Compression and Extension Monotonic Loading of a Carbonate Sand

H. Salehzadeh[1], M. Hassanlourad[2], D. C. Procter[3], C. M. Merrifield[3]

Received: February 2008; Accepted: November 2008

Abstract: *The unique behaviour of carbonate sediments under shear loading has stimulated in investigating of their geological and engineering properties. Their shapes are very different varying from needle shaped to platy shaped. Hence, it is important to examine their fabric effect on soil response under shearing condition. To this aim a series of small scale laboratory element testing were carried out on North Cornwall Rock" beach sand. Non-cemented and cemented Carbonate sand response under compression and extension loading and different initial density and confining pressure with samples allowed to be drained were investigated and compared. The results show that the sand shear strength under Extension loading is lower than compression regarding to anisotropic fabric due to platy and needle shape of grains. The anisotropy is reduced with increasing the confining pressure and initial relative density with non-cemented sand. Furthermore, present of cement bounds reduces the anisotropy especially in low confining pressures.*

Keywords: *Carbonate Sand, Fabric Anisotropy, Monotonic Test.*

1. Introduction

Basically, there is some inherently anisotropy in granular soils considering of particles shape and deposition direction. Therefore, the initial anisotropy affects the shear strength and behaviour regarding the principal stresses and deposition direction. A number of researchers have investigated this factor on the silicate sandy soils (e.g. Arthur and Menzies 1972, Oda and Koishikawa, 1977, Symes et al. 1984, Lam and Tatsuoka, 1988, Muhunthan et all. 2000, Andrei and Lade 2003).

The soil most abundant in tropical marine environments is carbonate sediments which have inherently high crushing properties distinguishing them from low crushable sediments such as silicate soils. Carbonate materials exist in a wide variety of forms, which are often composed of calcium or other carbonates with soft grains that are weak and readily crushable.

Isotropic normal compression and monotonic compression response of non-cemented and cemented carbonate sediments have been studied by many researchers (e. g. Airey et al., 1988, Coop and Atkinson, 1993, Ismail et al., 2000 and 2002 and Sharma and Ismail, 2006). However, it is very hard to find results from extension tests on non-cemented and cemented carbonate sediments in the literature. Also the wide variation of the origin of calcareous soils, due to their offshore locations and the related fauna that make their formation, merit more research into the behavior of these soils individually.

In this paper results of some compression and extension triaxial tests on a carbonate sand in loose, dense and loose weekly cemented states are presented. Results demonstrated the different shear strength in compression and extension regarding the relative direction of principal stresses and deposition directions.

2. Material

The non-cemented carbonate sand was obtained from the coast of North Cornwall at Rock" beach opposite Padstowe-England. The soil was predominantly of biogenic origin of recent sediments. Particle size distribution and some physical properties of the Rock sand are presented in Fig. 1 and Table 1 respectively.

Calcite and aragonite were the most prevalent carbonate minerals distinguishable in the soil using XRD method. Skeletal spongy shaped carbonate particles are most abundant containing intraparticle voids (Fig. 2).

The intraparticle porosity of the sand, was measured using the method described by Golightly (1988) and was found to be 4%, which is consistent with the values the same author achieved for Dog's Bay sand (i.e. 4-6%). The mean carbonate content value was measured 90%.

[1] *Assistant Professor in the Iran University of Science and Technology, I.R.IRAN*
[2] *Islamic Azad University, Zanjan Branch.*
[3] *Senior lecturers in the Manchester University, U. K.*

Table 1 Some physical characteristics of the Rock sand.

Skeletal spongy and needle shaped	Carbonate particles complexity
4%	Intraparticle porosity
2.72	mean solid specific gravity
90 %	mean carbonate content
14.8 kN/m^3	Max. dry density
11 kN/m^3	Min. dry density
1.47	Max. void ratios (e_{max})
0.83	Min. void ratios (e_{min})
well-graded, fine-medium sand (Fine fraction=40%, medium fraction=50%)	classification

A triaxial apparatus, common during all the tests was designed to apply monotonic loading. It was capable of testing soils under controlled stress or displacement and under either undrained or drained conditions.

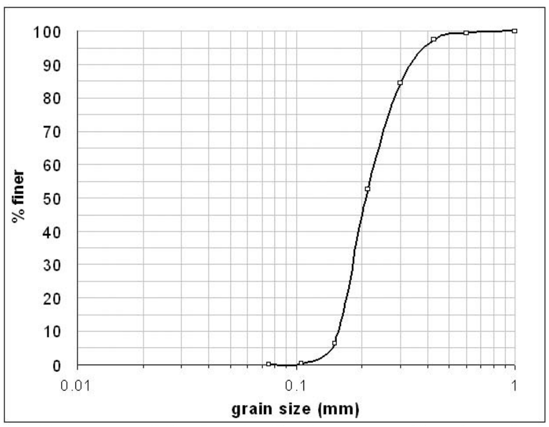

Fig. 1 Grain size distribution of North Rock beach sand.

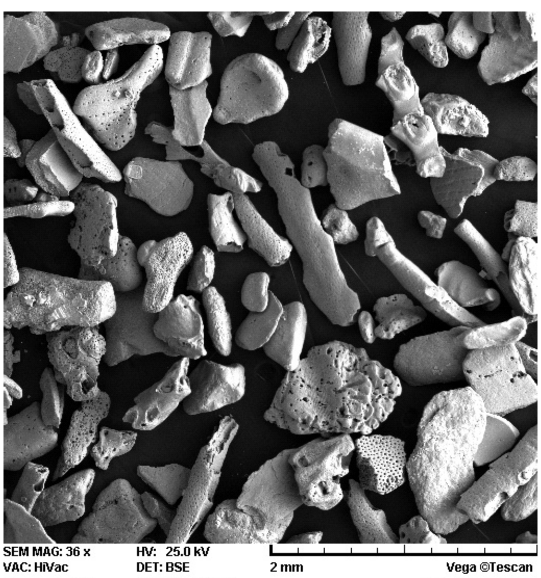

Fig. 2 SEM photo of North Rock beach sand.

3. Sample Preparation and Testing Planning

The boiled soil in water had been passed through a 1mm sieve into distilled water was poured into the former in two loose and dense states. The loose state was acquired by spooning the wet soil very slowly and with a spiral motion, in to the former which was already completely filled with distilled water. Tests on samples with relative densities in the ranges 40-52% after consolidation were selected as representing loose behavior. The dense state resulted by spooning the soil in layers and then vibrating the cell base against the machine's pedestal for each layer. Relative densities of 85-96% were achieved after consolidation. Tests on cemented samples with relative densities 44% after consolidation were done.

The cell and back pressure were increased incrementally so that the effective confining pressure varied between 10kPa and 20kPa until a back pressure of 220kPa was attained. The sample was left for 24 hours under the 10kPa effective confining pressure. Using this method B values of 0.95 or more were obtained**.** The soil samples were codssnsolidated to effective confining pressures ranging from 50 to 500 kPa.

After isotropic consolidation, for monotonic compression tests a simple ball bearing was used to transfer load from the loading ram to the sample's top platen. For monotonic extension tests, the loading ram was screwed to the sample’s top platen and then loaded as extension. Some typical tests, which are listed in Table 2 were selected and analysed.

4. Stress-Strain

The stress-strain of loose, dense and loose weakly cemented samples is plotted in Figures 3 to 5 respectively.

Table 2 The list of performed tests.

σ'$_0$ (kPa)	Dr (%)	Test	Cementation
100	40-52	Monotonic Compression	Non cemented
200			
300			
100	85-96		
300			
100	44		Cemented
300			
100	40-52	Monotonic Extension	Non cemented
200			
300			
100	85-96		
300			
100	44		Cemented
200			
300			

In compression, the loose sands show strain hardening behaviour up to at least 10% axial strain, followed by a small quantity of work softening at larger strains. At all confining pressures, shallow peaks are observed (Fig. 3).

The dense samples show a pre-peak strain hardening response followed by post-peak strain softening (Fig. 4).

The cemented specimens show a change in their response from brittle strain softening to ductile strain hardening with increasing confining pressure (Fig. 5). At 50kPa confining pressure, the peak strength is reached at a small strain (ε_a=1%). It is possible that at low strains, the shear resistance is largely mobilised through the cement bonds confirmed by the maximum deviator stress. Following bond failure, loss of strength occurs because the friction resistance mobilised between the "de-bonded" particles is less than the original bond strength. The "free" particles now behave in a similar way to a loose non-cemented carbonate material at low confining pressures. However, their behaviour may differ from the non-cemented sand because of the presence of cement surrounding the particles; changing particles shape (less angular) and possibly surface friction characteristics.

In extension, the stress-strain behaviour of the loose sample under a confining pressure of 100 kPa demonstrates a ductile response. With increasing confining pressure, the behaviour becomes more brittle and the peak strength is more pronounced and was reached at lower strains (Fig. 3).

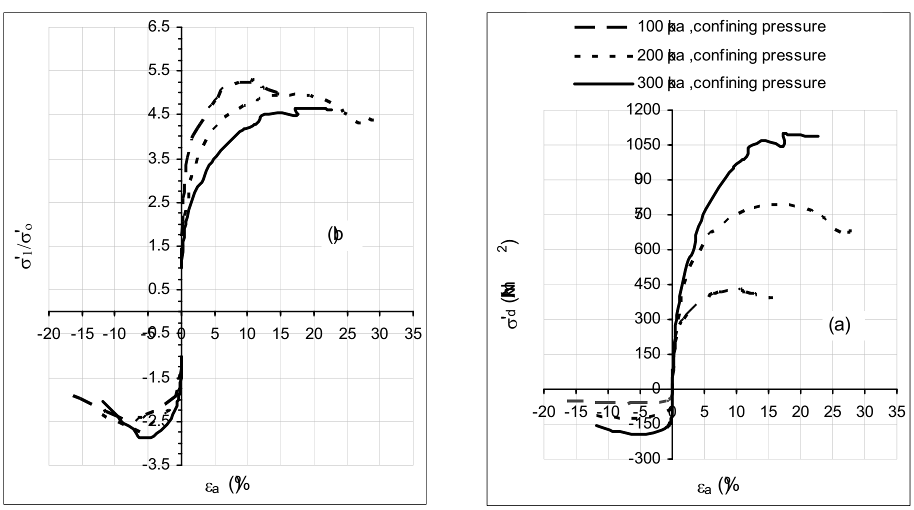

Fig. 3 (a) stress-strain and, (b) effective major principal stresses ratio-strain responses of the less interlocked carbonate sand under compression and extension shear tests.

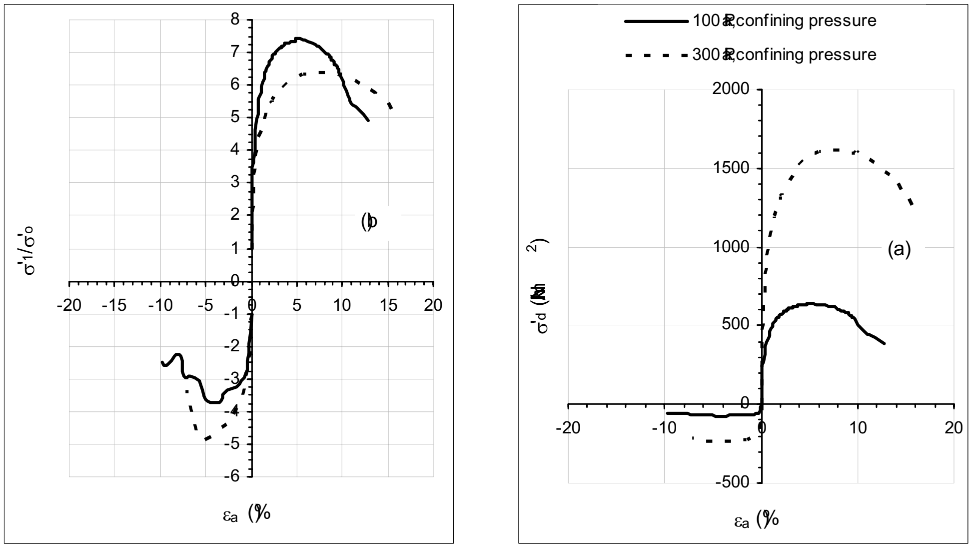

Fig. 4 (a) stress-strain and, (b) effective major principal stresses ratio-strain responses of the more interlocked carbonate sand under compression and extension shear tests.

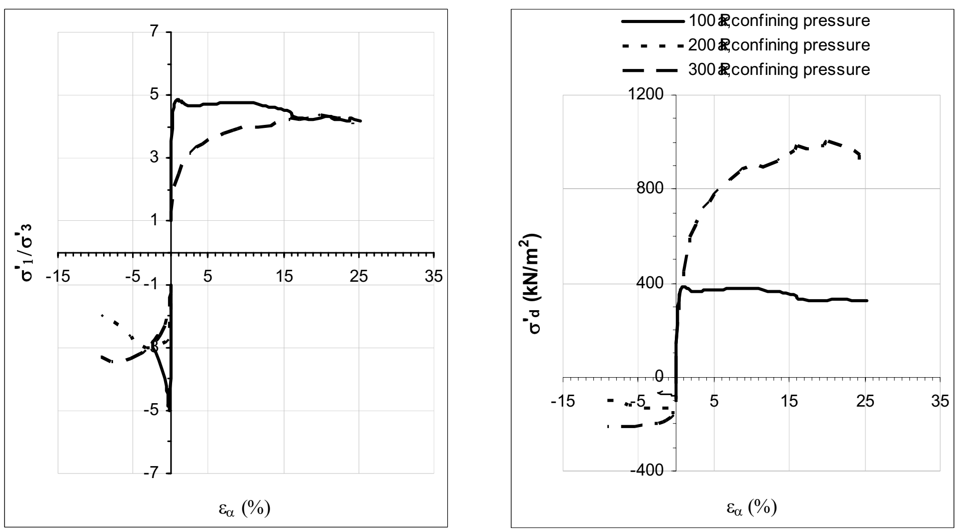

Fig. 5 (a) stress-strain and, (b) effective major principal stresses ratio-strain responses of the weakly cemented carbonate sand under compression and extension shear tests.

The dense sands exhibited brittle response with definite peaks **(**Fig. 4). The peak strengths were increased with increasing the confining pressure and were reached at approximately 4 to 5% axial strain.

In cemented specimens and at higher confining pressures, the bond component does not control peak strength. At a very early stage in the application of shear stress, the bonds are fractured throughout the sample. The frictional component of the "de-bonded" sand is large enough to control the subsequent strain hardening response of the material.

At the lowest confining pressure the cemented specimen showed a highly brittle behavior under extension (Fig. 5). The peak strength was attained at a very small axial strain (i.e. 0.35%). As the confining pressure increased, the behaviour converted from brittle with a pronounced peak to ductile with a plateau. Peak strength was achieved at higher axial strains with increasing confining pressure.

5. Shear Strength

According to Table 3, it is seen that for the loose sand the peak friction angles vary between 25^o to 29^o in extension and are about 14 ^{o} less than was found in compression for the same confining pressure (40-43)^o. Despite the limited quantity of data, especially the lack of extension tests at higher confining pressures the behavior is self-consistent and contradicts accepted soil mechanics which would normally anticipate equal peak extension and compression ϕ values (e.g. Atkinson, 1993).

For dense sand, values of 35^o to 41^o were obtained which are about 12^o less than for the compression tests (47-53) (Table 3). It is observed that $\phi'_{com.} > \phi'_{ext.}$ for all ranges of density.

There is indirect evidence in the literature (Lam and Tatsuka, 1988) which confirms the above observation. Lam and Tatsuka (1988) carried out a series of shear testing on cubical specimens of Toyoura sand with high feldspar content. The specimens were prepared by pluviating air-dried particles. The specimens were loaded in the direction of deposition and perpendicular to it. This resembles to the behaviour of the crushable carbonate sand.

The peak internal friction angles obtained by the authors are shown in Table 4. It can be observed that in extension loading, for dense samples ϕ'_p when subjected to loading in deposition direction (θ=0^o), was greater than when subjected to loading perpendicular to former direction (θ=90^o) (i.e. $\phi'_0 > \phi'_{90}$). For loose samples same regime was observed (i.e. $\phi'_0 > \phi'_{90}$).

Therefore it can be seen that the angle of shearing resistance reduces from compression to extension. This may be attributed to the much effect of crushing in the extension tests compared with the compression tests (Table 5). It may be stated that in compression increasing in p' causes more crushing which results in increase in ϕ'_p. However, in extension reduction in p' decreases the crushing and then ϕ'_p.

It can be argued that the reasons that $\phi'_{com.} >> \phi'_{ext.}$ can be attributed first to anisotropy for which in extension loading, shear forces develops along the grains but in compression develops across the grains. Secondly, at extension loading the mean effective stress (and particles breakage) is lower compared with compression loading for a constant confining pressure. Thirdly, axial strain to reach peak strength is less for extension loading than compression loading for same density. Constant values of C_{crush} ($\approx$1.1) in extension, which are deduced at the end of the tests, show that increasing the confining pressure has no effect on the crushing.

Table 3 Monotonic compression extension tests results.

Test	Packing	Con. Pre. (kPa)	After Con.	Peak values		
			Dr, %	σ'_1/σ'_o	ε_a, %	ϕ_p, degree
Compression	Loose	100	48	2.5	7.3	25
		200	52	2.7	5.8	27
		300	50	2.9	5.1	29
	Dense	100	96	3.7	3.6	35
		300	94	4.9	5.3	41
Extension	Loose	100	40	5.32	10	43
		200	43	4.96	17.7	42
		300	42	4.56	15	40
	Dense	100	80	8.69	6	53
		300	90	6.4	8.65	47

Table 4 Anisotropy effect on ϕ'_p under monotonic shear loading.

Source	Mode	Angle of loading respect to direction of deposition (degree)	ϕ'_p (degree)	
			Dense	loose
Lam and Tatsuoka	Compression	0	43	37
	Extension	90	38	33
Writer	Compression	0	50	42
	Extension	90	38	27

Table 5 Coefficient of crushing

Mode of testing	Confining pressure, kPa	State	C_{crush}^{*}
Monotonic Compression	100	Loose	1.50
	200		-
	300		2.12
	100	Dense	1.55
	300		1.89
Monotonic Extension	100	Loose	1.07
	200		1.14
	300		1.14
	100	Dense	1.14
	300		1.14

$^{*}C_{crush} = \dfrac{D_{10}(AfterTest)}{D_{10}(BeforeTest)}$, (Datta et all. 1979)

In another study, the effect of anisotropy induced by the method of sample preparation was studied by loading four cubic specimens in the direction of sample preparation and perpendicular to it. Fig. 6 shows a typical result. Specimens which were subjected to loading in the same direction in which they were prepared (θ=0), demonstrated higher strengths than the others. As can be seen, the peak strengths were obtained at low axial strain (i.e. approximately 1.5%). Both specimens showed a brittle behavior. It seems that isotropic specimens cannot be produced from this carbonate sand. The inherent properties of the material itself, such as particle shape and size, cause the material to be deposited in an anisotropic packing which is influenced by the direction of gravity.

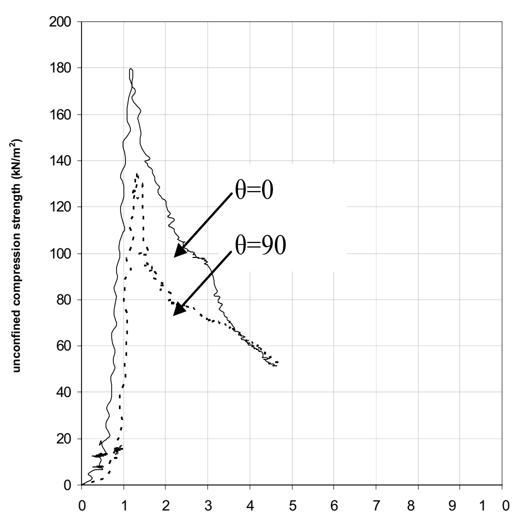

Fig. 6 Anisotropy effect using unconfined compression test results on cubic specimens.

6. p'-q Plots

Using the following formula:

$$q = c\cos\phi + p'\sin\phi$$

Or $q = a + p'\tan\alpha,\ \sin\phi = \tan\alpha$ (1)

Where; $q = (\sigma'_1 - \sigma'_3)/2$, $p' = (\sigma'_1 + \sigma'_3)/2$, c= intercept with shear stress axis and ϕ= angle of shearing resistance

The mean linear failure envelopes developed from the Mohr's circle plots are plotted in q-p' space for the non-cemented and cemented sand (Fig. 7). According to Fig. 6, the following relations are induced:

Less interlocked $\dfrac{q_{EXt.}}{q_{Com.}} = 0.75$ (2)

More interlocked $\dfrac{q_{EXt.}}{q_{Com.}} = 0.91$ (3)

It is seen that, the extension/compression shear strength ratio in less interlocked (loose) samples are lower than in more interlocked (dense) ones.

It is reasonable to expect that the samples will fail in extension rather than in compression, when tested under undrained reversed symmetrical cyclic loading. Unless the material develop large negative pore pressures excess in the extension mode.

7. The Anisotropy Effect on Extension and Compression Strength

The changes of (extension principal stress ratio)/(compression principal stress ratio) are demonstrated in Fig. 8 against confining pressure. As shown in Fig. 8, in non-cemented sand (loose and dense), the anisotropy decreases with increasing the confining pressure and reducing the initial void ratio (dense or more interlocked state). In other words, the principal stresses ratio $((\sigma'_1/\sigma'_3)_{Ext.}/(\sigma'_1/\sigma'_3)_{Comp.})$ is increased with increasing the confining pressure and reducing the initial void ratio. These are contradicting with the findings of Oda and Koishikawa (1977). These researchers stated that the anisotropy of sand is increased with decreasing of its void ratio and applied confining pressure. Perhaps, this contradiction is due to especial shape and crushability potential of the used carbonate sand. Because, increasing the confining pressure increases the particles breakage and reduces their slenderness and flatness ratio. It is noted that the mentioned researcher stated that the anisotropy is increased with the increase of flatness of particles.

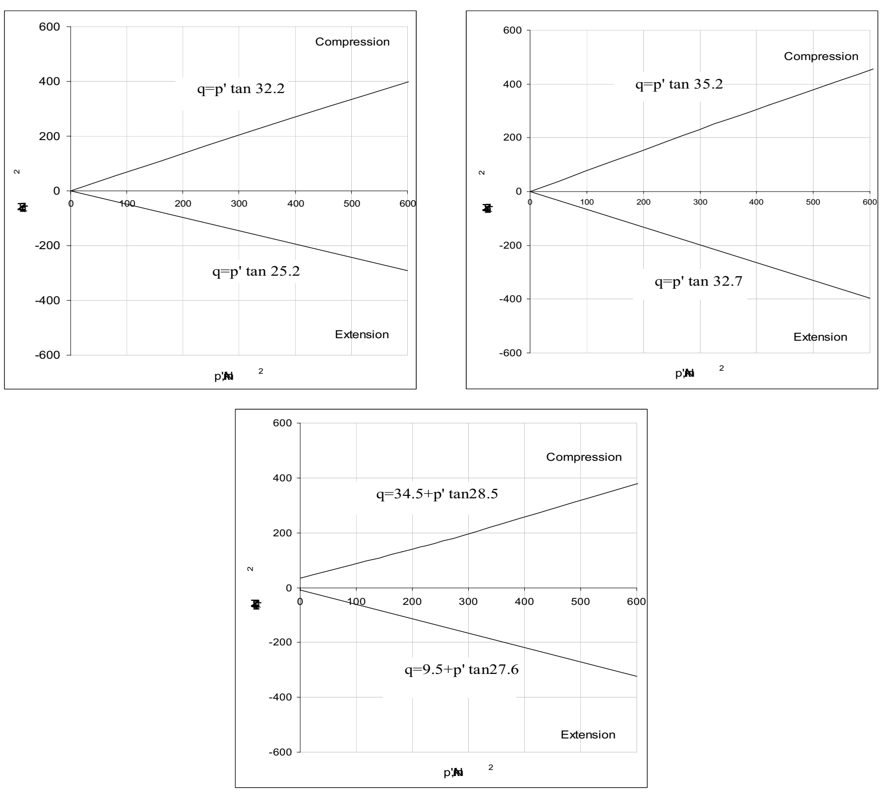

Fig. 7 Failure envelopes in compression and extension for: (a) less interlocked, (b) more interlocked, (c)

It seems that, increasing the confining pressure reduces the anisotropy in cemented sand. In other words, the safe remained bonds during isotropic compression dominate the initial anisotropy in low confining pressure (100kPa) and the fractured bonds particles during isotropic compression in high confining pressure (300kPa) behave like none cemented sand.

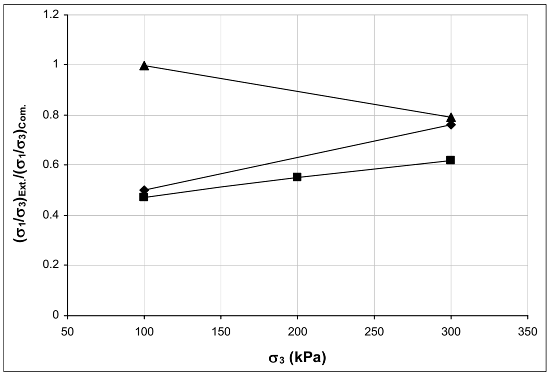

Fig. 8 The changes of (extension principal stress ratio/compression principal stress ratio) against confining pressure.

8. Conclusion

Based on the monotonic compression and extension tests on North Cornwall Rock" beach carbonate sand it is concluded that:

1. The produced fabric during sample preparation is some anisotropic considering of platy and needle shapes of sand grains.
2. The anisotropic fabric results in lower extension shear strength in comparison with compression one due to different stress-strain behavior, mechanism of shearing and particle breakage value.

The anisotropy is greater in loose state in comparison with dense one. However, the anisotropy is decreased with increasing the confining pressure. In cemented sand, the anisotropy is lower than in non-cemented sand and increasing the confining pressure increases the anisotropy.

References

[1] Airey, D. W., Randolph, M. F. , and Hyden, A.M. (1988), "The Strength and Stiffness of Two Calcareous Sands", Proc., Int. Conf. Calcareous Sediments, Perth, Western Australia, ISSMFE, Vol. 1, pp. 43-50.

[2] Andrei, V. A., and Poul V. L. (2003), "Effects of Cross Anisotropy on Three-Dimensional Behavior of Sand. I: Stress Strain Behavior and Shear Banding", J. Engrg. Mech., Vol. 129, No. 2, pp. 160-166.

[3] Arthour J. R. F., and Menzies B. K. (1972), "Inherent Anisotropy in a sand", Geotechnique, Vol. 22, No. 1, pp. 115-128.

[4] Coop, M. R. and Atkinson, J. H. (1993), "The Mechanics of Cemented Carbonate Sands", Geotechnique, Vol. 43, No. 1, pp. 53-67.

[5] Golightly, C. R. and Hyde, A. F. L. (1988), "Some Fundamental Properties of Carbonate Sands", Proc. Ist Inc. Conf. on Calcareous Sediments, Perth, Australia. Vol. 1, pp. 69-78.

[6] Fookes, P. G. and Higginbottom, I. E. (1975), "The Classification or Description of Near Shore Carbonate Sediments for Engineering Purposes", Geotechnique, Vol. 25. pp. 406-411.

[7] Ismail, M. A., Joer, H. A., Merit, A., and Randolph, M. F. (2002), "Cementation of Porous Material Using Calcite", Geotechnique, Vol. 52, No. 5, pp. 313-324.

[8] Ismail, M. A., Joer, H.A., Merit, A. and Randolph, M. F. (2000b), "Sample Preparation Technique for Artificially Cemented Soils", ASTM, Geotec. Test. J., Vol. 23, No.2, pp. 171-177.

[9] Ismail, M. H., Joer, H. A., Sim, W. H., and Randolph, M. F., (2002), "Effect of Cement Type on Shear Behavior Of Cemented Carbonate Soil" J. Geotech. Geoenviron. Eng., Vol. 128, No. 6, pp. 520-529.

[10] Lam, W. K. and Tatsuka, F. (1988), "Effects of Initial Anisotropic Fabric and σ_2 on Strength and Deformation Characteristics of Sand", Soils and Foundations, Vol. 28, No.1, pp. 89-106.

[11] McClelland, B. (1988), "Calcareous Sediments: An Engineering Enigma.", Proc. 1st Int. Conf. on Calcareous Sediments, Perth, Australia. Vol. 2. pp. 777-784.

[12] Menzies, B. K. (1986), "A Computer Controlled Hydraulic Triaxial testing System", Advanced Triaxial Testing of Soil and Rock, ASTM STP 977, Philadelphia, pp. 82-94.

[13] Shambhu, S., Sharma and M. Fahey (2006), "Evaluation of cyclic shear strength of two Carbonate Cemented Soils", J. Geotech. Geoenviron. Eng., Vol. 129, No. 7, pp. 608-618.

[14] Oda, M. and Koishikava, I., (1977), "Anisotropic Fabric of Sands", Proc. of the Ninth Inter. Conf. on Soil Mechanic and Foundation Eng., Okyo, pp. 235-238.

[15] Muhunthan B., Masad E. Assaad A., (2000), "Measurement of Uniformity and Anisotropy in Granular Materials", Geotechnical Testing Journal, Vol. 23, No.4.

[16] Symes, M. J. R., Gens, A., and Hight, D. W., (1984), "Undrained Anisotropy and Principal Stress Rotation in Saturated Sand", Geotechnique, Vol.34, No. 1, pp. 11-27.