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The behaviour of an artificially cemented sandy gravel

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Abstract. The major section of the city of Tehran, Iran has been developed on cemented coarse-grained alluvium. This deposit consists of gravely sand to sandy gravel with some cobbles and is dominantly cemented by carbonaceous materials. In order to understand the mechanical behaviour of this soil, a series of undrained triaxial compression tests and unconfined compression tests were performed on uncemented and artificially cemented samples. Portland cement type I was used as the cementation agent for preparing artificially cemented samples. Uncemented samples and lightly cemented samples (1.5% cement) tested at high confining pressure showed contractive behaviour accompanied with positive excess pore water pressure and a barrelling failure mode. However, cemented samples and uncemented samples tested at low confining pressure (25 and 50 kN/m^2) showed dilative behaviour accompanied with negative excess pore water pressure. Shear zones were formed in these samples and a clear peak in excess pore water pressure and stress ratio against strain could be observed. Test results showed that the limiting stress ratio surface for cemented samples is curved and expands as the cementation and density increase. Unconfined compression strength of cemented samples increases with increases in cementation and density as well.

Key words. artificial cement, cemented soil, limiting stress ratio, pore water pressure, sandy gravel, shear strength, stress path.

1. Introduction

Geotechnical engineers often deal with cemented and bonded materials. Stress–strain, stiffness, bearing capacity and mechanical parameters of cemented soils are affected by the amount and type of cementation. The effect of cementation is especially important at low confining pressures where effective cohesion has an important role on the stability of near surface earth structures such as slopes.

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Cemented coarse-grain soils (sand, gravely sand and sandy gravel) are present in many parts of the world. Steep natural slopes and nearly vertical cuts may remain stable in such soils. However, sudden instability may occur when the bonds are broken. Cementation in naturally cemented soils can be attributed to several sources as follows:

(i) Existence of agents such as silica, hydrous silicates, hydrous iron oxides and carbonate deposits between grains,
(ii) Cold welding between the soil grains,
(iii) Presence of a matrix of silts and clays between sand and gravel particles.

Experimental research on naturally cemented soils is rare because (i) acquiring undisturbed samples from naturally cemented course grained soils is extremely difficult, (ii) naturally cemented soils are heterogeneous and the degree of cementation and the density is variable. Therefore, a major part of experimental studies on the behaviour of cemented soils has been on artificially cemented samples.

In the last two decades, a number of investigators have studied stress–strain and strength characteristics of cemented soils using triaxial, direct shear, unconfined compression, Brazilian, resonant column and plane strain tests. Most of these experimental works were carried out using artificially cemented samples and some of them are reviewed in the following:

Clough et al. (1981) reported a set of triaxial, Brazilian and unconfined compression tests to examine the effects of cementation and density on the behaviour of naturally and artificially cemented sands. Test results showed that stiffness and peak strength increased with increases in density and the amount of cementing agent. They suggested that in addition to cementation and density, grain size distribution and grain size arrangement (fabric) have an important role on the behaviour of cemented sands.

Leroueil and Vaughan (1990) reviewed studies that had been carried out by others on cemented and structured soils. They suggested that structure in soils such as stiff over consolidated clays, clay-shales, cemented sands, residual soils, artificially bonded soils and weak rocks may arise from many different sources but they have similar behaviour. They have shown within the paper that the yield in structured soils can occur under compression, shear and swelling stresses.

Airey (1993) carried out conventional and stress path triaxial tests to examine the behaviour of natural calcarenite soils. He used tension testing for estimating of the degree of cementation of naturally cemented samples. He suggested that the cementation increases the shear modulus and the size of the yield locus; however, its effects on the volumetric response are negligible.

Coop and Atkinson (1993) carried out tests on artificially cemented carbonate sands. Test results showed that the direction of the stress path, the drainage conditions and the confining pressure affect the peak strength. They showed that the stress–strain behaviour of cemented samples depends on the position of the initial state of the stress of the soil with respect to the yield locus of the bonding.

Toll and Malandraki (1993) put forward a framework based on a similar approach (identifying zones of behaviour relating the stress to the bond yield surface and failure envelope) and Malandraki and Toll (2000) extended this to look at the behaviour of an artificially weakly bonded soil under range of different stress path directions. Malandraki and Toll (2001) showed that bond yield is an anisotropic process and that different yield points could be seen if the stress path direction was changed, even if the sample had previously yielded following another stress path direction. They suggested that the bond yield surface can expand and shrink but it does not move.

Das et al. (1995) carried out Brazilian and unconfined compression tests on the artificially cemented sands. They showed that tensile and unconfined compression strength increase with increase in cement content while tensile and compressive strains at failure decrease.

Ismail et al. (2002) studied the effect of cement type on the shear behaviour of a cemented calcareous sand. They used three different types of cement agent (Portland cement, gypsum and calcite) for preparing artificially cemented samples. They used unconfined compression strength as milestone for comparing the three types of cementation. Triaxial tests on samples of different cementation type with similar unconfined compression strength and density, showed different effective stress paths and post yield responses. The Portland cemented samples showed ductile behaviour while calcite and gypsum cemented samples exhibited brittle yield.

Haeri et al. (2002) carried out some large direct shear tests on uncemented and artificially cemented gravely sands using lime as a cementing agent. The tested base soil was obtained from the Tehran alluvium. The results showed that cementation increased the cohesion intercept and that the failure envelope for cemented samples was curved.

Asghari et al. (2003) performed a series of triaxial tests on uncemented and artificially cemented samples using hydrated lime as a cementing agent. Tests results show that shear strength increases with increasing cement content but the influence of the cementation decreases as the confining pressure increases. They also indicated that the failure envelope for cemented samples is curved and not linear.

Saxena and Lastrico (1978), Dupas and Pecker (1979), Acar and El-Tahir (1986), Lade and Overton (1989), Krisek et al. (1992), Airey (1993), Gens and Nova (1993), Cuccovilo and Coop (1997), Schnaid et al. (2001) have also contributions on the study of the behaviour of cemented soils.

The mechanical behaviour of cemented alluvium of Tehran, Iran has been studied and is reported in the present paper. Tehran alluvium is highly variable in grading, density and degree of cementation. Therefore it is extremely difficult to obtain undisturbed representative samples. The cemented coarse-grained alluvium of Tehran often consists of gravels, sandy gravels to gravely sands with some cobbles, silts and clays. However, the present work has been carried out on North Tehran alluvium where the grading of the deposit is mainly sandy gravel. This research is in the same line as the study reported by Asghari et al. (2003), however, the extent of the present work is much wider and the type of cementation is different. In this research, the effects of cementation and density on the behaviour of Tehran alluvium has been extensively studied by using artificially cemented samples. Portland cement type I was used as the cementing agent in preparing the artificially cemented samples.

2. Experimental Program

Studying the mechanical behaviour of naturally cemented soils is difficult because it is extremely difficult to obtain undisturbed samples and prepare test specimen for triaxial testing. Moreover, the cemented coarse-grained alluvium of Tehran is heterogeneous both in grading and cementation. Therefore the mechanical characteristics of this cemented alluvium were investigated using artificially cemented samples. Portland cement type I was used as a cementing agent in this research. Grain size distribution, dry density and cement content are the main controlling factors considered and tested in this research. The samples were prepared with constant grain size distribution of the base soil, at two densities and five different cement contents. Consolidated undrained triaxial tests were performed at different confining pressures including 25, 50, 100, 300 and 400, 450, or 500 KN/m^2.

2.1. TEST MATERIAL

Tehran alluvium is highly variable in gradation and cementation. The range of cementation in this deposit varies from completely uncemented to strongly cemented. The size of the material present in the Tehran deposit ranges from boulders and gravels near the mountains in the north of Tehran to clay and silt size in the south of Tehran where the land is fairly flat. This research was performed on the alluvium from a region in Tehran where the gradation of the soil is in the range of sandy gravel with some cobbles. Regarding the limitation on the maximum particle size for triaxial testing of 100 mm diameter samples the average grain size distribution of the tested soil, which is called the base soil, is shown in Figure 1. The grain size distribution of the base soil was maintained constant and was selected as representative of the Tehran cemented alluvium. As shown in Figure 1 the base soil is sandy gravel according to BS standard 5930 and consists of 6% fines, 29% sand and 65% gravel (SW-SM in the Unified Soil Classification System). The properties of the base soil are given in Table 1.

2.2. TEST PROGRAM AND PROCEDURE

The test program was planned so that cementation effects on the behaviour of the sandy gravel material could be understood. The test program involved consolidated undrained triaxial tests on uncemented and artificially cemented samples of the base soil. Unconfined compression tests were also carried out on the artificially cemented samples. A standard triaxial apparatus was used for testing sandy gravel samples having 100 mm diameter and 200 mm height. Portland cement in amounts of 1.5%, 3%, 4.5%, 6% and 9% of the weight of dry soil was used to prepare artificially cemented samples. The samples were prepared at two dry unit weights of 18 kN/m^3

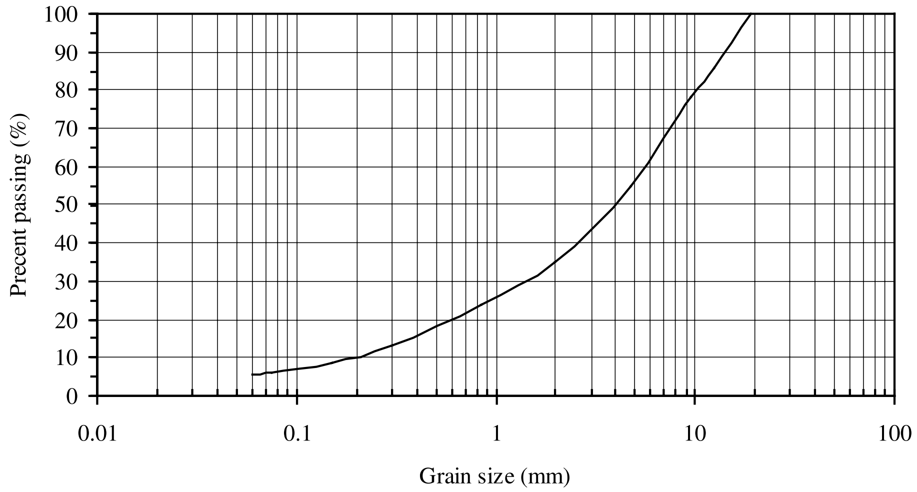
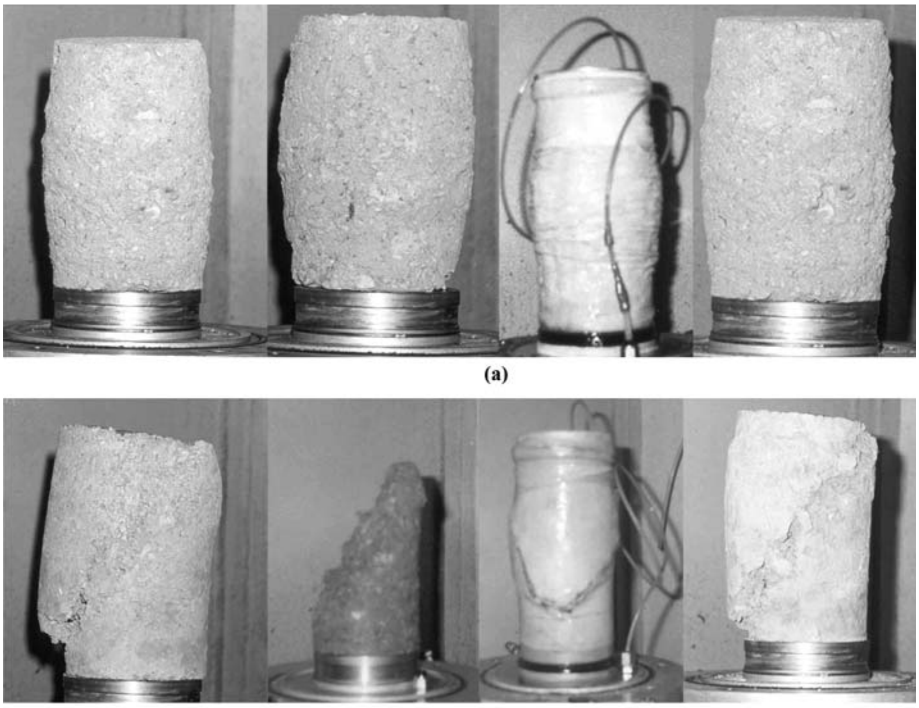


Figure 1 Grain size distribution of the sandy gravel

Table 1 Physical properties of the sandy gravel

Effective grain size *D*10	Medium grain size *D*50	Coefficient of uniformity *C*u	Coefficient of curvature *C*c	Specific gravity of solids, *G*s	Maximum void ratio, e_{max}	Minimum void ratio, e_{min}
0.2 mm	4 mm	28	1.8	2.6	0.63	0.34

(relative density of 70%) and 17.5 kN/m^3 (relative density of 50%). Two relative densities 70% and 50% are, respectively, corresponding to the dense and medium dense states of the tested material. Samples were tamped into a mold in eight equal layers to achieve the desired unit weight. The base soil forming each layer was first dried and weighed, then the appropriate amount of cement was introduced to the mixture and finally 9% of water was added. This water content is the average optimum water content of the compacted base soil obtained from Proctor compaction tests. Triaxial tests were conducted on the uncemented samples immediately after sample preparation. However, artificially cemented samples were placed in a humid room for curing after preparation of the sample and before triaxial testing. The samples then were tested after 28 days, which is the curing time for Portland cement. The triaxial tests were performed in consolidated undrained conditions at five different confining pressures between 25 kN/m^2 and 500 kN/m^2. The samples were saturated by flushing with carbon dioxide gas before flushing with de-aired water. The process of saturation was followed by increasing the back pressure until a *B* value greater than 0.95 was obtained. During this process an effective stress of approximately 20 kN/m^2 was maintained on the sample. *B* values greater than 0.95 could not be achieved in some samples with high cement content because of the high stiffness of the samples. Increasing the back pressure did not show any changes in *B* value in such samples, therefore it was concluded that the samples were fully



(b)

Figure 2 Prevailing failure modes of samples: (a) barrelling in uncemented samples, (b) shear zone in cemented samples

saturated. After saturation, the samples were isotropically consolidated and were sheared under strain control at a strain rate of 0.15% per minute.

3. Test Results and Discussions

A total of 103 consolidated undrained triaxial tests on uncemented and artificially cemented sandy gravel have been carried out. Test results are presented as including stress–strain curves, stress paths, pore pressure development and failure envelopes and cementation effects are discussed. The test results from this research have been compared with the results of a parallel study, which used a similar base soil but with a different cementing agent (Asghari et al., 2003). The effect of cementation type is also discussed in this paper.

The test results have been analysed using σ'_1, σ'_3, ε_a, ε_v, Δu, E, q and p', where σ'_1 and σ'_3 are, respectively, the axial and confining effective stresses on a cylindrical sample, ε_a and ε_v are respectively the axial and volumetric strains, Δu is excess pore water pressure, E is the initial stiffness, q and p' are respectively deviatoric and mean effective stresses. q and p' are defined as:

$$q = \sigma'_1 - \sigma'_3, \quad p' = (\sigma'_1 + 2\sigma'_3)/3.$$

3.1. FAILURE MODE

Figure. 2 shows a picture of the typical failure modes of uncemented and cemented samples. Uncemented samples, which were tested at confining pressures further than 100 kN/m^2 showed barrelling failure modes during shear. However, shear zones formed in samples tested at the lowest confining pressure i.e. 25 kN/m^2. Uncemented samples at intermediate confining pressures (50 and 100 kN/m^2) and relative density of 70% showed barrelling failure mode at low strain levels of up to 10% and then shear zones were formed after 10% strain.

Lightly cemented samples (1.5% cement) showed a brittle failure mode accompanied by shear zones at low confining pressures. The same samples tested at high confining pressures showed a barrelling failure mode at small strain followed by shear zones at strains greater than about 8%. Other cemented samples (3%, 4.5%, 6% and 9% cement) showed a brittle failure mode accompanied by shear zones. As would be expected, the brittle behaviour increased with increasing of cementation and density and decreased with an increase in confining pressure. The thickness of the shear zone in cemented samples varied between 2 and 5 cm and the inclination angle of the shear zones with respect to horizontal varied between 60° and 70°s.

3.2. STRESS STRAIN BEHAVIOUR

Deviatoric stress (q), excess pore water pressure (u) and stress ratio (q/p') plotted against axial strain are shown in Figures 3–8 for uncemented and cemented samples at the two prepared relative densities. In these figures, the tests results are named based test condition. CU stands for consolidated undrained tests. A two-or three-digit number indicating confining pressure follows the CU caption. This number is followed by a dash and a one-digit number showing the number of repeated tests on a specific sample with specific cement content tested under the same confining pressure.

Test results for uncemented (Figure 3(a)) and lightly cemented samples (1.5% cement, Figure 4(a)) show that the stiffness increases with an increase in confining pressure for such soils. However, the effect of confining pressure on the stiffness is much less significant for strongly cemented soils (Figures 6(a), 7(a) and (8a)). As can be observed from Figures 3(a)–8(a) stiffness increases with the increase in cement content. This fact is better observed in Figure. 9, which shows the change in initial stiffness with cement content at confining pressure of 100 kN/m^2. As seen from this figure, the initial stiffness increases with increase in cement content and relative density. The stress–strain curves for uncemented and lightly cemented samples (Figures 3(a) and 4(a)) do not usually show any peak strength, while those for highly cemented samples (including 4.5%, 6% and 9% cement) show clear peak strength (Figures 6(a), 7(a) and 8(a)).

Stress ratios against axial strain are presented in Figures 3–8 as well. The value of the maximum stress ratio, $(q/p')_{\max}$, for each test group decreases with an increase in confining pressure as can be seen from Figures 3(c)–8(c). However, the value

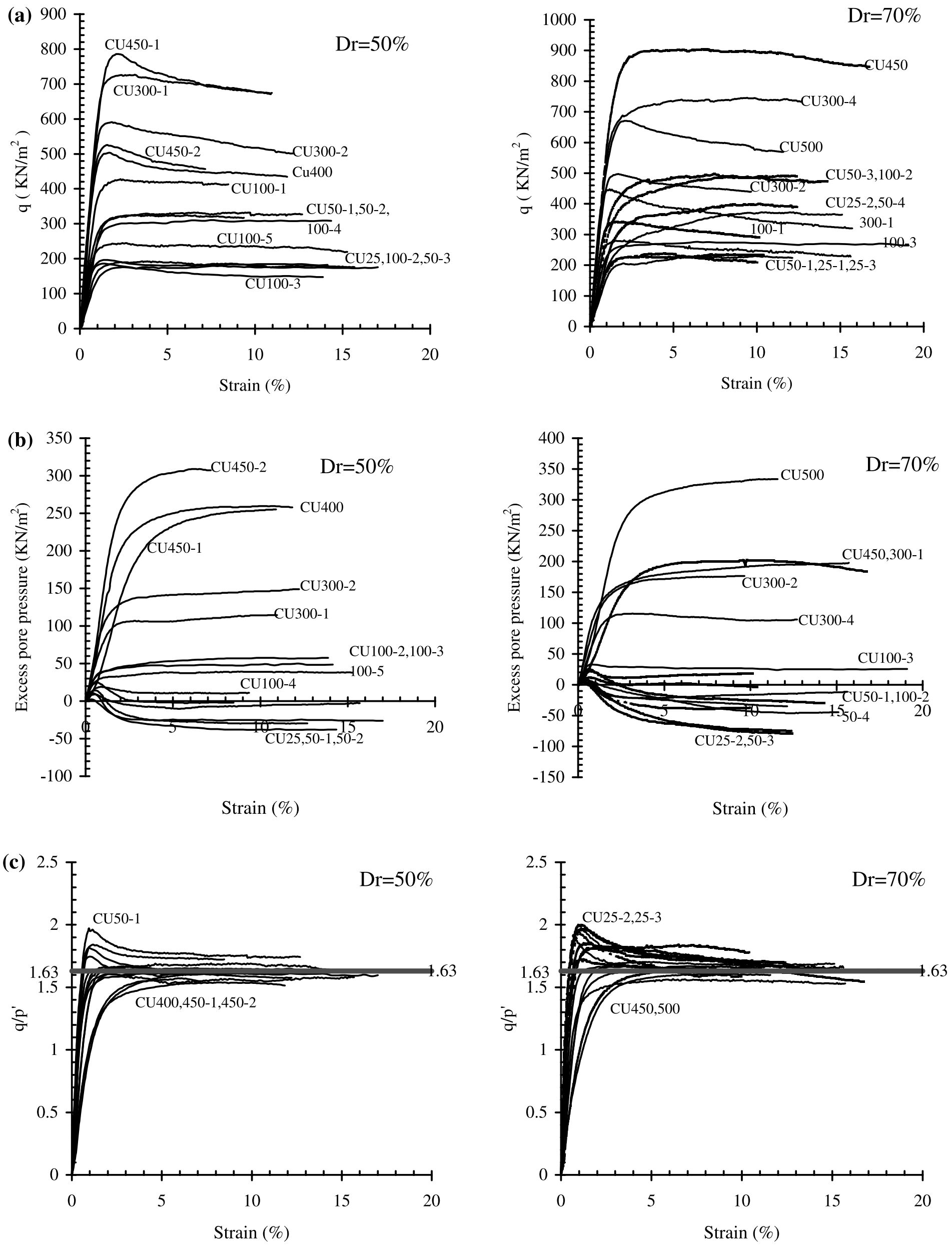


Figure 3 Behaviour of uncemented sandy gravel samples: (a) deviatoric stress, (b) excess pore water pressure, (c) stress ratio

increases with relative density and cement content. Consider the maximum value of $(q/p')_{\max}$ for each test group (for the same relative density and the same cement content), which is associated with the lowest confining pressure. This value, which is

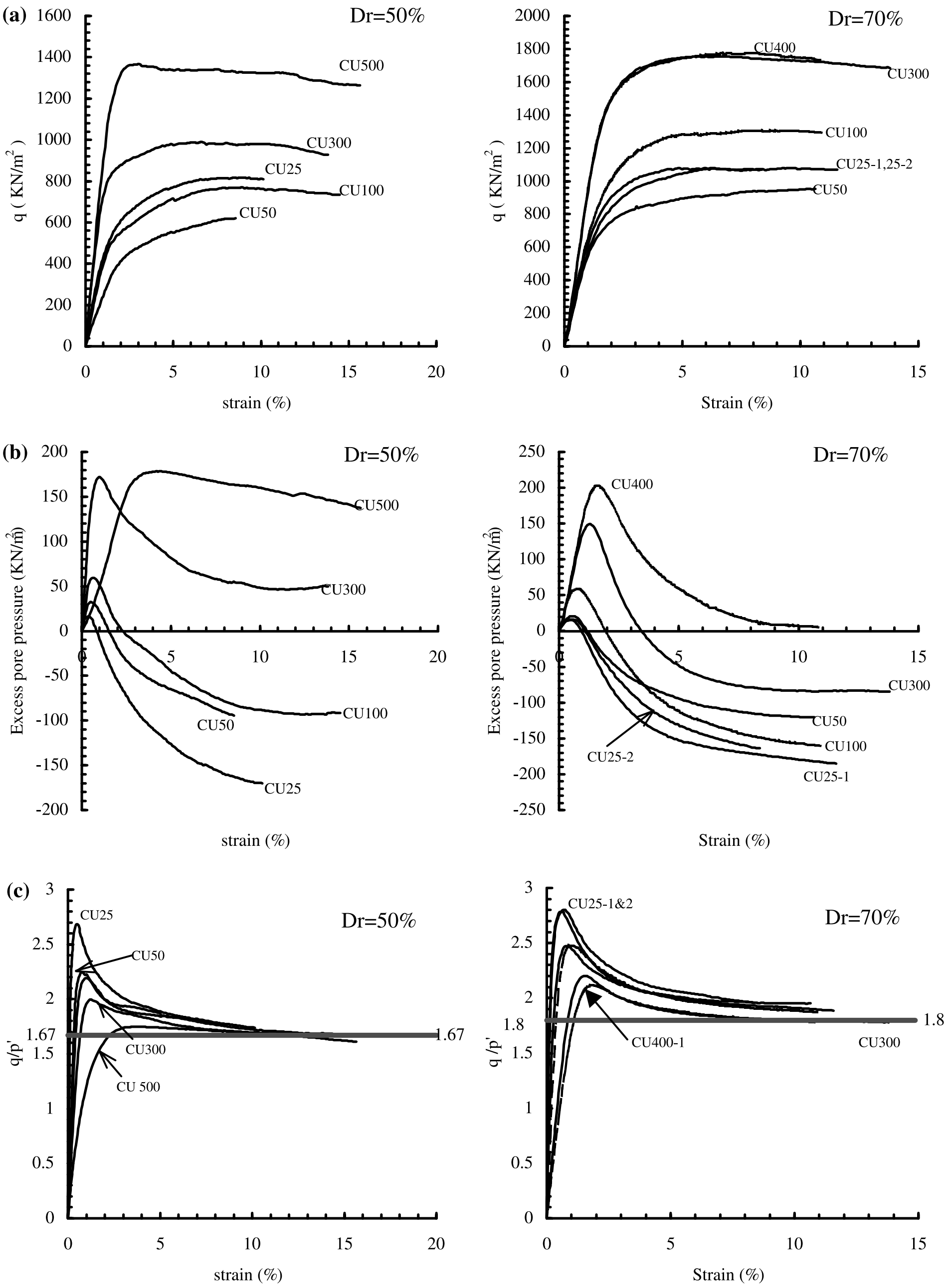


Figure 4 Behaviour of artificially cemented sandy gravel samples (1.5% cement): (a) deviatoric stress, (b) excess pore water pressure, (c) stress ratio

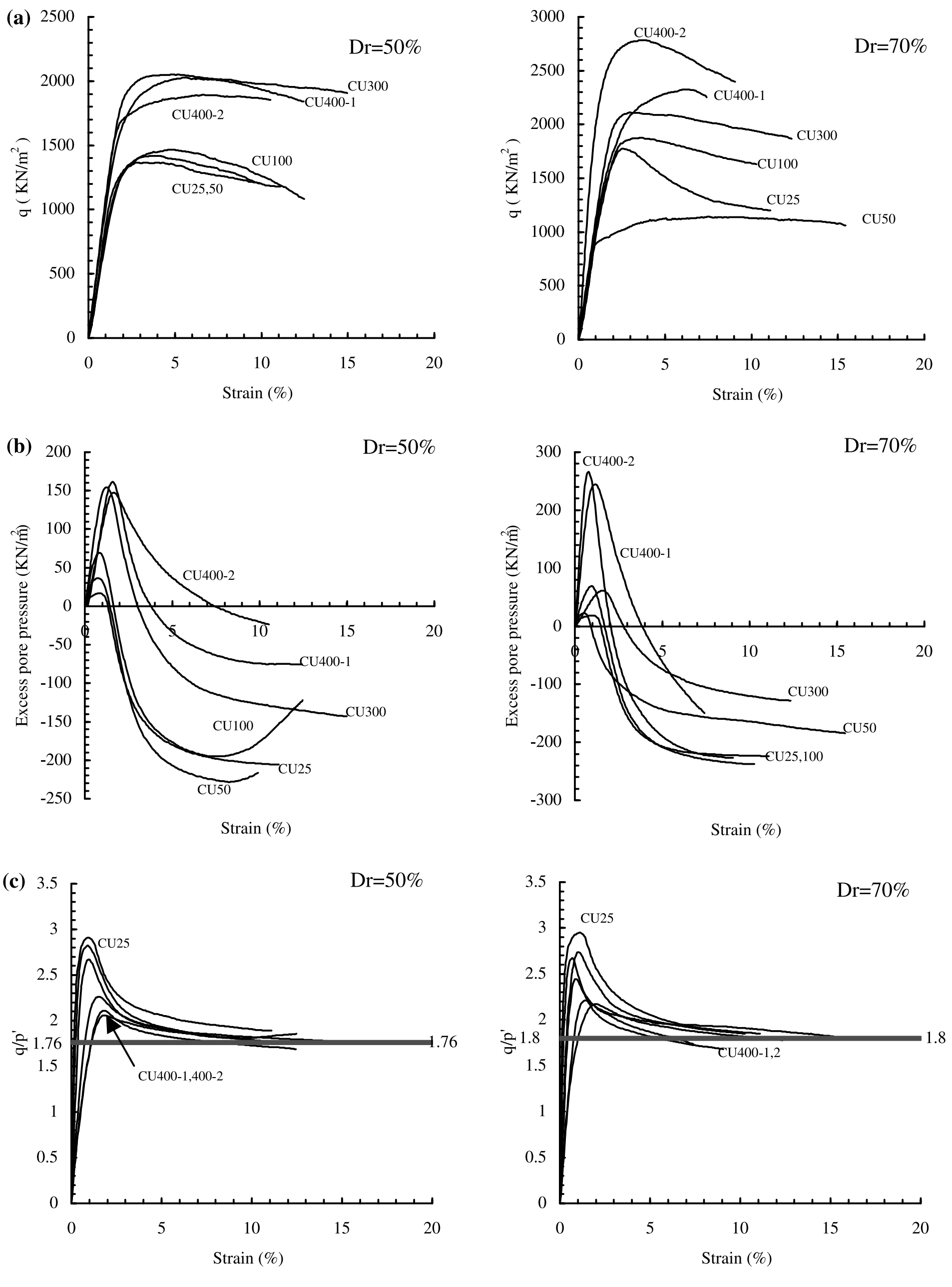


Figure 5 Behaviour of artificially cemented sandy gravel samples (3% cement): (a) deviatoric stress, (b) excess pore water pressure, (c) stress ratio

observed from Figures 3(c)–8(c), varies between 1.97 for uncemented samples with a relative density of 50% to 3.0 for cemented samples with a relative density of 70% and cement content of 4.5% and more. A similar value of 3.0 is seen for cemented

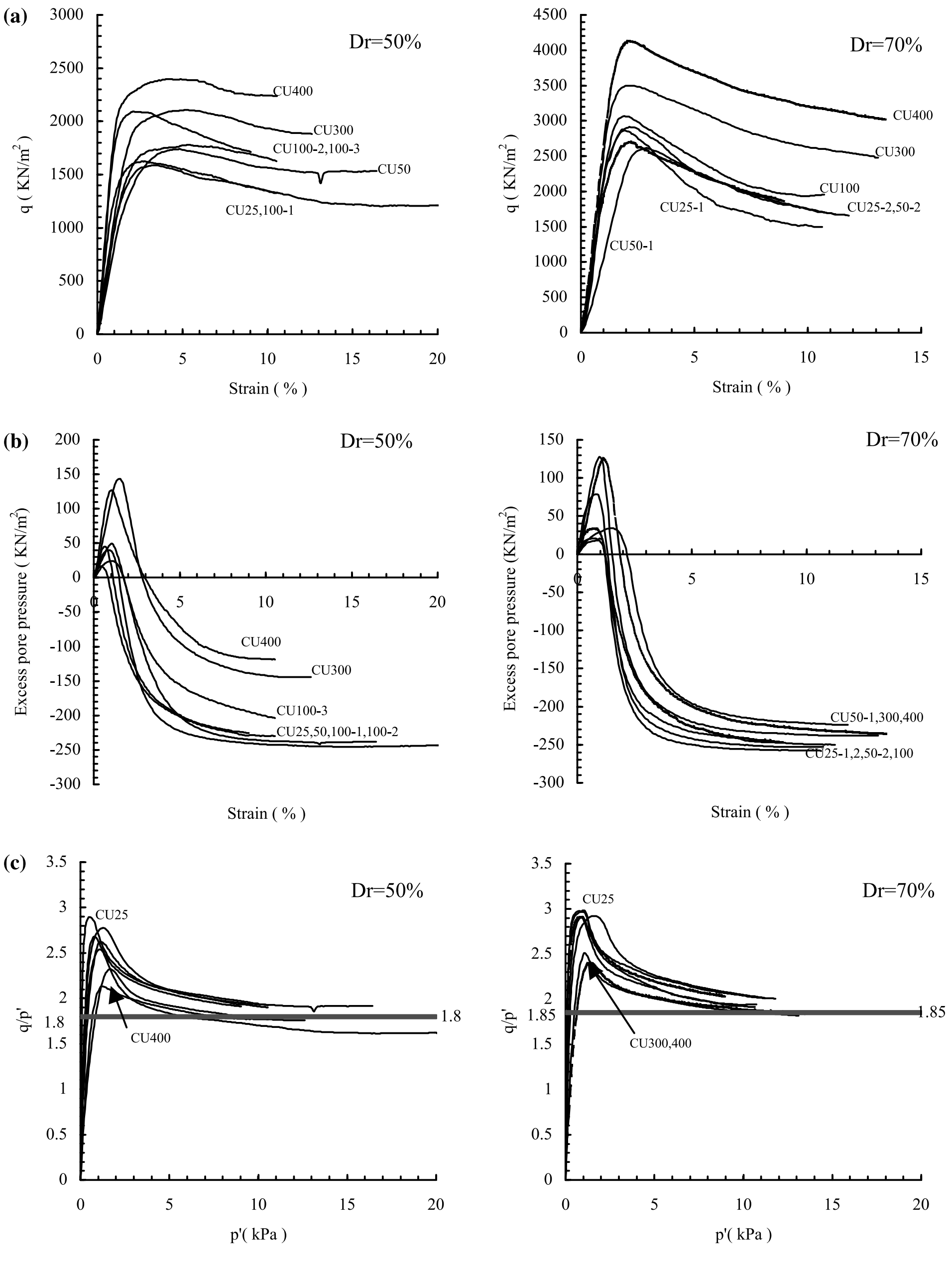


Figure 6 Behaviour of artificially cemented sandy gravel samples (4.5% cement): (a) deviatoric stress, (b) excess pore water pressure, (c) stress ratio

samples with a relative density of 50% and cement content of 9%. The maximum value, 3.0, is equal to the slope of drained effective stress path or the slope of tension cut off line. This subject will be discussed further below.

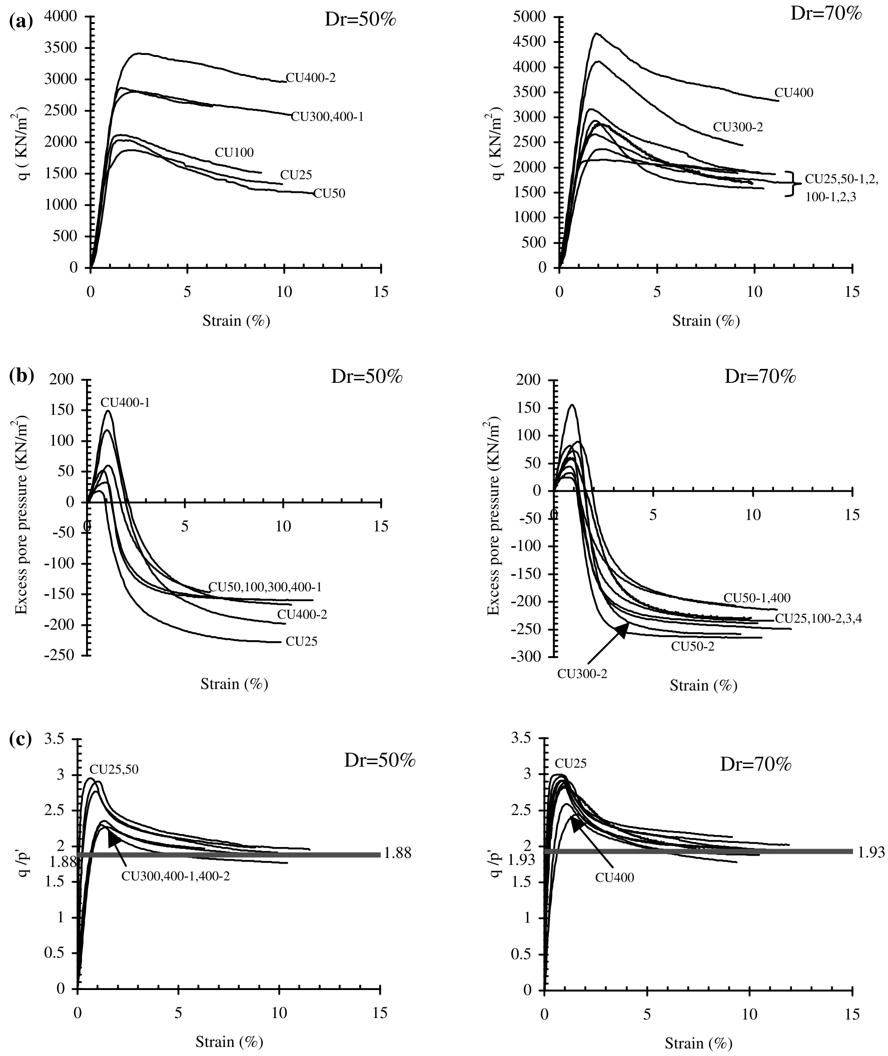


Figure 7 Behaviour of artificially cemented sandy gravel samples (6% cement): (a) deviatoric stress, (b) excess pore water pressure, (c) stress ratio

In triaxial tests, the stress ratio usually approaches a constant value at ultimate state (high strain levels). However, the tests carried out (especially on cemented samples) had to be terminated at medium strain levels and therefore the ultimate state usually could not be reached. This was due to the fact that the membranes used had usually been damaged at medium strain levels due to the presence of coarse subangular grains in the base soil. However, the ultimate stress ratio can be defined as shown in Figures 3(c)–8(c). The ultimate stress ratio is the slope of critical state line in q–p' space, which is also called (M). The value of ultimate stress ratio, M, for

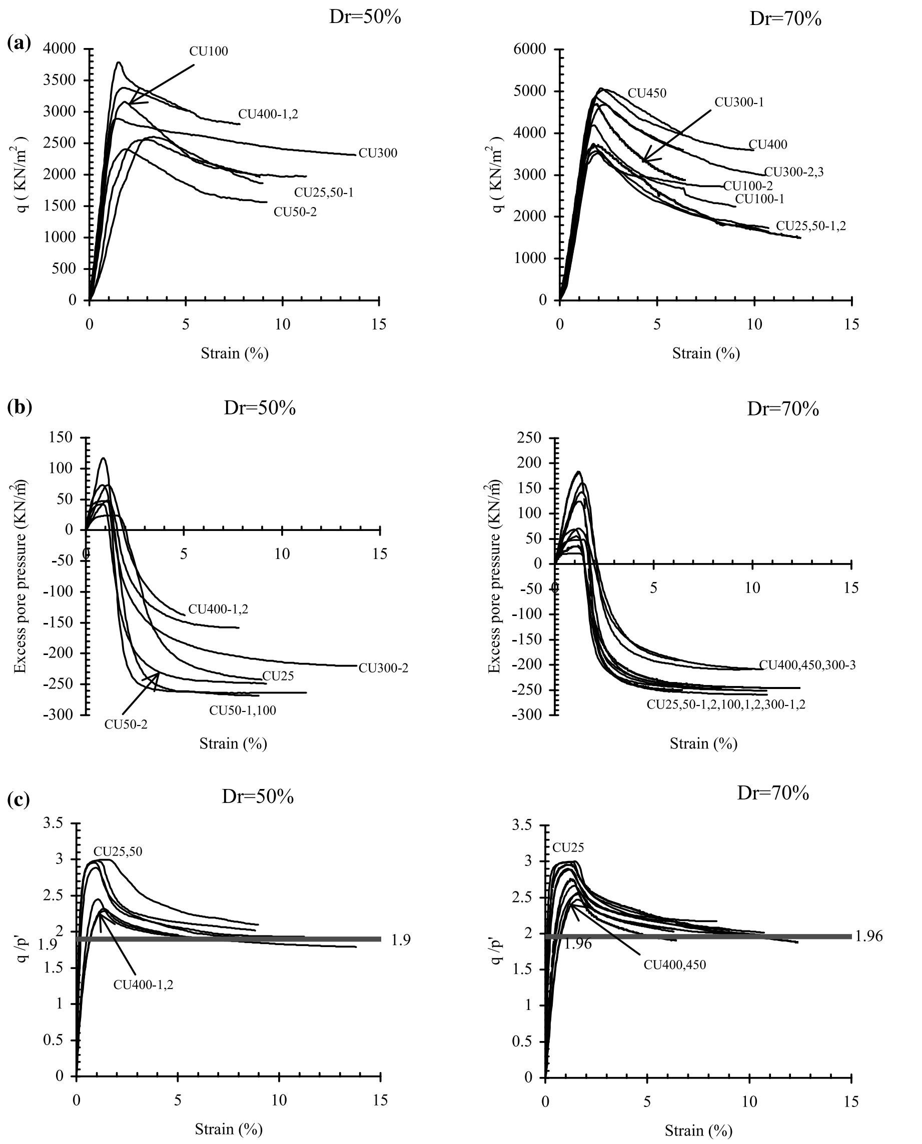


Figure 8 Behaviour of artificially cemented sandy gravel samples (9% cement): (a) deviatoric stress, (b) excess pore water pressure, (c) stress ratio

uncemented and cemented samples are given in Table 2 and the variation of M with respect to cement content is plotted in Figure 10. Table 2 and Figure 10 indicate that the ultimate stress ratio, $(q/p')_{\mathrm{ult}}$ or M, is constant for uncemented samples. However, the value increases for cemented samples with increase in cement content and density.

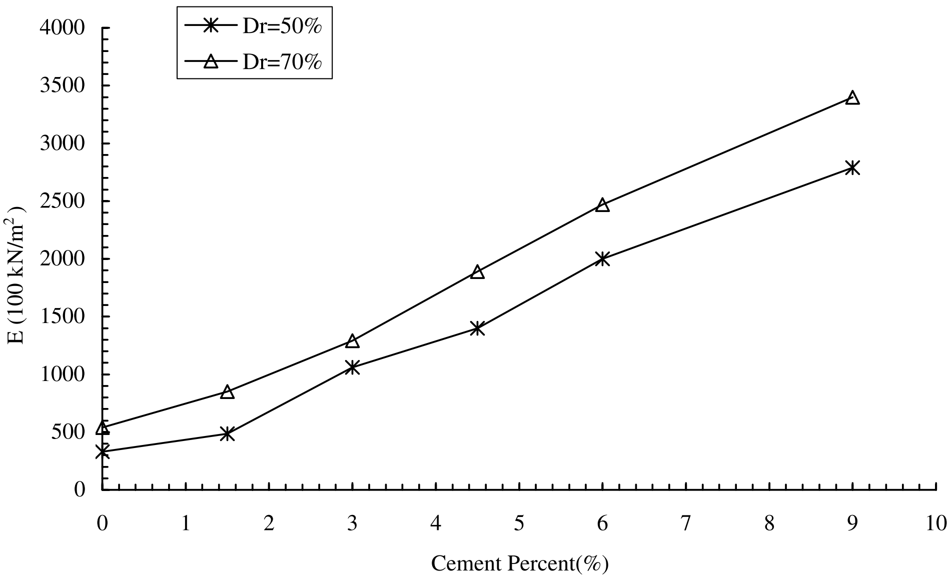


Figure 9 Variation of initial stiffness against cement content

3.3. EXCESS PORE WATER PRESSURE

Development of excess pore water pressure during shear loading is shown in Figures 3(b)–8(b) for uncemented and artificially cemented samples. Excess pore water pressure in the uncemented samples at high confining pressures remained positive during the whole process of shear loading, meaning that the behaviour of the soil remains contractive from the beginning to the end of the tests. Test results on the other cemented and uncemented samples show that the excess pore water pressure is

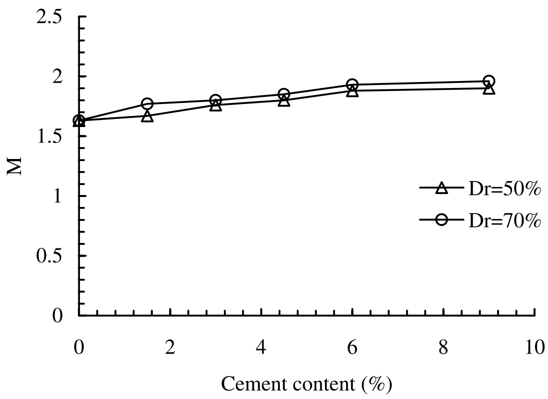


Figure 10 Variation of ultimate stress ratio against cement content

Table 2 Ultimate stress ratio at two relative densities and different cement contents

Cement content (%)	0	1.5	3	4.5	6	9
Relative density 50%	1.63	1.67	1.76	1.8	1.88	1.9
Relative density 70%	1.63	1.77	1.8	1.85	1.93	1.96

positive at the beginning of shear (tendency for sample contraction) and changes to negative as shear strain increases (tendency for sample dilatation).

As can be observed from Figures 3(b)–8(b), the maximum negative excess pore water pressure increases with increase in cement content. This increase is more visible for the samples with a cement content of up to 4.5%. The test results on samples with 6% and 9% cement content, however, do not show such an increase in negative excess pore water pressure compared to that of the samples with 4.5% cement content. This behaviour can be attributed to the greater tendency of the more cemented samples to dilate up to a cement content of 4.5%; meaning that the set of 'cemented pockets'(groups of particles hold together by the cementing agent), which are formed during shear in cemented samples, causes the tendency for dilation in undrained tests and dilation itself in drained tests. The pore water pressure responses suggest that the size of such cemented pockets increases, as the cement content increases, up to a cementation value of 4.5% in this study. After that, as is observed, the negative pore pressure reaches a similar limiting value, suggesting that the increase in cement content does not have any considerable effect on the formation of such cemented pockets.

The change from positive to negative excess pore water pressure in uncemented to lightly cemented samples is gradual showing a change from ductile to slightly brittle behaviour (see Figures 3(b) and 4(b)). However, this transition is extremely sharp in highly cemented samples, showing very brittle behaviour (Figures 7(b) and 8(b)). In the middle range of cementation, the ductile behaviour changes to brittle one as the cement content increases (Figures 5(b) and 6(b)).

3.4. STRESS PATH

The stress paths for the tests on uncemented and cemented samples are shown in Figure 11. Critical state lines are also shown in these figures. The critical state lines shown in Figure 11 are consistent with the M values shown in Figures 3(c)–8(c) and given in Table 2. In fact, both graphs of q/p' vs. axial strain and stress paths were studied simultaneously to reach to a consistent and acceptable M value or critical state line for any cement content and each density. It should be noted that the chosen critical state line might not represent the true critical state because most of the samples showed a shear zone. Therefore, it is difficult to say whether the stress conditions are representative of the sample as a whole. In addition the strain levels to which the samples were subjected are not that high and may not be sufficient to arrive at the real critical state condition. However, it can be seen from the pore water pressure responses (Figures 3(b)–8(b)) that a constant value was usually achieved by the end of the test, suggesting that a true ultimate state was virtually achieved.

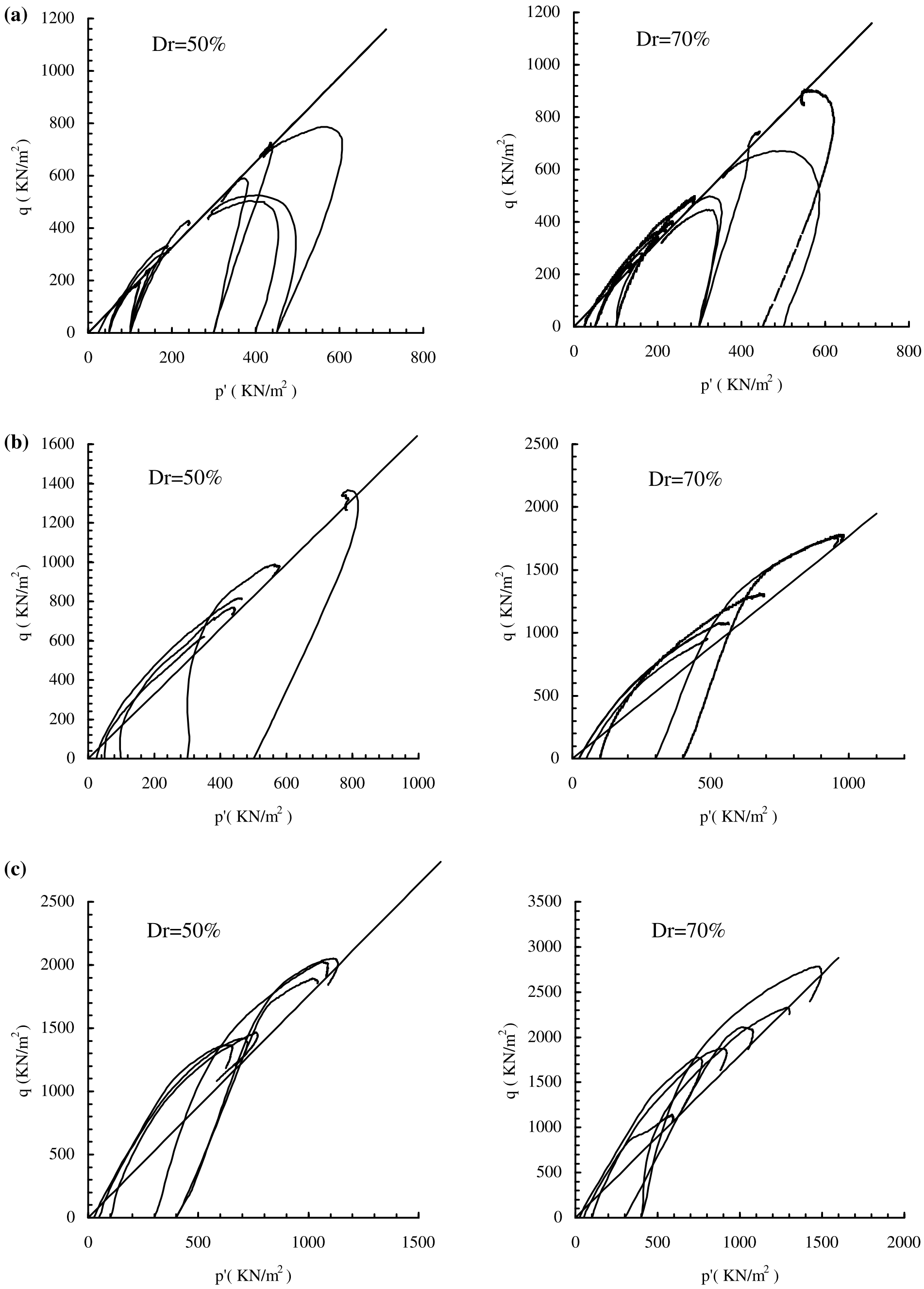


Figure 11 Stress paths for two relative densities 50% and 70%: (a) uncemented, (b) 1.5% cement, (c) 3% cement, (d) 4.5% cement, (e) 6% cement, (f) 9% cement

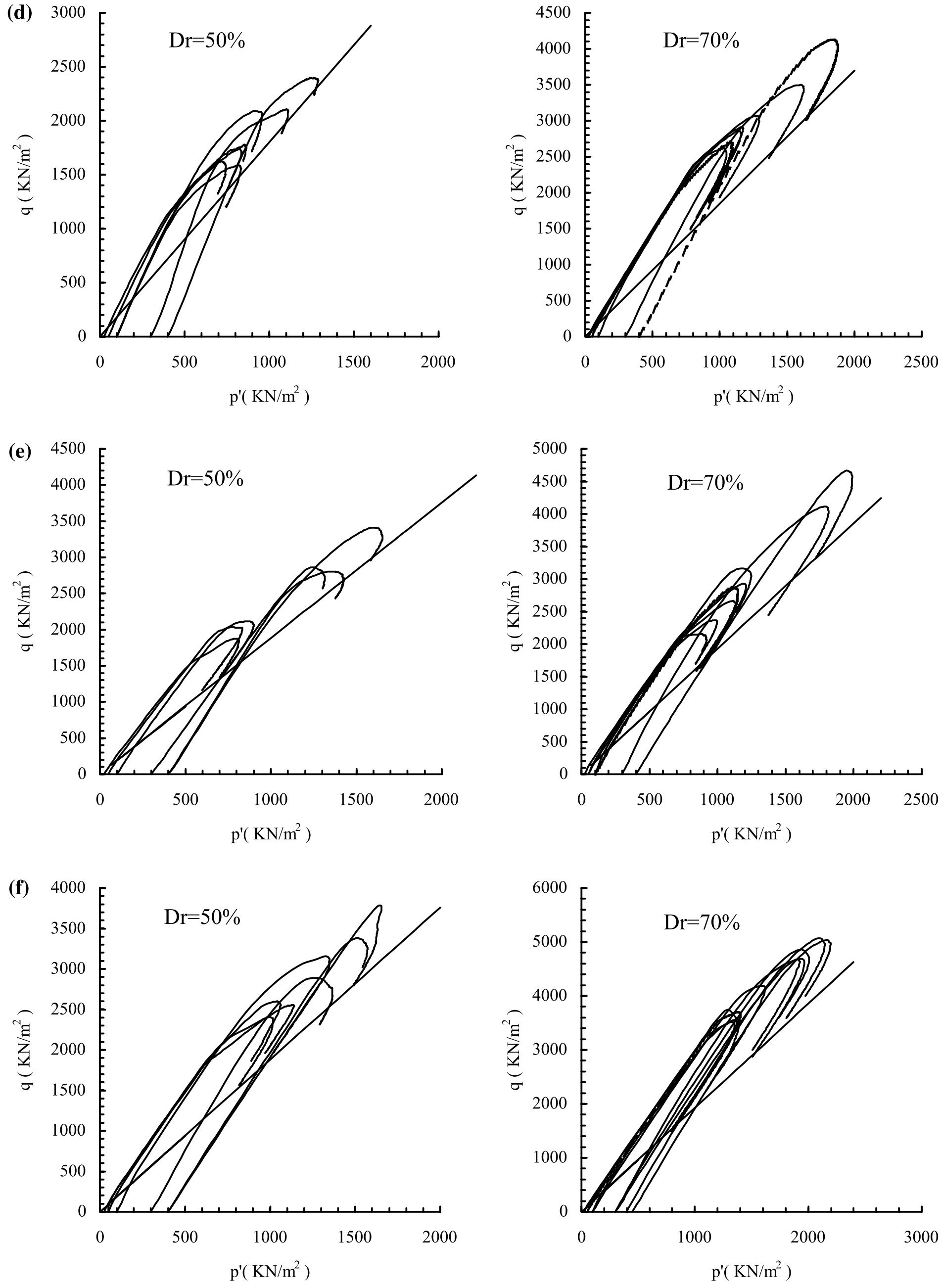


Figure 11 Continued.

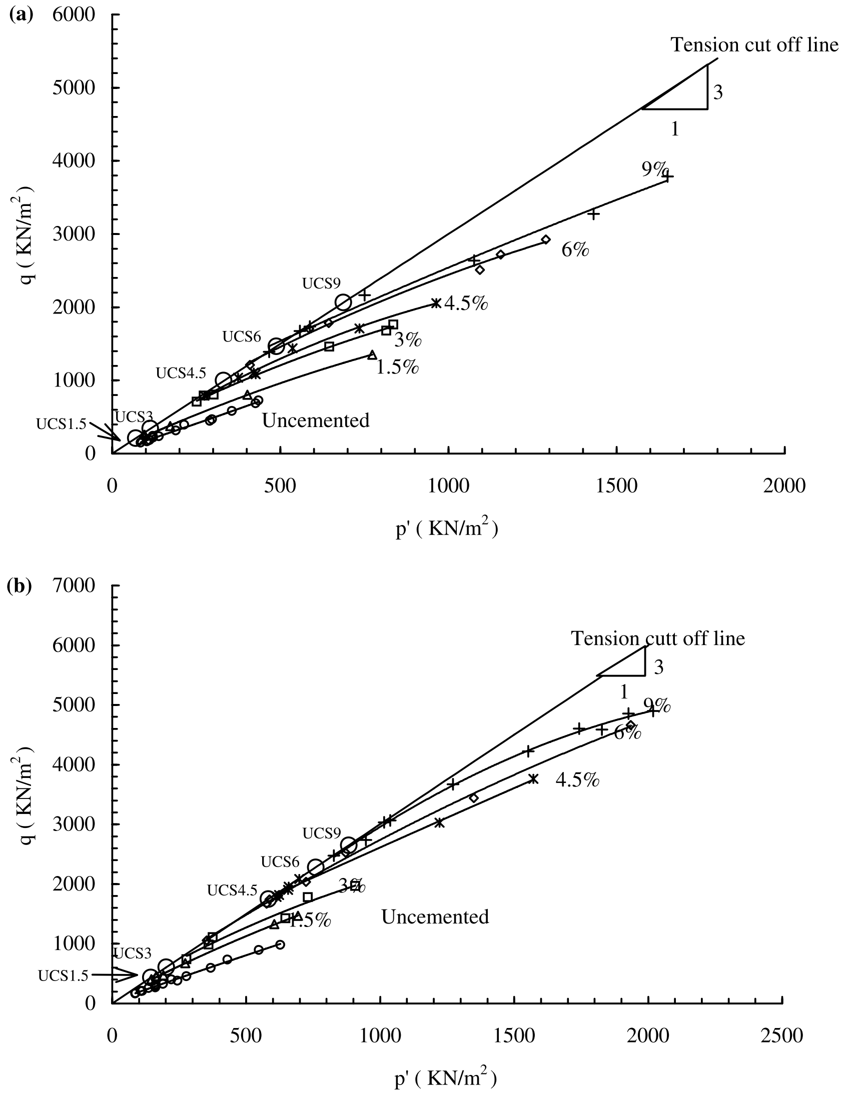


Figure 12 Limiting stress ratio surfaces and unconfined compression tests results: (a) relative density of 50%, (b) relative density of 70%

Uncemented samples tested at low confining pressures and all cement samples tested showed dilative behaviour. The stress paths for these tests can be observed in Figure 11. As can be seen from this Figure the maximum stress ratio $(q/p')_{\max}$ is reached first, and then the stress path continues to climb until the maximum deviatoric stress is reached. The stress paths then fall to reach to the critical or ultimate state. Uncemented samples tested at high confining pressures, however, showed contractive behaviour. As can be seen from Figure 11(a), the maximum deviatoric stress is reached first for this group of tests. The ultimate state and maximum stress ratio conditions coincide at the end point of such tests.

The slope of the beginning of the stress path for cemented samples, before the maximum stress ratio is reached, (as can be observed from Figure 11) approaches that of a drained stress path (slope of 3) as the cement content increases. This fact shows that the cementation produces increase in stiffness and therefore the generation of pore water pressure is small (i.e. there is a reduced tendency for volumetric strain to occurs).

3.5. LIMITING STRESS RATIO SURFACE

The limiting stress ratio surface in q–p' space for uncemented and cemented samples with two relative densities of 50% and 70% are shown in Figure 12. The limiting stress ratio surfaces in this study are considered as the best 2nd order polynomial lines through the points associated with $(q/p')_{\max}$ for each set of tests. The tension cut off line (stress path for $\sigma'_3 = 0$) is also shown in Figure 12. As can be observed, the limiting stress ratio surface expands as cement content and density increase. The limiting stress ratio surfaces are limited to the tension cut off line at low stress levels. For each set of tests associated with any amount of cementation, the intersection point of limiting stress ratio surface and tension cut off line is different. This point moves to higher stress levels for samples with higher cementation. The intersection point of limiting stress ratio surface and tension cut off line should be the same as the point associated with uniaxial tests. To check this fact, a number of unconfined compression tests were carried out and the results are plotted in Figure 12 with open circles. We can observe that such open circles coincide fairly well with the intersection of the limiting stress ratio surface with the tension cut off line. For each amount of cementation, however, there are some small disagreements between some of these points (particularly for USC3), which can be attributed to the different test conditions in uniaxial and triaxial tests. The results of unconfined compression tests on cemented samples for different cement content and relative densities are shown in Figure 13. As can be observed from this figure, the unconfined compression strength increases with an increase in cement content and density. However, there is a sudden change at 3% cement content.

4. Effect of Cement Type on Limiting Stress Ratio Surface

Limiting stress ratio surfaces for 1.5%, 3% and 4.5% cement and relative densities of 50% and 70% are shown in Figure 14. The results of the tests carried out by Asghari et al. (2003) on the same base soil with lime cementation are also shown in this figure. The relative density of the samples tested by Asghari et al. is about 68%, very similar to the 70% tests in the present study. We can observe from Figure 14 that for 1.5% cement content (low cementation) the test results on lime-cemented samples coincide with the results of this study using Portland cement as cementing agent. However, the results for 3% lime cementation fall below the test results associated with Portland

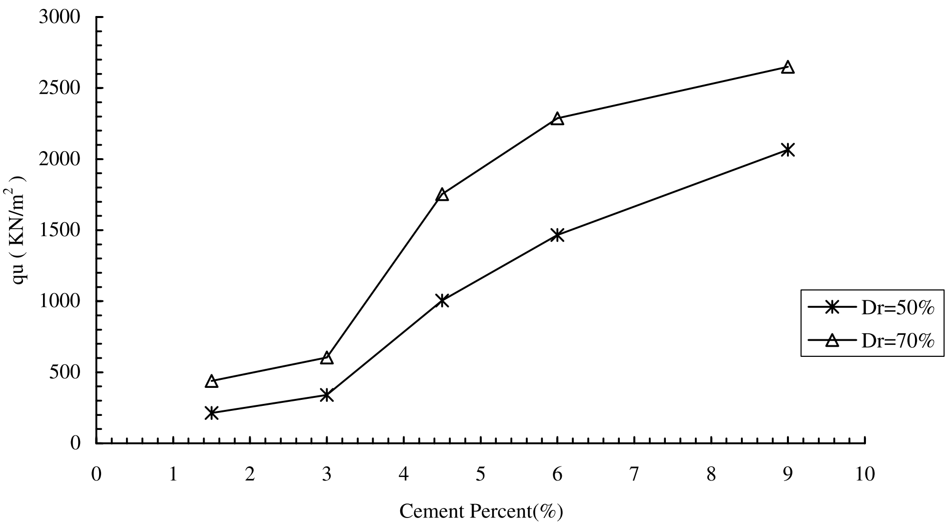


Figure 13 Variation unconfined strength with cement content

cement for $D_r = 70\%$ (this study). The values of the maximum stress ratio for 4.5% lime cementation and a relative density of about 70%, again fall below the limiting stress ratio surface for 4.5% cement content and $D_r = 70\%$ for Portland cement sample. It is interesting to note that indeed, the values of the maximum stress ratio for 4.5% lime cementation and a relative density of about 70%, coincide with the limiting stress ratio surface of 4.5% cement content and $D_r = 50\%$ for Portland cement samples. This shows that Portland cement provides more strength compared to that of the lime especially at higher cement contents.

Triaxial tests were not carried out on lime-cemented samples with higher than 4.5% cementing agent. However, direct shear tests on the same soil with higher cementing agent show that for this soil, an increase of lime from 4.5% decreases the cemented soil strength (Haeri et al., 2002). From the results of this study, however, the limiting stress ratio surface expands or the strength increases as the Portland cement content increases even beyond 4.5%. This difference is due to the different hydration process between lime and Portland cement as cementing agent.

5. Conclusions

An extensive series of laboratory tests including triaxial compression tests and unconfined compression tests has been carried out to understand the mechanical behaviour of a cemented coarse-grained material, sandy gravel, which is representative of the Tehran alluvium. Portland cement type I was used as the cementation agent for preparing artificially cemented samples. The test results provided the following main conclusions:

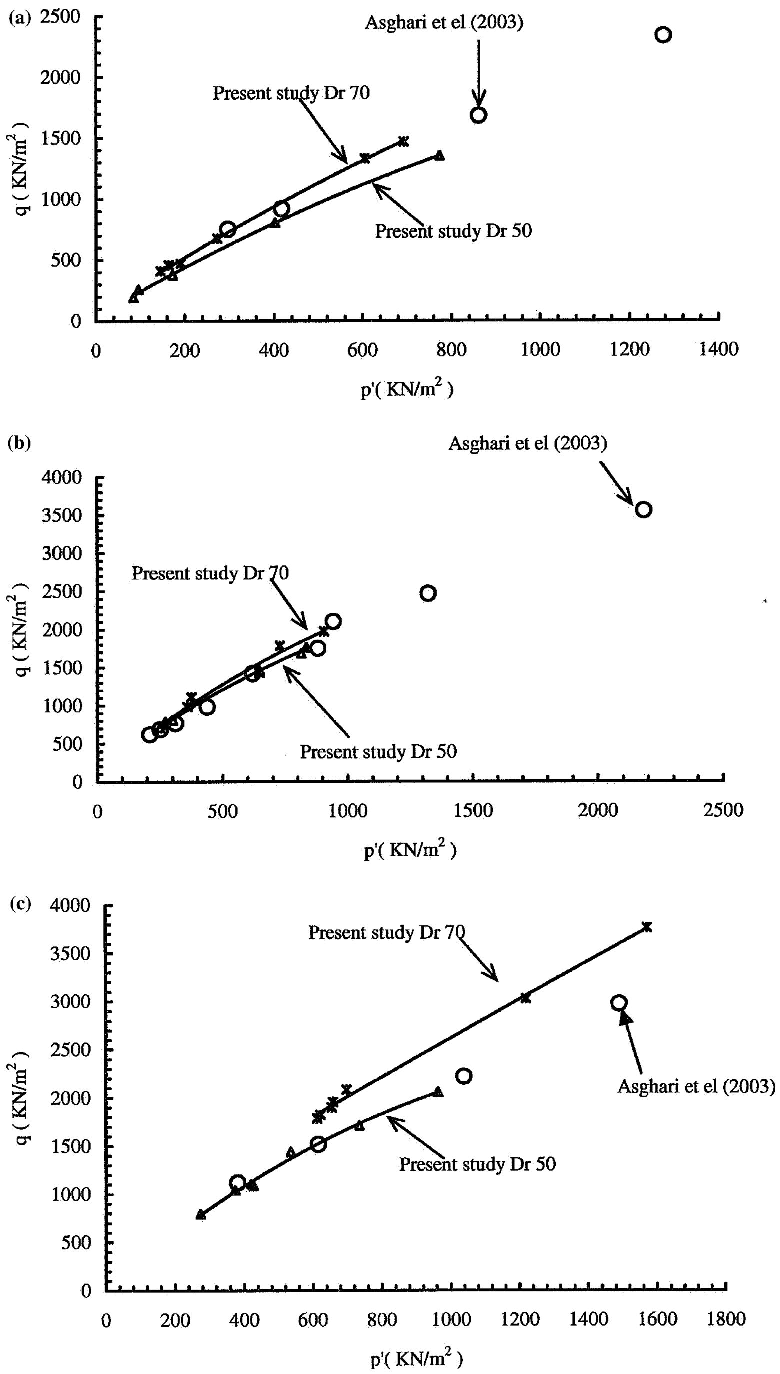


Figure 14 Comparison of the maximum stress ratio for tests on Portland cement and limy cemented samples: (a) 1.5% cement, (b) 3% cement, (c) 4.5% cement

1. Uncemented samples, when tested at confining pressures greater than 100 kN/m^2, and lightly cemented samples (1.5% cement) tested at a confining pressure of 500 kN/m^2 show contractive behaviour accompanied with a barrelling failure mode. The excess pore water pressure and stress ratio against strain do not show clear peaks for these samples. However, cemented samples (3%, 4.5%, 6% and 9% cement content) show dilative behaviour associated with a shear zone and a brittle failure mode. The excess pore water pressure and stress ratio against strain for these samples do show clear peaks.
2. Stress–strain curves show that stiffness increases with an increase in cement content and density. For uncemented and lightly cemented samples (1.5% cement) stiffness increases with an increase in confining pressure as well.
3. The maximum stress ratio, $(q/p')_{\max}$, for each test group decreases with an increase in confining pressure. The maximum value of $(q/p')_{\max}$ for each test group increases with an increase in cement content and density, but reaches a limiting value, i.e. 3.0, which is the same as the slope of the tension cut off line.
4. The excess pore water pressure for both uncemented samples at low confining pressures and all cemented samples are positive at the beginning of the tests. This value changes to negative as shear strain increases. The transition from positive excess pore water pressure to negative in uncemented and lightly cemented samples are gradual, while it is extremely sharp in highly cemented samples.
5. The slope of critical state line, M, increases with increase in cement content and density.
6. The limiting stress ratio surface defined by $(q/p')_{\max}$ for each set of tests, expands with the increase in cementation and density. The limiting stress ratio surfaces are limited by the tension cut off line.
7. Unconfined compression strengths of cemented samples increase with increases in cement content and density. The results of unconfined compression tests coincide reasonably well with the intersection points of the limiting stress ratio surface and the tension cut off line.
8. Comparing the limiting stress ratio surfaces for two types of cementation (lime and Portland cement) shows that the type of cementation is not very important at low cement contents (1.5% cement), however, as the amount of cementing agent increases the limiting stress ratio surfaces for the samples cemented with Portland cement are higher than those for lime cementing.
9. Increase in relative density results in the expansion of limiting stress ratio surface and increase in the initial stiffness, maximum stress ratio $(q/p)_{\max}$ and the tendency to dilation during shear.

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