A Carbonate Sand Particle Crushing under Monotonic Loading

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Abstract: *The unique behaviour of carbonate materials under shear loading has stimulated in investigating of their geological and engineering properties.*
Carbonate soils composed of calcium or other carbonates and most abundant in tropical marine environments are of interest from geotechnical view, especially for offshore engineers engaged with Fossil-based fuel exploitation. This was initiated in the early 1960's, when the first offshore borings in the Persian Gulf identified layers of calcarenite and thick layers of sand containing visible shell fragments.
For the purpose of exploiting gas and oil resources in hot and temperate climates (e.g. Persian Gulf) off-shore structures have been placed on carbonate soils. The carbonate sediments are high crushable compared with low crushable sediments such as quartzic soils.
To examine the crushability of these problematic sediments a series of monotonic compression, extension and post-cyclic triaxial tests under different densities and confining pressures was carried out to study the crushing behaviour of "Rock" carbonate sand obtained from Cornwall, England.
It was shown that crushing coefficient decreases with increasing in maximum principal effective stress ratio for both loose and dense states. It seems that for skeletal carbonate sand maximum and minimum dry densities will be changed during shearing loading. In other words, even though the sample has experienced an increase in density, it may also have experienced a reduction in relative density.

Key words: *carbonate sand, crushing, drained monotonic, post-cyclic*

1- Introduction

Carbonate sands are classified as crushable soils. This property is mainly due to the presence of intraparticle voids within the body of carbonate particles. Datta et al. (1979), Chaney et al. (1982) and Nauroy et al. (1988) showed that the presence of intraparticle makes an important contribution to particle crushing and the compressibility of such sediments. The carbonate sand compresses approximately three times more due to the collapse of intraparticle voids as well as the fracture of thin shelly platey fragments.
Consequently the interparticle fluid could be water whereas the intraparticle fluid could be gas (e.g. air). Intraparticle porosity is defined as:

$$n_{intra} = \frac{\text{volume of intraparticle voids}}{\text{total volume}} \qquad \text{eqn. 1}$$

An intraparticle void is a void entirely enclosed within a particle and therefore the fluid within is isolated from the free fluid in the interparticle voids.
Values of n_{intra} up to 20% have been reported by Nauroy and Le Tirant (1983) for calcareous sand. Coral and algal sands have high values of n_{intra} typically over 10%, whilst molluscan sands usually exhibit values of between 2 and 10%. Non-skeletal sands would be expected to have little intra-particle porosity, and siliceous sands none at all. Table 1 shows values of n_{intra} for three carbonate sands reported by Golightly and Hyde (1988) using the above relationship.

Table 1. n_{intra} for three carbonate sands reported by Golightly and Hyde (1988).

Location	Grain characteristic	Grain size	n_{intra}%
Dog's Bay, West Coast of Ireland	Skeletal mollusc fragments with a small quantity of formaminiferal grains	Very angular platy, medium	4-6
Ballyconneely, West Coast of Ireland	Large amount of branches and fragments of calcareous red alga	Irregular shape, subrounded, coarse	7-15
Bombay mix, Western Indian Continental Shelf	Mixture of ooids, pisoliths, mollusc fragments and formaminifera with trrigenous components	Rounded and subrounded, medium	3-7

The intraparticle voids ratio is not affected by grain packing, and therefore this parameter is classified as a physical property of the soil. That is not the case for the interparticle voids ratio which is a mechanical property of the soil and is affected by grain packing.

The presence of intraparticle voids increases the weakness under applied pressures. When direct and shear stresses are mobilised at interparticle contact points, the particles act as a "micro particulate material". Each particle contains thin walls separating fine voids. The interparticle contact point stresses are transmitted through these thin walls, which cause either particle fracture or void compression.

2- Methods to measure particle crushing

Particle crushing has been of interest for several decades to many researchers involved in testing soils under different pressures (e.g. Terzaghi and Peck, 1948; Lee and Farhoomand, 1967,; Datta et al., 1979; Bopp, 1994). This factor has great influence on the behaviour of soil.

There are different approaches to quantify particle crushing. Among them the commonly accepted method of measuring crushing in carbonate sands is that proposed by Datta et al. (1979). A crushing coefficient (C_{crush}) is suggested, where;

$$C_{crush} = \frac{\text{\% of particles finer than } D_{10} \text{ of original soil after being subjected to stress}}{\text{\% of particles of original soil finer than } D_{10} \text{ of original soil}}$$

eqn.2

By definition, the denominator has the value 10%. This definition of crushing is considered by various researchers (e.g. Golightly, 1989) to be suitable for use with carbonate sands where a measurable amount of crushing occurs.

3- Background

Datta et al. (1979) studied the crushing response of four types of carbonate sands under isotropic consolidation and shear loading (25% axial strain). They reported that the onset of crushing is related to a decrease in the maximum principal effective stress ratio $(\sigma_1, \sigma_3)_{max}$. They found that crushing increases with increase in confinement both under isotropic and shearing conditions, but is significantly less in isotropic compression, where confining pressures up to 800 kPa produce no evidence of crushing. They also quoted for carbonate soils from west coast of India crushing coefficient of 1-3 from oedometer tests under pressures 800-10000 kPa and1.2-7 after shear. The coefficient of crushing increases for carbonate soils whose main constituents are thin-walled particles.

The tests of Datta et al. (1979) and Golightly (1989) on different carbonate soils also showed that crushing during shearing altered the behaviour of dense carbonate sands from that of a dilatant brittle material under low (100 kPa) effective confining pressure to a more contractive, plastic material, under high effective (>1MPa) confining pressures.

In a further paper, Datta et al. (1980) explained that crushing depends essentially on the plastic strain developed in the soil and is not influenced by the type of loading (e.g. whether static or cyclic). Datta et al. (1979) also tried to relate crushing with the shear strength of carbonate soils. The empirical relationship proposed by the authors is given below. The power factor (in this case -0.6) depends on the type of material and its initial voids ratio;

$$\frac{K}{K_1} = (C_{crush})^{-0.6} \qquad \text{eqn. 3}$$

Where;

K= maximum principal effective stress ratio (σ_1, σ_3) K1= value of K corresponding to no crushing (i.e. C_{crush}=1) and can be any value less than K

C_{crush}= crushing coefficient

Hull et al. (1988) studied crushing in six off-shore Australian carbonate sands with different initial densities and under confining pressures ranging from 50 kPa to 400 kPa. The authors observed that under isotropic consolidated drained conditions, volumetric strain at peak stress increases with crushing coefficient and a non-linear relation can be found between them.

Evans (1987) and Coop (1990) showed that for non-cemented biogenic carbonate sands from Dog's Bay most particle crushing resulted from isotropic compression and that the application of shear stress caused little additional degradation. This contradicts the Datta et al. (1979) findings, but may be attributed to the different type of soils the authors used.

Salehzadeh and Ghazanfari (2004) studying on carbonate sand demonstrated higher particle crushing for loose sample than dense sample under monotonic drained tests. It was shown that crushing coefficient decreases with increasing in maximum principal effective stress ratio for both states.

To summarise, it can be concluded that compared with siliceous sands, crushing results in:

- Reduction in the maximum principal effective stress ratio (σ_1, σ_3),
- Alteration of the volume change behaviour from dilatant to contractive,
- Change in the stress-strain relationship from brittle softening to plastic hardening
- Increase in volumetric strain (ε_v) at peak stress,

- increase in the failure strain,,

Also particles crushing increases with:
- increasing quantity of intraparticle voids,
- increasing proportion of thin-wall-plate-like shell fragments,
- increasing angularity of particles,
- increasing size of angular particles,
- decreasing mineral hardness,
- increasing confining pressure,
- increasing deviator stress level,
- decreasing fine fraction of the soil.

4- Material tested

The non-cemented Rock carbonate sand used in this research was predominantly of biogenic origin of recent sediments.

A summary of some physical characteristics of this material is listed in Table 2.

Table 2. A summary of some physical characteristics of material tested.

Calcite and aragonite	most prevalent carbonate minerals
Skeletal spongy shaped	carbonate particles complexity
4%	intraparticle porosity
2.72	mean solid specific gravity
90 %	mean carbonate content
0.125	D_{10}(mm)
0.17	D_{30}(mm)
0.21	D_{50}(mm)
0.23	D_{60}(mm)
1.84	coefficient of uniformity
1	coefficient of curvity
40°	standing angle of repose (ϕ_{cv})
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 few coarse particles	Classification

5- Sample conditioning and testing

The samples were set-up, tested and the variables measured in a routine discipline. Tests on samples with relative densities in the ranges 34-39% were selected as representing loose behaviour and 80-90% for dense.

It seems that for skeletal carbonate sand maximum and minimum dry densities will be changed during shearing loading. Therefore, if the relative density were calculated on the basis of the unknown post-cyclic limiting voids ratios, it is probable that this value would be less than the initial value. In other words, even though the sample has experienced an increase in density, it may also have experienced a reduction in relative density.

The limiting value for the back pressure of 220 kPa was determined by experience. Using this method B values of 0.95 or more were obtained. Achieving greater B values is very difficult and time consuming.

Samples were consolidated isotropically to effective confining pressures ranging from 50 to 500 kPa.

A programme to conduct monotonic compression triaxial, extension and post-cyclic tests on loose and dense samples was planned and some typical tests were selected and analysed, which are listed in Table 3.

6- Crushing analysis

It is difficult to sieve a crushable soil using a mechanical procedure without causing further crushing. It is possible that during sieving, fragile needle and platy shaped particles fracture altering the particle size distribution. Despite this, the samples were oven-dried and sieved and any resulting particle crushing is assumed to be minimal.

6-1- Post-consolidation

In specific volume-mean effective stress space, it is found that for the range of the applied isotropic confining pressure (up to 500 kPa), no signs of plastic yielding in the response of the samples are observed. Generally these samples behave in a very incompressible manner. The response is a line almost parallel to the mean effective stress axis.

The loose sand shows some evidence of yield at 500 kPa confining pressure. This trend might be due the onset of crushing of the loosely packed particles. This suggests that the yield point of the loose sand may be reached at a low mean effective stress (i.e. 500 kPa). It may be concluded that prior to yield, all samples behave like elastic overconsolidated soils. Therefore, no sign of crushing can be reported.

Table 3 . List of tests

Soil state	Test No.	Dr; %	Confining Pressure; kPa	Notation
Monotonic /	9	44	500	9/MCOL-500
Compression / Loose	12	40	100	12/MCOL-100
	53	42	300	53/MCOL-300
Monotonic /	60	80	100	60/MCOD-100
Compression /	52	90	300	52/MCOD-300
Dense	14	86	500	14/MCOD-500
Monotonic /	87	48	100	87/MEXL-100
Extension /	88	52	200	88/MEXL-200
Loose	86	50	300	86/MEXL-300
Monotonic /	98	96	100	76/MEXD-100
Extension /	97	95	200	63/MEXD-200
dense	96	94	300	62/MEXD-300
Monotonic /	37	49	100	37/MPL-0.10
Post-cyclic/	41	49	100	41/MPL-0.125
Loose	39	48	100	39/MPL-0.15
	35	51	100	35/MPL-0.175
Monotonic /	30	80	100	30/MPD-0.20
Post-cyclic/	47	85	100	47/MPD-0.25
dense	28	79	100	28/MPD-0.30

6-2- Post-compression shearing phase

Figure 1 shows an example of carbonate sand particles after testing. There is evidence of severe particle crushing especially in the loose sample.

Figure 2 and Figure 3 show the particle size distribution curves for various samples at the end of each test. Both loose and dense sands demonstrate particle crushing. Crushing coefficients were calculated for non-cemented sand using Datta et al.'s (1979) method, and are plotted against the confining pressure in Figure 4. The loose samples demonstrate more crushing than the dense samples, for a given confining pressure.

Figure 5 illustrates the variation of the coefficient of crushing with respect to the peak principal stress ratio. It is found that the coefficient of crushing increases with decreasing peak principal stress ratio. Linearity for the dense sand is better than for the loose sand. The values of the peak principal stress ratio related to the non-crushing condition (i.e. C_{crush}=1) have been determined by extrapolation. Values of 6.8 and 15, for loose and dense sands, respectively, are found. These values can be used to estimate the coefficient of crushing occurring under a specified effective stress

ratio, K.
Considering K as the maximum principal effective stress ratio under a given confining pressure and K1 as the maximum principal effective stress ratio for no crushing, Figure 6 can be plotted. There is a reasonably linear relationship between the normalised maximum principal effective stress ratio, and the coefficient of crushing when plotted on a log-log scale. Mathematically, it can be defined as :

$$K/K_1=(C_{crush})^{-0.834} \quad \text{eqn. 4}$$

This equation resembles that of Datta et al. (1979), but their exponent was –0.6 and it can be used to predict C_{crush}.

6-3- Post-extension shearing phase

Using D10 as a basis for comparison, Table 4 Shows the crushing coefficient (C_{crush}). In extension, unincreased crushing ($C_{crush} \cong 1.1$) occurred. Constant values of C_{crush} ($\cong 1.1$) in extension, which are deduced at the end of the tests, show that increasing the confining pressure has no effect on the crushing.
In compression both loose and dense specimens showed increased crushing with confining pressure with loose showing the greater effect (C_{crush}=4.62 (loose) and 2.5 (dense) for σ_o=100 kPa).

6-4- Post-cyclic shearing phase

Results of the post-cyclic particle grading tests on the loose and dense samples are plotted in Figures 9 and 10. Sieve analyses of the original untested material and material subjected to monotonic testing only are also included.

The coefficient of crushing (C_{crush}), based on the definition of Datta et al. (1979), is shown for monotonic post-cyclic loading in Table 5. For cyclic loading on loose sands it is seen that the coefficients of crushing increase with cyclic stress ratio (1.20 to 1.42 for cyclic stress ratio of 0.125 to 0.175) and are less than the coefficient of crushing for the non-uncycled monotonic sample (C_{crush}=1.50). This implies that cycling has a lesser crushing effect than initial monotonic loading. On the other hand, it is seen that coefficient of crushing is constant for post-cyclic monotonic loading (C_{crush}=1.60). This implies that with increasing cyclic stress ratio the effect of particle crushing during subsequent monotonic loading is reduced. In other words, samples which have experienced greater cyclic stress ratios exhibit less crushability when subjected to monotonic loading. It should be noted that the coefficients of crushing of 1.50 and 1.60 were obtained at large strains (>15%), whereas the coefficients of crushing of 1.20 to 1.42 were obtained at 5% cyclic strain. Overall, a uniform repeatable behaviour in terms of crushing is observed.

For dense samples, the coefficient of crushing (C_{crush}) after cyclic loading increases from 1.41 to 1.65 when the cyclic stress ratio is increased from 0.20 to 0.30, respectively. The coefficient of crushing (C_{crush}) for test 47/CYD-0.25 is comparable with that of the non-cycled monotonic loading test (60/MCOD-100) which means that cycling appears to have the same effect on particle crushing. Again, the coefficients of crushing for non-cycled monotonic loading test andpost-cyclic monotonic test were obtained at large strains (>15%), whereas the coefficients of crushing of 1.41 to 1.65 were obtained at 5% cyclic strain.

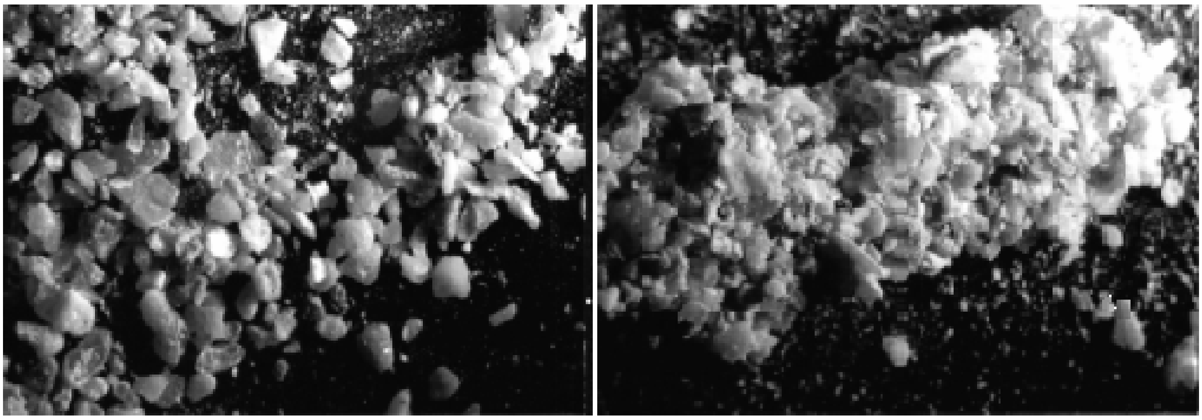

Figure 1. Example of carbonate sand particles after testing; (a) dense sand, (b) loose sand.

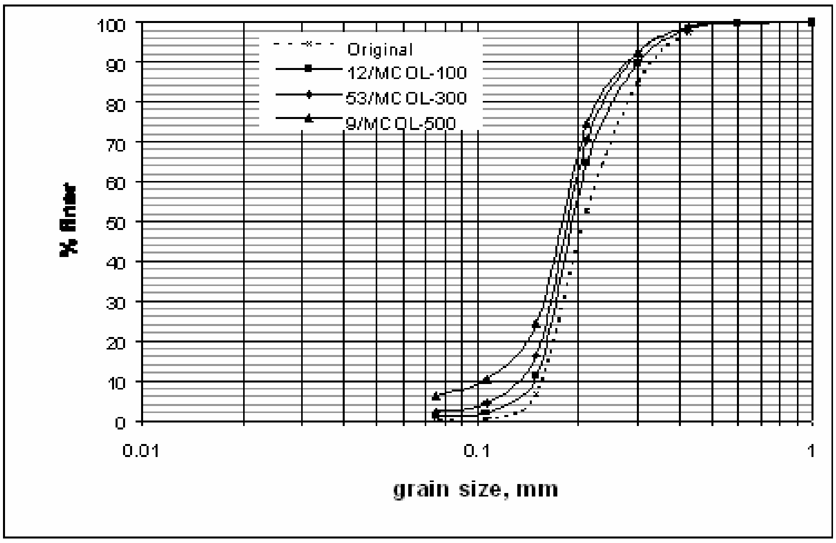

Figure 2. End of test particle size distribution curves for loose sand after monotonic compression triaxial loading.

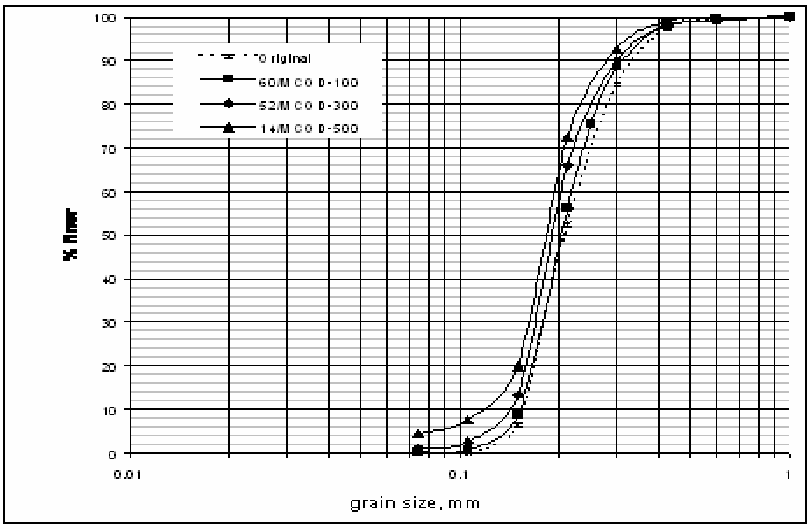

Figure 3. End of test particle size distribution curves for dense sand after monotonic compression triaxial loading

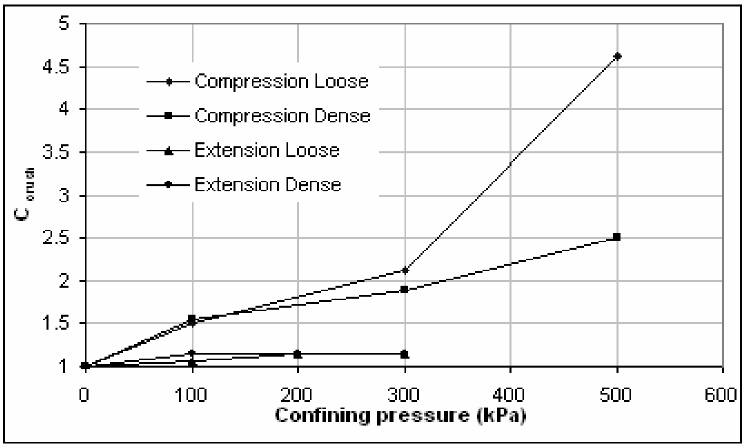

Figure 4. Crushing coefficient versus confining pressure using Datta et al.'s (1979) method for both loose and dense sands

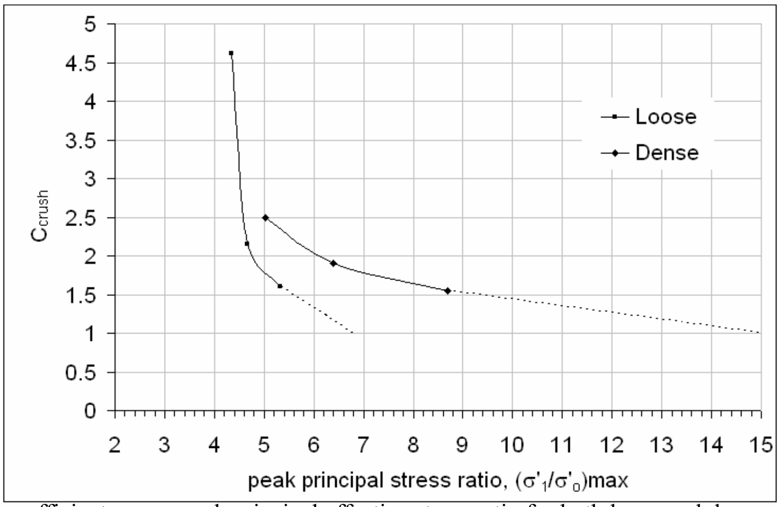

Figure 5. Crushing coefficient versus peak principal effective stress ratio for both loose and dense sands under compression.

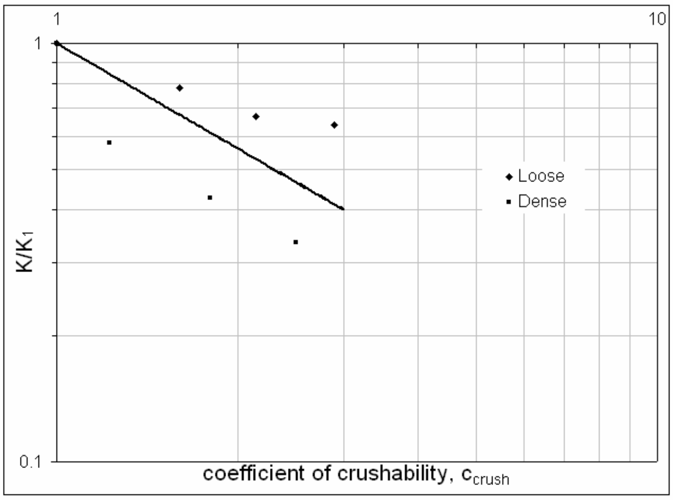

Figure 6. Linear relationship between dimensionless normalised maximum principal effective stress ratio and coefficient of crushing on a log-log plot for both loose and dense sands under compression.

Table 4. Coefficient of crushing for

Mode of testing	Notaion	C_{crush}
Monotonic Compression	12/MCOL-100	1.50
	53/MCOL-300	2.12
	9/MCOL-500	4.62
	60/MCOD-100	1.55
	52/MCOD-300	1.89
	14/MCOD-500	2.50
Monotonic Extension	87/MEXL-100	1.07
	88/MEXL -200	1.14
	86/MEXL -300	1.14
	98/MEXD-100	1.14
	97/MEXD -200	1.14
	96/MEXD -300	1.14

Table 5. Coefficient of crushing obtained from drained monotonic post-cyclic triaxial monotonic tests compression tests (σ'o=100 kPa).

state	test number	D_r, % after initial consolidation	C_{crush} after first monotonic test	C_{crush} (after cyclic test)	C_{crush} (after post-cyclic monotonic test)
Loose	12/MCOL-100	40	1.50 (end of test)	-	-
	117/CYL-0.125	42		1.20	
	56/CYL-0.15	39		1.30	
	16/CYL-0.175	35		1.42	
	41/MPL-0.125	49	-		1.60
	39/MPL-0.15	48	-		1.60
	35/MPL-0.175	51	-		1.60
dense	60/MCOD-100	80	1.50 (end of test)	-	-
	49/CYD-0.20	80		1.41	
	46/CYD-0.25	80		1.50	
	48/CYD-0.30	80		1.65	
	30/MPD-0.20	80	-		1.20
	47/MPD-0.25	85	-		1.51
	26/MPD-0.30	86	-		1.40

The results obtained for coefficient of crushing following post-cyclic monotonic loading are not consistent and compared with the loose samples a more random behaviour in terms of crushing is observed.

7- Conclusions

The following conclusions respect to the crushability of "Rock" carbonate sand can be made:

1- "Rock" Carbonate sand consists of 2-dimensional particles, either platy or needle shaped with sharp edges tending to produce point contacts, compared with 3-dimensional shaped siliceous sands, which are mostly angular to round.

2- It is difficult to sieve this crushable soil using a mechanical procedure without causing further crushing. It is possible that during sieving, fragile needle and platy shaped particles may fracture and the particle size distribution altered.

3- Under isotropic consolidation, "Rock" carbonate particles probably fracture due to buckling, however, under shearing; particles probably fracture due to bending. Therefore, significantly more crushing occurs during shearing than during isotropic compression. This needs
further investigation to study the mechanism and factors effecting crushing of particles (i.e. buckling or bending).

4- Crushing at high confining pressures for both loose and dense sands questions the φ_{cv} concept for "Rock" carbonate sand and the conventional model for the ultimate state of constant volume shearing may be invalid.

5- Using D10 as a basis for comparison, minimal crushing occurs during shearing under extension. While in compression both loose and dense samples show increased crushing with confining pressure.

6- In the loose samples the coefficient of crushing measured after the cyclic test increases with cyclic stress ratio and is less than the coefficient of crushing measured at the end of a non-cycled monotonic test. This implies that cycling has a lesser crushing effect than initial monotonic loading.

7- In the loose samples at higher cyclic stress ratios (e.g. 0.125 to 0.175), the quantity of particle crushing during cycling is greater than at lower cyclic stress ratios (e.g. 0.10).

8- In the loose samples the coefficient of crushing is constant for post-cyclic monotonic testing. This implies that with increasing cyclic stress ratio, the effect of particle crushing during subsequent monotonic loading is reduced. In other words, samples which have experienced greater cyclic stress ratios exhibit less crushability when subjected to monotonic loading.

9- The coefficient of crushing measured after cyclic loading increases with cyclic stress ratio for dense sand.

10- During cycling, crushing produces a new particle size distribution with different limiting voids ratios. This questions the reliability of using relative density values to assess quantitatively the effect of cyclic loading on crushing.

11- It is possible that there is a threshold cyclic stress ratio when severe particle crushing is activated; the quantity of crushing appears to be independent of the cyclic stress ratio, once in excess of this threshold value.

12- It seems that for skeletal "Rock" carbonate sand maximum and minimum dry densities will be changed during shearing loading. Therefore, if the relative density were calculated on the basis of the unknown post-cyclic limiting voids ratios, it is probable that this value would be less than the initial value. In other words, even though the sample has experienced an increase in density, it may also have experienced a reduction in relative density.

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