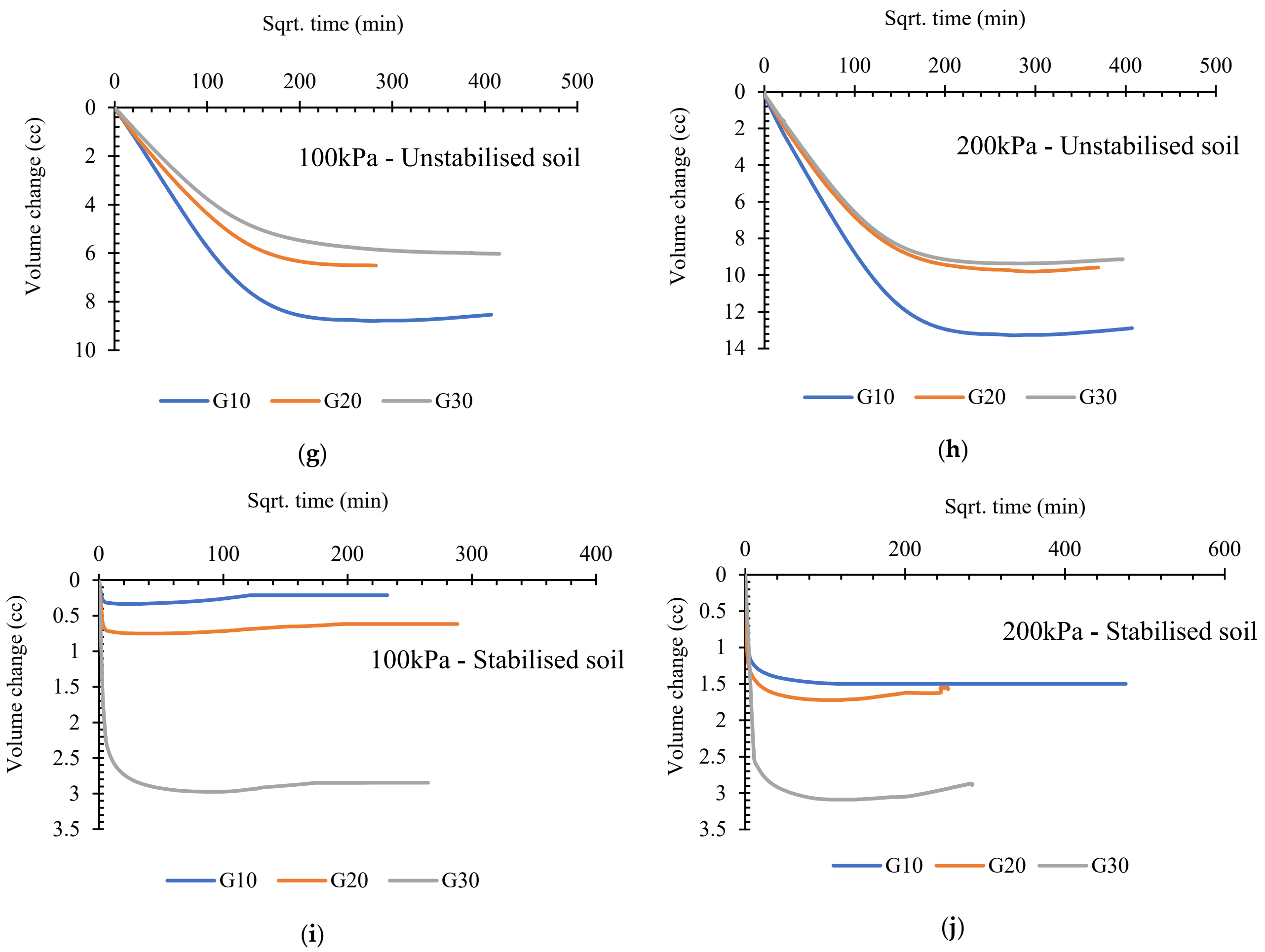


(**a**) (**b**)

(**c**) (**d**)

(**e**) (**f**)

Figure 3. *Cont.*



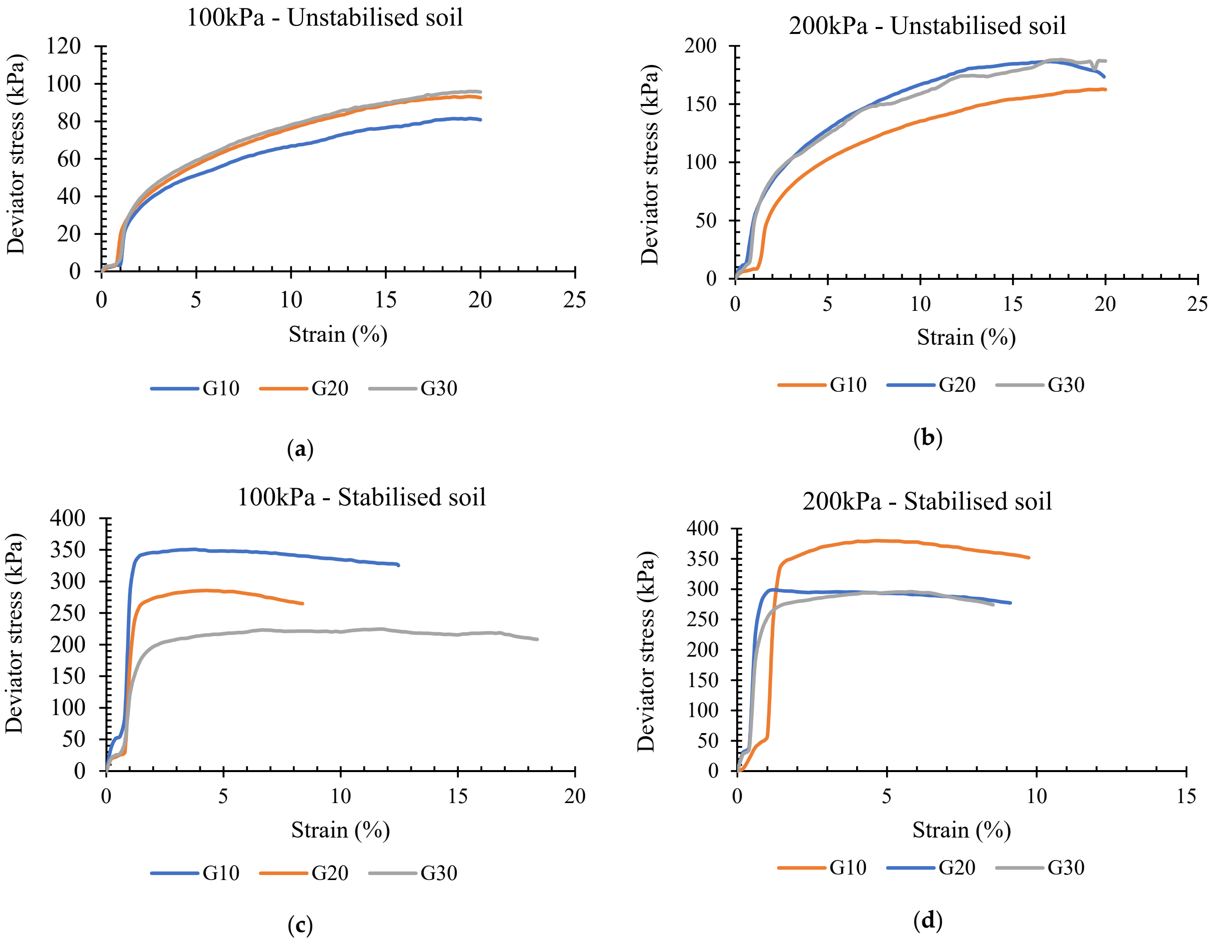
(**g**) (**h**)

(**i**) (**j**)

Figure 3. Volumetric and pore pressure responses: (**a**) 10% gypsum content and constant effective pressure of 100 kPa; (**b**) 10% gypsum content and constant effective pressure of 200 kPa; (**c**) 20% gypsum content and constant effective pressure of 100 kPa; (**d**) 20% gypsum content and constant effective pressure of 200 kPa; (**e**) 30% gypsum content and constant effective pressure of 100 kPa; (**f**) 30% gypsum content and constant effective pressure of 200 kPa; (**g**) consolidation of unstabilised soil under constant effective pressure of 100 kPa; (**h**) consolidation of unstabilised soil under constant effective pressure of 200 kPa; (**i**) consolidation of stabilised soil under constant effective pressure of 100 kPa; (**j**) consolidation of stabilised soil under constant effective pressure of 200 kPa.

3.2. Undrained Shearing

The behaviour of the non-stabilised and stabilised soils in the shearing stage is shown in Figure 4. For the non-stabilised soils, samples with higher quantity of the sulphates tended to fail at higher stresses compared to those having lower proportions of sulphates. Moreover, shearing carried out under greater constant effective pressure (i.e., cell pressure and back pressure) tended to fail at higher deviator stresses. The soils containing lower amounts of sulphates were, therefore, softer than those with higher proportions of the same. This behaviour of gypsum acting as a stabilising agent was apparent under the consolidation stage as previously observed. This would normally consist of a complex set of processes occurring concurrently but systematically and involving ionic exchanges in the initial stages in which case the addition of gypsum would increase the ion concentration in the soil as a result of hydrolysis, further leading to a reduction in the diffuse double layer thickness of the clay and a decrease in pore spaces. Subsequently, hydration would occur, contributing to increased cementation of the stabilised soil mix. In the process, crystallisation of gypsum would also ensue, enabling bonding, wrapping, and filling of the pore spaces resulting in a denser microstructure and, hence, increased cohesion.



(**a**) (**b**)

(**c**) (**d**)

Figure 4. Stress–strain curves: (**a**) unstabilised soil under 100 kPa constant effective pressure; (**b**) unstabilised soil under 200 kPa constant effective pressure; (**c**) stabilised soil under 100 kPa constant effective pressure; (**d**) stabilised soil under 200 kPa constant effective pressure.

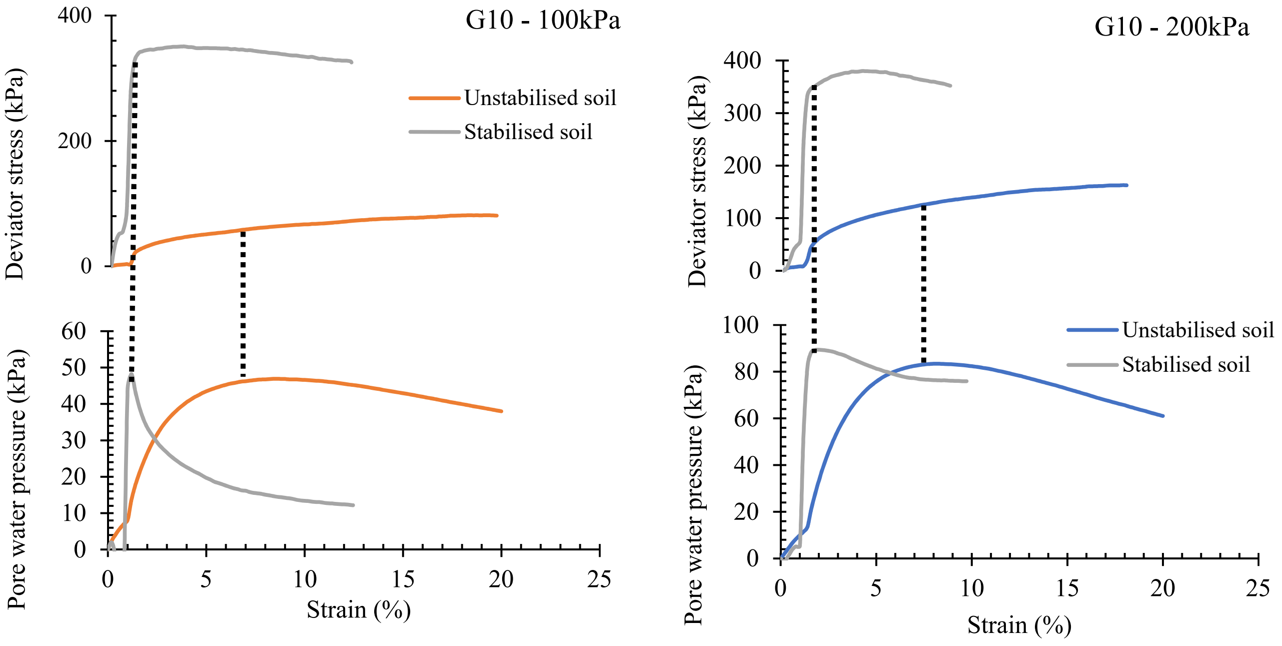
Generally, the unstabilised soils are characterised by ductile behaviour with contraction and then yielding observed at approximately 40 kPa (constant effective pressure of 100 kPa) and 75 kPa (constant effective pressure of 200 kPa), followed by strain hardening (Figure 4a,b). Furthermore, it is clear that the behaviour of the unstabilised soils with 20% and 30% gypsum were almost indistinguishable under the two constant effective pressures, thus indicating the lower effect of sulphate content beyond 20%. Beyond the yielding point, the percentage increase in stress between samples having 10% and 20–30% of gypsum was approximately 20% under the two constant pressures.

On the other hand, the stabilised soils tended to behave differently with characteristics almost opposite to those of the non-stabilised ones. As Figure 4c,d indicate, the stabilised soils were initially ductile with the yielding followed by an almost perfectly plastic behaviour up to about 6–8% of the strain before finally undergoing a small amount of strain-softening. However, it is interesting to note that the stabilised soils with a lower percentage of the sulphate tended to produce a higher level of plastic failure compared to those possessing higher amounts of the sulphates. Hence, beyond the yielding point and at the constant effective pressure of 100 kPa, failure tended to occur at stresses of about 350 kPa, 290 kPa, and 210 kPa for soils having 10%, 20%, and 30% of the gypsum content, respectively. On the other hand, the failure rates were higher under a higher constant effective pressure although the behaviour of stabilised soils containing 20% and 30% of the gypsum were almost the same. In this regard, plastic failure occurred at approximately 385 kPa and 300 kPa for the stabilised soil containing 10% gypsum and 20% (or 30%) gypsum, respectively.

In terms of the mechanism of change and as suggested above, even though the quantity of GGBS required to prevent any potential negative effects of sulphates should be approximately twice or even much more than the amount of lime in the mix [11], this may not have been sufficient to prevent the decrease in the strength behaviour of the soils having very high gypsum content. The process involving the dissolution of gypsum can cause a reduction in cementation capability by aiding sulphate ion coating [37]. Moreover, the presence of unreacted gypsum and the development of ettringite minerals can disturb the soil matrix, leading to larger voids and a subsequent strength reduction at 28 days of curing [38].

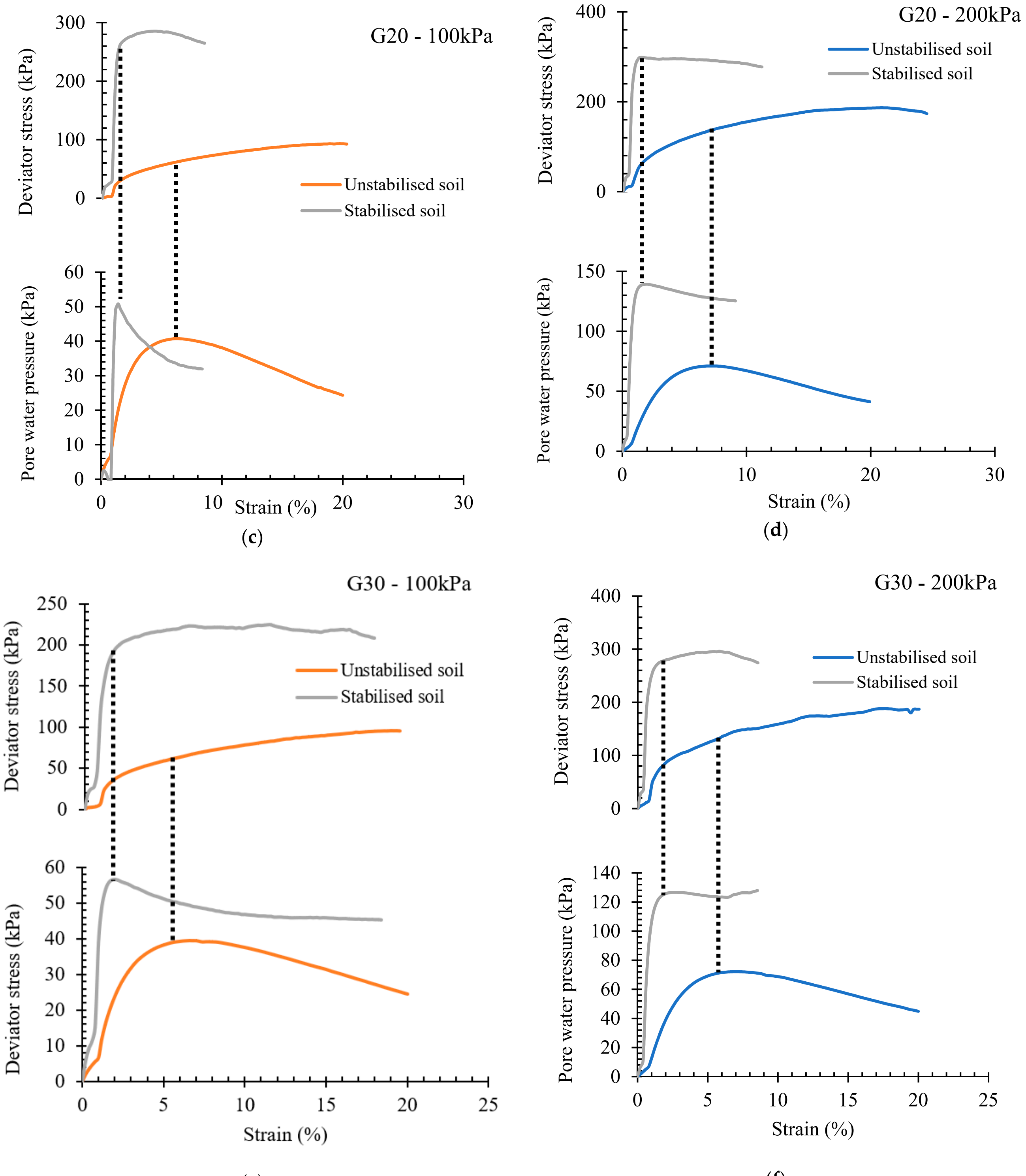
3.3. Non-Stabilised vs. Stabilised Soils

Figure 5 is a comparison of the behaviour of unstabilised and stabilised soils along with the pore water pressure changes occurring at the critical yielding points of the stress–strain curves. The failure stresses experienced by the stabilised soils were generally greater than those of the non-stabilised soils. This was mostly due to the hardening of the soils, ultimately resulting in pozzolanic activities from treatment with the calcium-based agents. For soils containing 10% gypsum, the approximate increment in failure stress when the soils were stabilised was much higher (450%) under a constant effective pressure of 100 kPa as compared to the stabilised soils (140%) when the constant effective pressure was 200 kPa (Figure 5a,b). For soils containing 20% gypsum, the increase in stress at failure when stabilised with the additives (lime and GGBS) was about 200% and 70% under constant effective pressures of 100 kPa and 200 kPa, respectively (Figure 5c,d). When the amount of gypsum was raised to 30% in the soil, treatment with the additives caused a rise in the failure by about 120% and 70% under the constant effective pressure of 100 kPa and 200 kPa, respectively (Figure 5e,f). Thus, it is clear that the increase in cell pressure did not seem to necessarily cause a remarkable increment in the failure stress of the soils when stabilised with the binders.



(**a**) (**b**)

Figure 5. *Cont.*



(**c**) (**d**)

(**e**) (**f**)

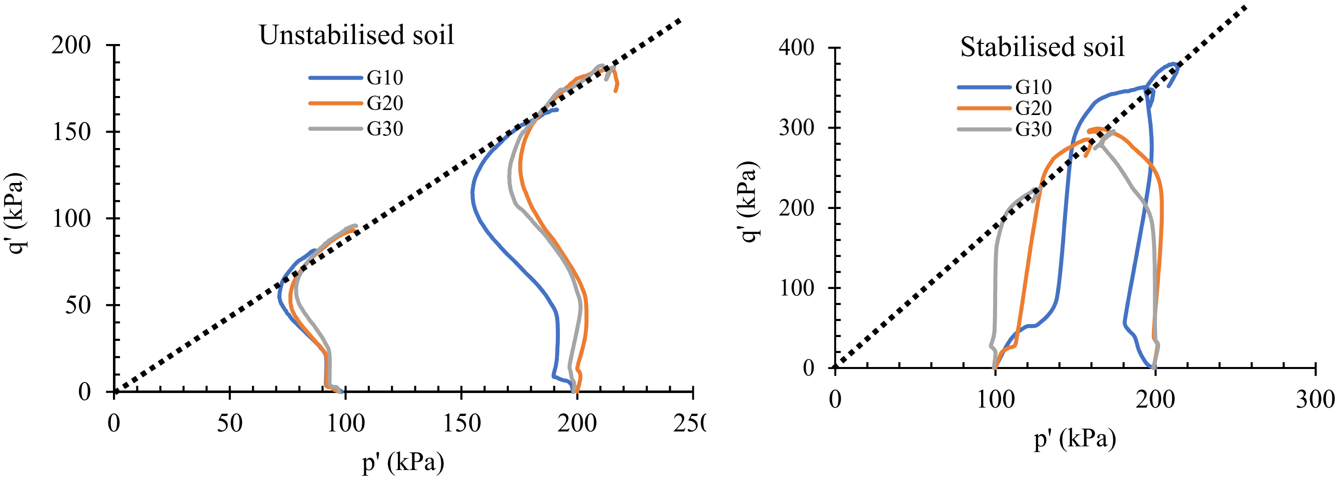
Figure 5. Stress–strain curves with pore pressure changes: (**a**) 10% gypsum content and constant effective pressure of 100 kPa; (**b**) 10% gypsum content and constant effective pressure of 200 kPa; (**c**) 20% gypsum content and constant effective pressure of 100 kPa; (**d**) 20% gypsum content and constant effective pressure of 200 kPa; (**e**) 30% gypsum content and constant effective pressure of 100 kPa; (**f**) 30% gypsum content and constant effective pressure of 200 kPa.

It is important to note that, for normally consolidated (or lightly over-consolidated) clays, axial stress would be expected to reach an ultimate value at a relatively large strain and is usually accompanied by a rise in pore water pressure to a steady value. Meanwhile, for heavily over-consolidated clays, the axial stress would tend to increase to a peak value and then decrease with subsequent increase in strain. In this case, the pore water pressure would increase initially and then decrease [29].

An examination of the pore water pressure curves indicates that the non-stabilised soil containing 10% gypsum and subjected to constant effective pressures of 100 kPa and 200 kPa was lightly over-consolidated. However, when stabilised with the additives, heavy over-consolidation tended to occur. Moreover, as could be observed, the stabilised soils were seemingly more over-consolidated under the lower cell pressure. Moreover, shearing carried out under the lower cell pressure showed that the stabilised soil possessing 10% gypsum was more over-consolidated when compared to those containing higher amounts of gypsum which perhaps explains why the percentage increase in the failure stress of the former was the highest (i.e., about four times that of the non-stabilised soil). In the same vein, as indicated by the stress–strain and the corresponding pore pressure curves, the stabilised soils containing 20% and 30% gypsum tended to be mostly normally consolidated, especially when shearing was carried out under the higher cell pressure compared to the those having 10% of gypsum, leading to the lower volumetric strain and higher failure stress observed overall.

3.4. Stress Path

Figure 6 shows the stress path in the *p*–*q* space for the non-stabilised and stabilised soils. The behaviour of the soils as described above was mostly confirmed from these stress-path diagrams. The non-stabilised sulphate soils all tended to be lightly over-consolidated with more dilation of the sulphate soils containing higher percentages of gypsum (Figure 6a). Meanwhile, mixed behaviours of the stabilised samples were observed as indicated by the stress paths of Figure 6b. A detailed examination of Figure 6d,e indicates that the stabilised soils possessing higher quantities (20% and 30%) of gypsum tended to be normally consolidated when in the shearing stage are subjected to higher constant effective pressures. On the other hand, the stabilised sulphate soil with 10% gypsum tended to be over-consolidated (Figure 6c). In general, it can also be appreciated from the stress-path diagrams that the attainment of a higher failure stress (upper failure envelope or CSL) occurred with stabilisation of the sulphate soils.



(**a**) (**b**)

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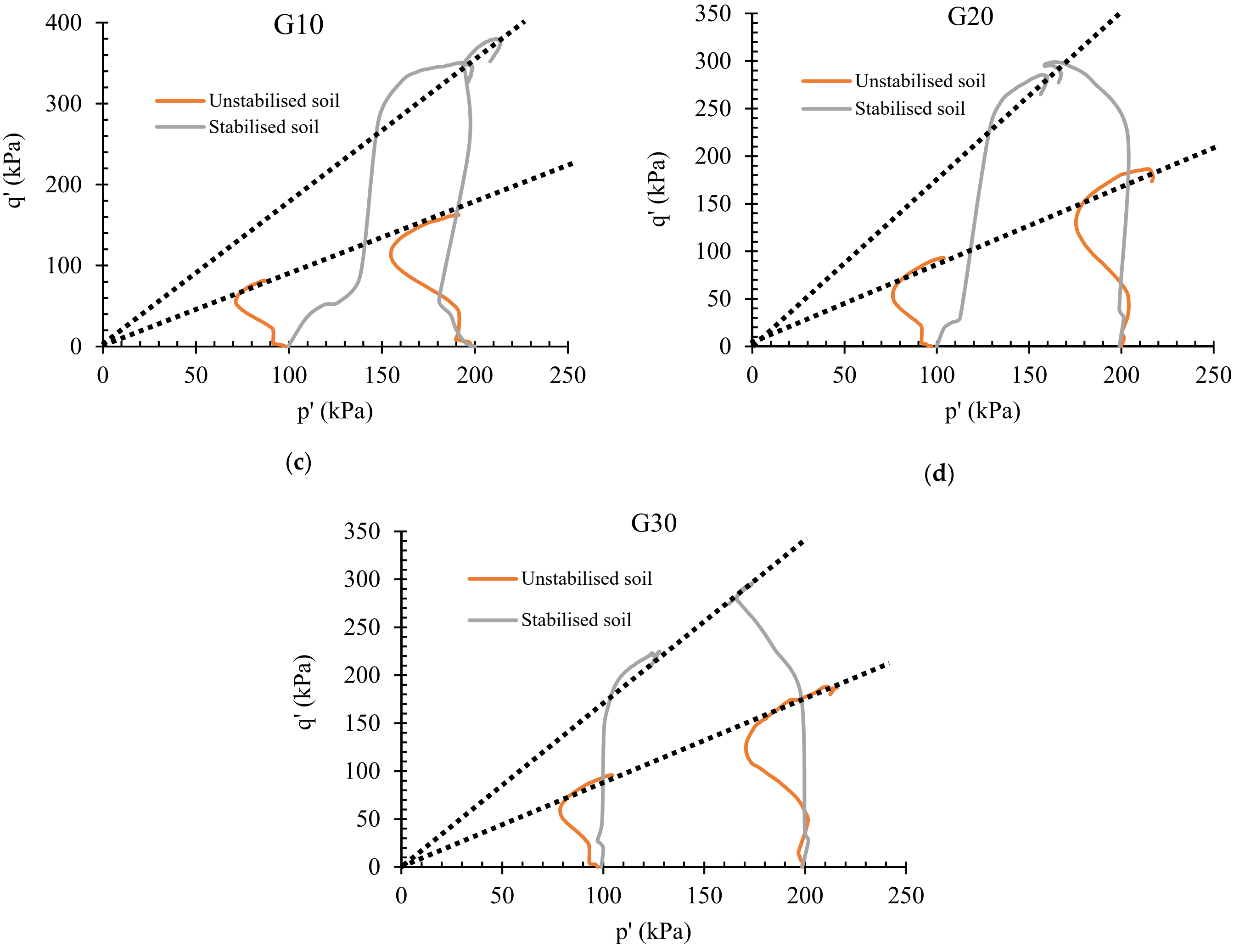


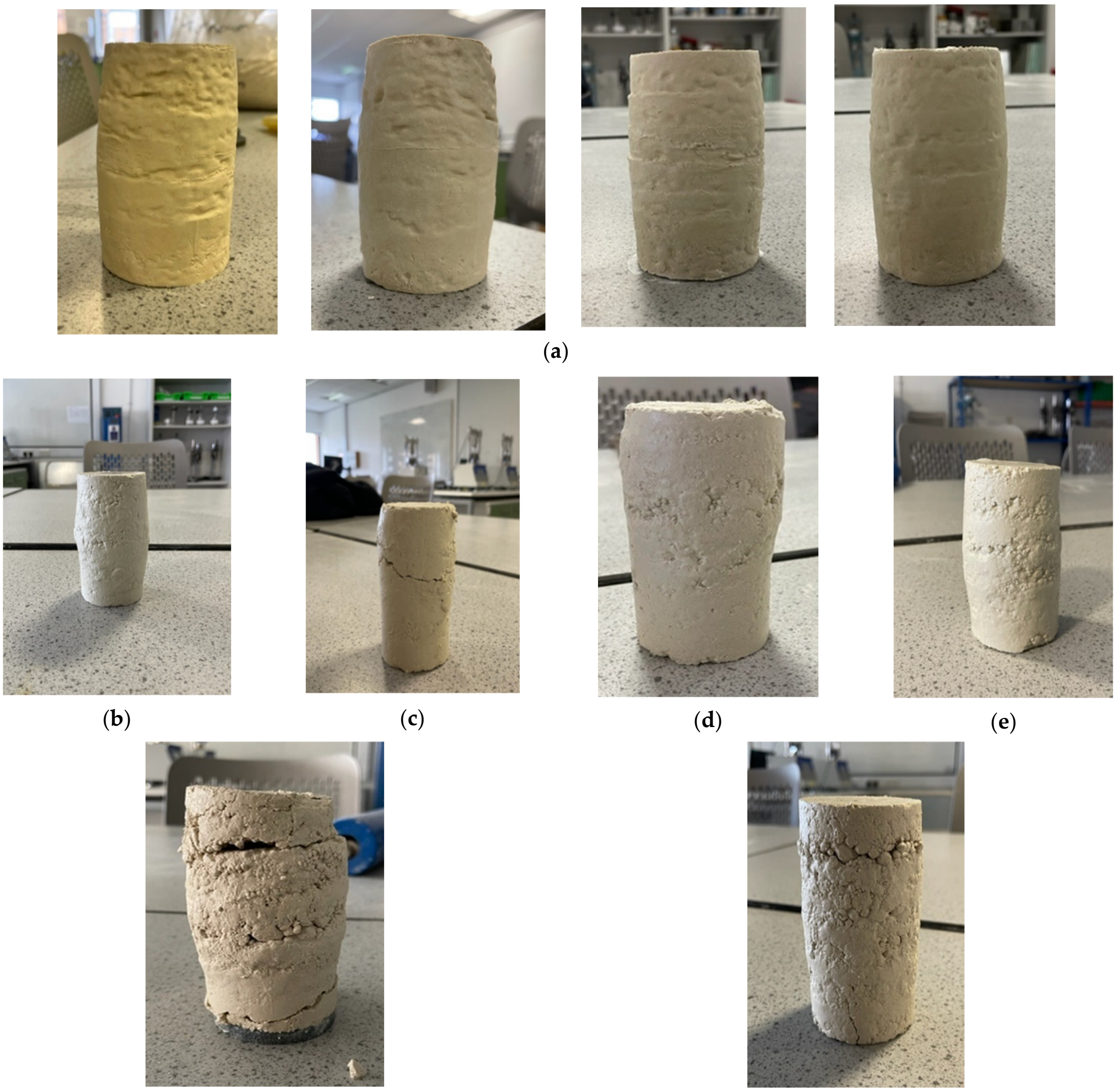
(**c**)

(**d**)

(**e**)

Figure 6. Stress path in *p*–*q* space: (**a**) unstabilised soils; (**b**) stabilised soils; (**c**) 10% gypsum content for unstabilised and stabilised soils; (**d**) 20% gypsum content for unstabilised and stabilised soils; (**e**) 30% gypsum content for unstabilised and stabilised soils.

Figure 7 indicates the behaviour of the samples at failure. The pattern of failure for the unstabilised soils was almost similar (with failure indicated as a barrelling of the specimens) from close observations irrespective of the quantity of gypsum content and the level of cell pressure under which they were subjected. Hence, these are represented by few general illustrations for brevity (Figure 7a) These illustrations confirm the general ductile behaviour of the unstabilised soils with subsequent yielding or failure occurring at relatively larger strains. However, for the stabilised soils, shearing or failure indicated as cracking or faults across most of the cylindrical samples was clearly noticeable. As already observed from the stress–strain curves shown previously, the stabilised soils were initially ductile (under very small strains) with the yielding point followed by an almost perfectly plastic behaviour up to about 6–8% of the strain before finally undergoing small amounts of strain softening. Stabilised soils with a higher content of gypsum (particularly 30% gypsum) seemed to be the most brittle and, hence, possessed the lowest yielding or failure stress under the two cell pressures, as noticed previously.



(**a**)

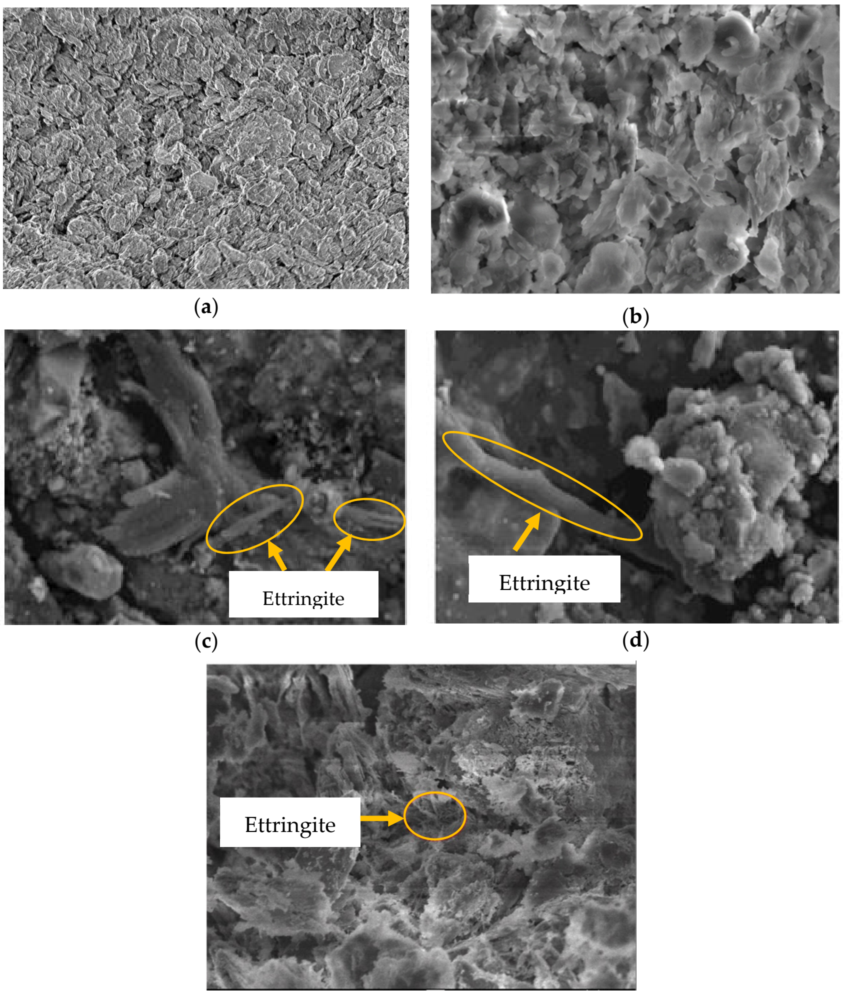
(**b**) (**c**) (**d**) (**e**)

(**f**) (**g**)

Figure 7. Behaviour of samples at failure: (**a**) unstabilised soils; (**b**) stabilised soils with 10% gypsum under 100 kPa constant effective pressure; (**c**) stabilised soils with 10% gypsum under 200 kPa constant effective pressure; (**d**) stabilised soils with 20% gypsum under 100 kPa constant effective pressure; (**e**) stabilised soils with 20% gypsum under 200 kPa constant effective pressure; (**f**) stabilised soils with 30% gypsum under 100 kPa constant effective pressure; (**g**) stabilised soils with 30% gypsum under 200 kPa constant effective pressure.

3.5. Microstructual Analysis

The compacted "pure" kaolinite seems to possess a leaf-like structure as indicated by Figure 8a. A complex set of reactions could occur when kaolin clay comes in contact with water, one of which can involve the formation of an extensive absorbed film resulting from the existence of a concentration gradient composed of water molecules and exchangeable ions. When gypsum is added to kaolinite, depending on the quantity and curing duration, this may lead to a reduction in volume or increased strength, thus acting as a stabilising agent. Soluble gypsum does contain calcium hydroxide, and the process of ionic exchanges (Ca^{2+} from gypsum) with the soil (Al^{3+} and Si^{+}) and the development of the products of hydration (such as calcium silicate hydrate (CSH) or calcium alumino-silicate hydrate (CASH)) would lead to increased strength or volume reduction. Figure 8b, therefore, indicates a well-connected soil matrix with fewer pores made possible by wrapping of the fabric by gypsum with formation of polymer clusters. When stabilised by lime, the possibility of ettringite formation was increased (Figure 8c,d). Although they could be deleterious in some respects especially in terms of soil deformation; it should also be noted that ettringite formation is a necessary process during the hydration process for an increase in strength. As indicated previously, for a gypsum–lime–GGBS–soil system, complex sets of reactions occurring at the same time are feasible. GGBS hydration with lime as a catalyst or an activator in the presence of gypsum can lead to the formation of C–S–H and ettringite products. Furthermore, soil–lime reactions in the presence of gypsum would cause the formation of C–A–S–H and ettringite products including the development of nucleation sites in the soil and GGBS surfaces. GGBS present in greater quantities compared to lime would tend to have more influence and, thus, contribute more to the formation of cementitious gels required for hydration to enhance a reduction in ettringite formation. The lime–GGBS ratio needed for this to occur should be approximately 2 or more [11]. However, because of the higher amounts of sulphates present in the stabilised soil as is the case in this study, the amount of GGBS used (8% by dry weight of soil) compared to that of lime (4% by dry weight of the soil) may not have been sufficient to prevent any potential negative effects of sulphates, resulting in the traces of ettringites suggested in Figure 8e. As also noted previously, the presence of unreacted gypsum and the development of ettringite minerals may disturb the soil matrix, leading to larger voids and subsequent strength reduction at 28 days of curing [38,39].



(**a**) (**b**) (**c**) (**d**) (**e**)

Figure 8. Scanning electron micrograph: (**a**) Pure soil (kaolinite); (**b**) sulphate soil; (**c**) lime-stabilised sulphate soil [36]; (**d**) lime-stabilised sulphate soil [36] (**e**) lime/GGBS-stabilised sulphate soil [15].

4. Conclusions

This article investigated the consolidated undrained shear strength behaviour of a stabilised high-sulphate soil system consisting of gypsum using lime and incorporation of GGBS. The main highlights and conclusions from this research are as follows:

- In terms of the quantity of sulphates in the non-stabilised soils, there was seemingly a greater volume change observed for soils containing a lower proportion of sulphates. This indicates that calcium sulphate which is a soluble salt with relatively less alkalinity is capable of binding particles of soils together.
- The volume change increased with the quantity of sulphates in the stabilised soil even though the quantity of GGBS utilised as an ettringite suppressor was twice more than that of lime.
- Generally, the stabilised sulphate soils showed some initial ductile response with the yielding followed by an almost perfectly plastic behaviour up to about 6–8% of the strain before finally undergoing small amounts of strain softening.
- Higher levels of plastic failure were achieved at a higher constant effective stress for the stabilised soils containing lower percentages of sulphates compared to those possessing higher amounts of the sulphates.

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