# MEDIUM DENSE NON-CEMENTED CARBONATE SAND UNDER REVERSED CYCLIC LOADING

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***Abstract:*** *Carbonate materials are mostly found in tropical areas, where exploiting gas and oil resources are of high concern. Their unique behavior under shear loading first was recognised during oil resources investigations in the Persian Gulf. Off-shore structures have been placed on carbonate soils which are highly crushable.During storms cyclic loading imposes on the bases of structures lied down on seabed. Cyclic loading, therefore, may trigger liquefaction phenomenon which leads to soil collapse and a catostrophic event. Therefore, stability of these expensive structures need to be investigated. To this aim carbonate sand in medium dense to medium dense state was considered and its response under varied cyclic shear stress ratio was studied.*

## 1. Introduction

International interest in carbonate soils grew 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. However, the high carbonate content of these sands was not recognised at first.

McClelland (1988) quoted that the first awakening to the unusual behaviour of these materials came from pile driving operations during the construction of a platform for Lavan Petroleum in Iranian waters in 1968. The major occurrence of carbonate sediments is usually between 30$^{o}$N and 30$^{o}$S latitudes (Rodgers, 1957; Murff, 1987). However, they are not completely restricted to these latitudes (Chave et al., 1962; Lees & Buller, 1972).

Individual bioclastic grains are crushable because of their high angularity and the weakness of carbonate minerals. However, non-skeletal detrital grains are generally sub-rounded in shape and less brittle due to an absence of intra-particle voids and a strong non-carbonate nucleus. As a result, the mechanical behavior of sand masses made up of these two particle types could be very different under shear loading.

In the lifetime of an offshore structure it is reasonable to expect that a loading of $10^8$ cycles, equivalent to 30 years at 0.1 Hz, will be undergone by a structure. Offshore foundations are generally subjected to a number of storms with intermittent periods in which consolidation may occur.

The effects of waves on such sediments and on offshore structures and their foundations stimulated investigation of the cyclic shear behaviour of non-cemented and cemented carbonate soils. Various researchers have used the cyclic triaxial test for this purpose.

Some investigators studied the response of carbonate soils at high stress levels and relatively small numbers of cycles (e.g. Randolph et al. 1999). Others focused on the fatigue response of soils, under a large number of cycles at low stress levels. Usually tests are stress controlled with a constant cyclic shear stress amplitude being applied.

The cyclic shear stress ratio is defined as either a percentage of the static monotonic strength, $q_{cyc}/q_{peak}$ (usually applied to cohesive soils) or as percentage of the final isotropic consolidation phase $q/\sigma'_o$ or $\tau/\sigma'_o$ (usually applied to granular soils). For simplicity failure is considered to be the generation of large cyclic strains.

The works carried out by some researchers using the cyclic triaxial test to investigate the response of medium dense or relatively medium dense non-cemented and cemented carbonate sand are described in the following sections.

## 2. Background

Airey & Fahey (1991) working on medium dense and dense sands from the North-West Shelf of Western Australia with cycle stress ratios ranging from $q/\sigma'_o$= 0.149 to 0.889 and frequencies varying from 0.02 to 0.1 Hz, found that the samples of carbonate sediments always appeared to be closer to failure in extension than in compression, i.e. large extension strains developed. The authors did not comment as to whether this observation can be attributed to a material property as opposed to being a function of the testing technique. It was observed that prior to failure, the samples experienced a characteristic (phase transformation) state. This characteristic state is the point at which the rate of change of strain cycles accelerates towards failure. Airey and Fahey (1991) define the characteristic state as a linear envelop in q-p’ space lying below the ultimate critical state line.

Salleh (1992) working on sand from Dog’s Bay Ireland with medium density and with stress ratios ($q_{cyc}/q_{peak}$) from 0.25 to 0.5 and a frequency of 0.05 Hz., showed that for a given density and confining pressure (ranging from 50 kPa to 600 kPa), the number of cycles required to cause failure, decreased as the magnitude of cyclic deviator stress amplitude increased.

When the cyclic strength of carbonate sand was compared with that of siliceous sand (e.g., Seed & Lee, 1966), it was observed that under identical test conditions including relative density, the number of cycles to reach failure was greater for carbonate sands. The difference in the number of cycles to failure decreased with increasing stress ratio. This can be attributed to the effect of crushing which is dominant at high stress ratios, compared with the dominant effect of interlocking at low stress ratios. When samples are experiencing high stress ratios, particle interlocking reduces due to particle crushing and the shape of the grains changes from angular or semi-angular to semi-rounded siliceous-like grains and the general pattern of the behaviour of carbonate sand converges to the non-crushable siliceous response.

Salleh reported that for a given cyclic stress ratio, the higher the confining pressure, the greater is the number of stress cycles required to reach failure.

Hyodo et al., 1993 and 1994, working on isotropically consolidated samples of medium density sand from Dog’s Bay, Ireland with a stress ratio ($q/\sigma'_o$, $\sigma'_o$ as confining pressure) ranging from 0.4 to 1.2 and a frequency of 0.1Hz., observed liquefaction failure when the pore pressure reached the confining pressure accompanied by large extensive cyclic axial strains. Failure was defined by a 5% double amplitude axial strain limit. The authors obviously observed that the number of cycles required for failure increased with decreasing cyclic stress ratio. They noticed that confining pressure

**Table 1-** Summary of some aspects of soil tested.

| | |
|---|---|
| most prevalent carbonate minerals | Calcite and aragonite |
| carbonate particles complexity | Skeletal spongy shaped |
| intraparticle porosity | 4% |
| mean solid specific gravity | 2.72 |
| mean carbonate content | 90 % |
| $D_{10}$(mm) | 0.125 |
| $D_{30}$(mm) | 0.17 |
| $D_{50}$(mm) | 0.21 |
| $D_{60}$(mm) | 0.23 |
| coefficient of uniformity | 1.84 |
| coefficient of curvity | 1 |
| standing angle of repose ($\varphi_{cv}$) | 40° |
| classification | well-graded, fine-medium sand (fine fraction=40%, medium fraction=50%) with a few coarse particles |

(ranging from 100 kPa to 500 kPa) does not affect the cyclic strength of isotropic consolidated samples. This contradicts the results provided by Salleh on the same material and this might be attributed to the higher stress ratios or higher frequencies employed by Hyodo et al. (1993 & 1994) or even sample size and equipment effects. As explained earlier, particle crushing is dominant under higher stress ratios which is not the case at low stress ratios.

## 3. Material

A 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. The physical characteristics of this material are reported by Salehzadeh (2000). A summary of some aspects is as shown in Table1.

## 4. Data recording

Using a p.c. data derived from the calibrated devices (i.e. load cell, LVDT and PWP transducers) were monitored and the measuring changes in the cyclic loading, displacements and pore water pressure were recorded.

Two Geotechnical Digital System (GDS) controllers used for generating and measuring both water pressure and volume change (Menzies, 1986). When both GDS's were compared they showed good agreement displaying ± 1kPa changes in the ranges of 0 to 700 kPa pressure. Resolutions of ±1 kPa for pressure and ±1mm$^3$ for volume change can be attained.

Data Acquisition and Data Storage were included of an IBM-compatible PC/XT machine, capable of converting input signals to bytes through a DASH-16 analogue to digital converter (ADC) (manufactured by the Metrabyte Corporation) and stored on hard disc.

## 5. Sample Preparation and Testing

The method of sample setting up as well as

**Table 2-** Selected tests for analysis.

| Test No. | Dr; % | Confining Pressure; kPa | Cyclic stress Ratio, $\tau/\sigma'_{o*}$ | Notation |
|---|---|---|---|---|
| 42 | 62 | 100 | 0.1 | 42/CYL-0.10 |
| 117 | 42 | 100 | 0.125 | 117/CYL-0.125 |
| 56 | 39 | 100 | 0.15 | 56/CYL-0.15 |
| 16 | 35 | 100 | 0.175 | 16/CYL-0.175 |

sample testing and variables measuring was reported by Salehzadeh (2000). Tests on samples with relative densities in the ranges 30-50% were selected as representing medium dense or relatively medium dense behaviour.

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 pressure of 100 kPa.

A programme to conduct two-way cyclic triaxial tests was planned and some typical tests were selected and analysed, which are listed in Table 2.

## 6. Two-Way Reversed Cyclic Loading

The behaviour of the non-cemented carbonate sand in its medium dense state when subjected to two-way reversed cyclic loading is discussed.
Shear distortion causes irrecoverable rearrangement of the particle packing as meta-stable voids collapse. This mechanism is accelerated in the case of carbonate sands when crushing occurs at the particle contact points and small asperities break off and are then free to move into adjacent voids. This results in an increase in pore water pressure and reduction in effective stress accompanied by loss of stiffness and increase in shear strain, culminating in liquefaction and complete loss of strength.

In order to study the response of the medium dense carbonate sand under reversed cyclic loading, the results from tests 42/CYL-0.1, 117/CYL-0.125, 56/CYL-0.15 and 16/CYL-0.175 which were conducted under cyclic ratios of $\tau/\sigma'_o$ = ±0.10, ±0.125, ±0.15 and ±0.175, respectively are analyzed and the results from test 117CYL-0.125, are discussed as being typical of the material's behavior.

### 6.1. Overal pore pressure behaviour

During the generation of pore pressure, four phases were distinguished. After the first cycle a gradual reduction in the rate of pore water pressure change was observed until cycle 4. For all cyclic stress ratios this initial phase consists of relatively few cycles. Subsequently a steady state stage (cycles 4 to 75) developed during which the pore pressure ratio increases at a constant rate (i.e. 0.005 per cycle) and with cyclic stress ratio (Figure 1).

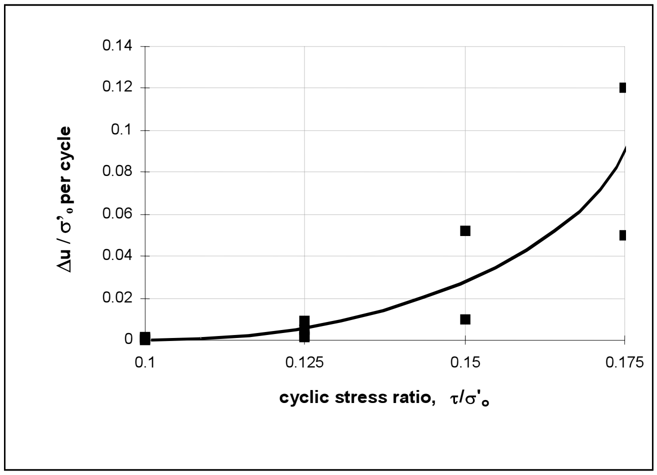


**Figure 1.** Pore water pressure ratio increase per cycle during steady state phase for medium dense carbonate sand.

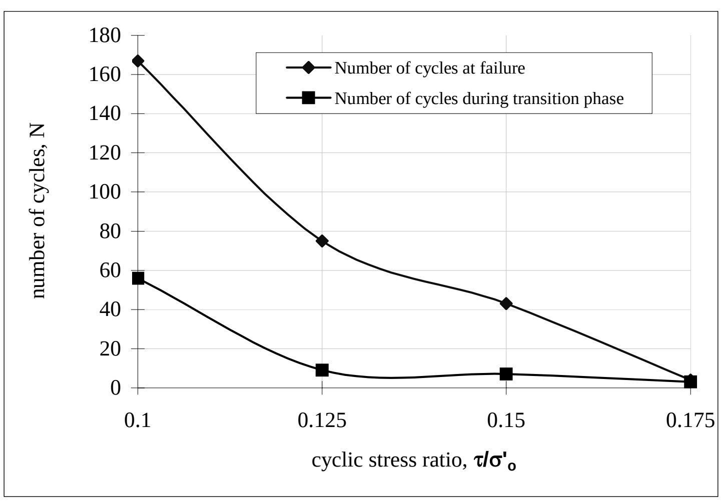


**Figure 2.** Number of cycles variation of transition and failure phases

The steady state phase was followed by a transition phase (cycles 76 to 83) when an increase in the rate of pore pressure generation was observed ending with a sudden increase to liquefaction, which can be named as the flow stage. Until the transition phase a constant pore water pressure amplitude was observed.

At the flow stage when the sand liquefied, the excess pore pressure ratio ($\Delta u/\sigma'_o$) reached unity. The condition at which liquefaction was triggered (start of the flow stage) occurred at $(\Delta u/\sigma'_o)_{mean}$=0.65 and $(\Delta u/\sigma'_o)_{min}$=0.57 (Table 3). This is significant as the specimen failed in extension. When the pore water pressure ratio reached unity, the sample lost all strength and uncontrolled deformation ($\varepsilon_a \approx 5\%$) caused the test to cease. This confirms the observation made that with medium dense sands partial and complete liquefaction occur simultaneously (Vaid & Chern, 1985).

It was seen that the number of cycles to failure decreases with the increasing of cyclic stress ratio. The parameter (b-a) (Table 3) indicates the number of cycles in the transition phase and is the difference between the two plots in Figure 2 which also decreases with increasing $\tau/\sigma'_o$.

From the first cycle to the end of the steady state stage, the material demonstrated a near rigid state and axial strains for test 117/CYL-0.125 were very small (0.04% to 0.045% double amplitude, see Table 3). Even for the cycle prior to liquefaction only a double amplitude axial strain of 0.12% was developed. This demonstrates that prior to the onset of liquefaction only a very small amount of deformation is needed to mobilise sufficient shear resistance in the soil necessary to maintain equilibrium.

At the onset of the transition stage the axial strain response drifted into the extension region which is a normal trend in two-way cyclic tests (Vaid and Chern, 1985).

It was observed that for all the selected tests the double amplitude axial strain both in the steady state and after the end of the transition phase increases with cyclic stress ratio but the values are still very small ($\leqq$0.16%) (Table 3).

### 6.2. Φ' mobilisiation

For some selected cycles during cycle 83, the Φ' mobilised response was in phase with the cyclic deviator stress. The maximum Φ' mobilised was 15$^o$ in compression and 20$^o$ in extension close to the reported $\Phi'_{peak}$=25$^o$ obtained from the monotonic extension test 81/MEXL-100 but significantly less than reported $\Phi'_{peak}$=43$^o$ from monotonic compression test 12/MCOL-100 (Salehzadeh, 2000). During cycle 84, the Φ' mobilised response was similar to cycle 83 but the maximum Φ' mobilised is 20$^o$ in extension close to the reported $\Phi'_{peak}$=25$^o$ obtained from the monotonic extension test 81/MEXL-100 (Salehzadeh, 2000). This may approve that the monotonic extension drained test is a good indicator to obtain the extension failure envelope for the carbonate sand tested under two way cyclic loading, in q-p' space.

### 6.3. Stiffness

It was observed that with increasing cycles the axial strain migrates from the compression to the extension strain region. The sample failed under large extension strain.

For the first cycle, stiffnesses were calculated using the peak to peak points in $\sigma_d$-$\varepsilon_a$ space

**Table 3.** Data obtained for medium dense sand and under two-way cyclic loading ($\sigma'_o$= 100kPa)

| Test No. | $D_r$% | $N_{ss}$ at the end of steady state | $N_f$ | b-a | At point when full liquefaction was triggered | | | | | $\varepsilon_a$ | | E(MPa) | | | Rate of change of pwp ratio in steady state phase per cycle |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | $\Delta u/\sigma'_o$ uncorrected | | | p' | q' | D.A. at cycle prior to liquefaction | D.A. at S.S. | Monotonic 12/MCOL-100 ($D_r$=40%) (c) | For first cycle in $\sigma_d$-$\varepsilon_a$ plot (peak to peak) (d) | Stiffness ratio (c)/(d) | |
| | | (a) | (b) | | min | max | mean | for $u_{min}$ | for $\Delta u_{min}$ | | | | | | |
| 42/CYL-0.10 | 62 (med) | 167 | 223 | 56 | 0.53 | 0.70 | 0.62 | 37 | -10 | 0.02 | 0.03 | 167 | 111 | 0.67 | 0.0012 |
| 117/CYL-0.125 | 42 | 75 | 84 | 9 | 0.57 | 0.73 | 0.65 | 30 | -12.5 | 0.12 | 0.045 | 159 | 96 | 0.60 | 0.005 |
| 56/CYL-0.15 | 39 | 43 | 50 | 7 | 0.54 | 0.82 | 0.68 | 31 | -15 | 0.13 | 0.08 | 160 | 92 | 0.58 | 0.008 |
| 16/CYL-0.175 | 35 | 5 | 7 | 3 | 0.33 | 0.70 | 0.52 | 49 | -17.5 | 0.16 | 0.11 | 159 | 72 | 0.45 | 0.05 |

S.S.= Steady State, D.A.= Double Amplitude

Quoted from Salehzadeh(2000)

Compression:

| | $D_r$,% | $\phi'_{peak}$ |
|---|---|---|
| 12/MCOL-100 | 40 | $43^O$ |
| 60/MCOD-100 | 80 | $53^O$ |
| 75/MCOC-100 | 44 | $41^O$ |
| 76/MCOC-100 | 44 | $39^O$ |

Extension:

| | $D_r$,% | $\phi'_{peak}$ |
|---|---|---|
| 87/MEXL-100 | 48 | $25^O$ |
| 98/MEXD-100 | 96 | $35^O$ |
| 92/MEXC-100 | 44 | $30^O$ |

(Table 3). The stiffness is also tabulated for the monotonic test 12/MCOL-100 (Salehzadeh. 2000) for the same level of deviator stress used for cyclic loading. It was observed that peak to peak stiffness decreases with cyclic stress ratio (Table 3).

However, it was seen that the monotonic stiffnesses from test 12/MCOL-100 are fairly constant for the different stress levels. This is because the maximum cyclic deviator stress of 35 kPa from test 16/CYL-100 is about 8% of monotonic deviator stress of 432 kPa from test 12/MCOL-100.

It can be seen from Table 3 that the ratio of peak to peak stiffness to monotonic stiffness decreases with increasing cyclic stress ratio (i.e. 0.67 to 0.45 for cyclic stress ratios of ±0.10 to ±0.175).

### 6.4. Stress path

Overall, the net increase in the pore water pressure resulted in migration of the undrained effective stress path towards the origin in the q-p' space (Figure 3). The mean effective stress at the end of each cycle reduces as the number of cycles increases. Liquefaction occurs when the extension stress path first touches the monotonic extension failure envelope as defined by Test 87/MEXL-100 (Salehzadeh, 2000).

It should be stated that all samples failed in extension and it seems that the monotonic compression tests are irrelevant in studying cyclic tests. It was seen that in some tests (i.e. 42/CYL-0.10) the failure envelope is not touched by stress path but sample was liquefied. This can be attributed to the non-

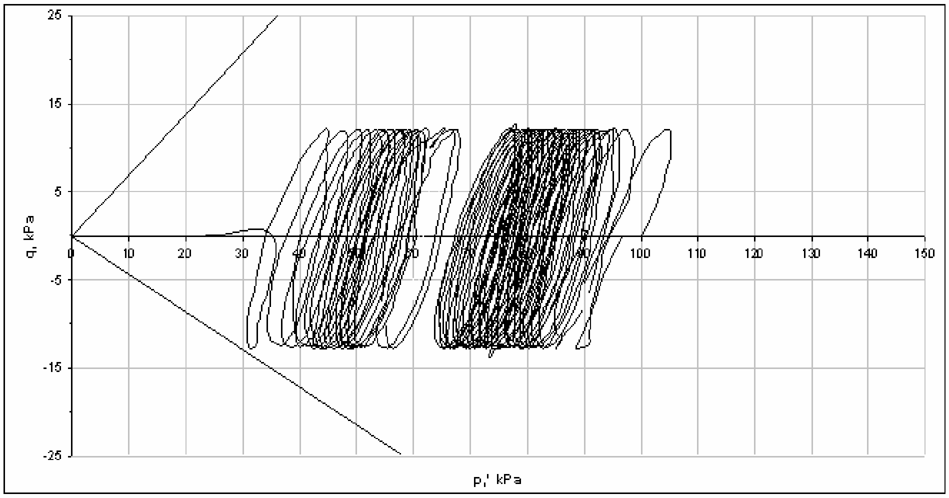


**Figure3.** Effective stress path for loose carbonate sand under a cyclic stress ratio of

uniformity of the pore water pressure distribution between the region where the sample is necking and the bottom of the sample at which the pore water pressure response was recorded. Test 56/CYL-0.15 conformed exactly with the test 117/ CYL - 0.125.

## 7. CONCLUSIONS

**1-** The mechanism of irrecoverable structured rearrangement is accelerated in the case of carbonate sands when crushing occurs at the particle contact points and small asperities break off and are then free to move into adjacent voids causing meta-stable voids to collapse. This results in an increase in pore water pressure and reduction in effective stress accompanied by loss of stiffness and increase in shear strain, culminating in liquefaction and complete loss of strength.

**2-** Four phases can be distinguished during the generation of pore pressure: an initial phase; a steady state phase with constant rate of change of pore water pressure; a transition phase; and liquefaction. Prior to the onset of liquefaction, only very small double amplitude axial strain of the order of 0.1% is sufficient to mobilise soil shear resistance necessary to maintain equilibrium.

**3-** Failure is considered to have occurred when the pore water pressure ratio reached unity. Subsequently, the sample lost all strength and uncontrolled deformation caused the test to cease. This confirmed that partial and complete liquefaction occur simultaneously.

**4-** The double amplitude axial strain both in the steady state and the end of the transition phase increases with cyclic stress ratio, however, the values are small (of the order of 0.15%).

**5-** Both the peak to peak stiffness modulus and the ratio of peak to peak stiffness to monotonic peak stiffness moduli decrease with increasing cyclic stress ratio.

**6-** Liquefaction occurs when the extension stress path first touches the monotonic extension failure envelope and failure occurs in extension. The monotonic compression failure envelope is irrelevant to two way cyclic failure.

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