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Triaxial compression tests on a crushable sand in dry and wet conditions

Essais triaxiaux sur un sable écrasable en conditions sèches et humides

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ABSTRACT A calcareous sand from the Persian Gulf is subjected to a series of dry and fully drained saturated triaxial shear tests. The samples are prepared at relative densities of 65% and either left dry or saturated. They are consolidated to confining pressures ranging from 50 to 750 kPa, and sheared until shear strains of 20%. It is shown that the stress-strain and strength characteristics of crushable sand are significantly affected by the presence of water. During shearing of wet samples, there is less dilation, the peak is postponed and a lower shear strength is reached compared to dry samples. Crushability is assessed by comparing the granulometry before and after the triaxial tests. While both dry and wet samples show breakage, the wet sand is consistently more crushable. It is stated that the higher crushability of the wet sand suppresses its dilation during shearing.

RÉSUMÉ Un sable calcaire du Golfe Persique est soumis à une série d'essais triaxiaux secs et consolidés-drainés. Les échantillons sont préparés à des densités relatives de 65% et soit laissés à sec soit saturé. Ils sont consolidés à des pressions de confinement allant de 50 à 750 kPa, et cisaillées jusqu'à contraintes de cisaillement de 20%. Il est montré que les caractéristiques de contrainte-déformation et la résistance au cisaillement du sable écrasable sont sensiblement affectés par la présence de l'eau. Pendant le cisaillage des échantillons saturés, il y a moins de dilatation, le résistance de pic est reportée et une résistance au cisaillement inférieure est atteinte par rapport aux échantillons secs. Aptitude à l'écrasement est évaluée en comparant la granulométrie avant et après les essais triaxiaux. Alors que les échantillons secs et saturés montrent également de la rupture, le sable saturé est toujours plus déformable. Il est précisé que l'aptitude à l'écrasement plus élevée de sable humide supprime la dilatation au cours de son cisaillement.

1 INTRODUCTION

In the recent decades the Gulf States have become a hub for major land reclamation projects intended for petrochemical structures, housing and leisure. New land is usually created with sand reclaimed offshore, in the warm semi-enclosed Persian Gulf. This sand consists mainly of shells, which are in fact angular, calcareous grains. The crushable nature of this sand is responsible for its specific geotechnical behaviour.

The interpretation of cone penetration resistance of crushable sand in terms of relative density is one area of increased complexity. This correlation can be established empirically in the controlled laboratory environment of calibration chambers (Jamiolkowski et al. 2003) and for silica sediments it can be captured with traditional bearing capacity theories in terms of shear angle. In calcareous sands however, the bearing capacity is lower than what is estimated from classical bearing capacity equations (Huang et al. 1999), which infer from the high friction angle of these sands a dilatant response whereas in reality it is contractant.

Indeed, crushing occurs during cone penetration, thereby progressively changing the density, the sand fabric and granulometry. The ultimate bearing resistance in calcareous sands then depends on their instantaneous state, friction angle as well as compressibility (Semple 1988). Reasonable correlations between q_c and D_r for application in crushable sands, considering both shear and compression characteristics, have been developed using spherical cavity expansion theory (e.g. Jiang & Sun, 2012).

Wang et al. (2011) performed plate load tests on calcareous sands from coral reefs of Nansha Islands and found that the bearing capacity of wet calcareous sand is only half of that of dry sand. There is no such difference in cone resistance of dry and wet silica sands and drained calibration chamber tests are simulated using dry conditions, simplifying the test set-up (Jamiolkowski et al. 2003).

Since bearing capacity is a result of shear and compression characteristics, different results for wet and dry conditions are expected in laboratory tests such as oedometer and triaxial tests. A test program was set up in the Ghent University Laboratory of Geotechnics, consisting of dry and wet compression and shear tests on a calcareous sand. The obtained mechanical parameters will be used as input for the spherical cavity expansion models designed for the cone penetration resistance of crushable sands.

One-dimensional compression tests on S2-sand (Wils et al. 2014) have shown the increased compressibility of wet calcareous sand. This paper is a report of a triaxial test series under rather common, moderate stress levels, and shows how the shearing angle of friction and dilatancy are affected by the water content.

From the viewpoint of a generalised stress-strain behaviour of cohesionless soils (Miura & O-Hara, 1979, Semple 1988, Coop 1988), it has been brought forward that the particle crushing phenomenon can be found in all sands from a certain stress level on, depending on their mineralogy. Therefore, it is expected that similar phenomena occur during the triaxial compression tests of calcareous sands as presented here and those done on Toyoura sand by Miura & Yamanouchi (1975) at pressures up to 50 MPa. These authors found that at high confining pressures, saturated samples showed more volumetric strain, lower shear strength and stiffness, and less dilatancy at failure, all due to the increased amount of particle breakage that is induced by the presence of water.

2 MATERIAL DESCRIPTION

For this study a calcareous sand from the Persian Gulf was tested. This "S2-sand" has angular grains and a high $CaCO_3$ content in the form of aragonite and calcite, reflecting its bioclastic origin. Because of the angular particles, the brittleness of the mother material and the intraparticle porosity, it is a very crushable sand. The grain size distribution is given in Figure 1, physical properties in Table 1.

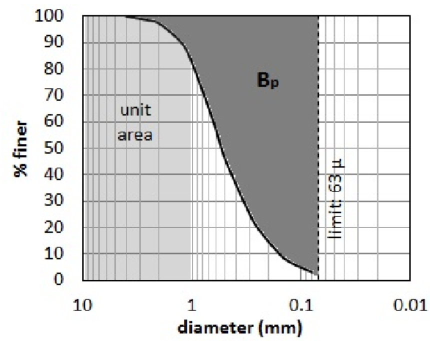

Figure 1. PSD of S2 sand. The breakage potential B_p is the area above the original PSD and in this study is adapted to particles larger than 63 µm because of the use of ASTM sieves.

Table 1. Physical properties of S2 sand.

CU	3.67
D_{50}	0.57 mm
B_p	0.92
$G_{s,intact}$	2.82 g/cm³
$G_{s,powder}$	2.88 g/cm³
$CaCO_3$	92%
e_{min}	0.832
e_{max}	1.382

3 TEST DESCRIPTION

All samples are reconstituted from weighted sieve fractions to maintain the same grading at the start of every test. As such, the breakage potential (the area above the grain size distribution curve indicated in Figure 1, Hardin 1985) remains the same for all tests, legitimating the comparison of breakage.

Triaxial compression tests were carried out on cylindrical specimens (height 88 mm, diameter 38 mm). Samples were prepared in 6 layers, using dry pluviation and gentle tamping following the undercompaction method of Ladd (1978) to obtain a homogeneous relative density D_r of 60%. Samples that were to be tested at higher consolidation stresses were prepared at slightly smaller densities, so that upon consolidation and at the start of the shearing phase, all samples would have a D_r of 65±3%. This density is a relevant in-situ value, it provides a stable structure that won't easily collapse upon saturation and can be reached without crushing.

Half of the samples were tested dry. The other half were saturated in the triaxial cell by flushing the samples with de-aired water and application of a back pressure, yielding B-coefficients of minimum 0.94. After saturation, S2-samples were consolidated under stresses of 50, 100, 200, 400, 500 and 750 kPa respectively. For the wet samples, the amount of drained water was recorded with a volume change transducer, and the density at the end of consolidation (i.e. at the start of shearing) could be calculated. For the dry sands ΔV-values were not known; it was assumed that they would compact equally to the wet sands, although earlier oedometer tests on S2-sand (Wils et al. 2014) proved that wet S2 is more compressible and would therefore be at higher density at the start of the shearing phase.

The consolidated samples were sheared at a strain rate of 0.1 mm/min and the deviator stress was recorded. Volume changes of the wet samples were also obtained and pore pressures were monitored to ascertain that shearing happened under drained conditions.

Tests were terminated at an axial strain of 20% of the initial sample height. Samples were then dried and sieved. The change in grain size distribution was measured using Hardin's relative breakage factor (Hardin 1985), which is the ratio of total to potential breakage. The potential breakage is defined in Figure 1 and the total breakage is the difference in area between the original PSD and the PSD after testing.

4 TEST RESULTS

The results are presented in Figure 2 in terms of principal effective stress ratios for the different levels of consolidation pressure. In these graphs a clear peak can be seen for the low confining stresses, which is to be expected for a sand of medium high density.

Figure 3 shows the peak results interpreted using the secant angle of friction (Bolton, 1986). The tests at the lowest pressure (50 kPa) yield the highest friction angle. Figure 4 depicts axial strain at failure, and shows that these tests also reached the lowest axial strain at failure. Figure 5 shows that breakage during this test is the smallest. Figure 6 illustrates the volumetric deformation of the wet tests for different axial strain levels, and shows that the drained triaxial test at 50 kPa yields the most dilatant volumetric strains. Tests at increasing confining pressure show a softening of the stress-strain curves (Figure 2), typical for shear tests under drained conditions on crushable sand. The deviator stresses increase with increasing pressure (Figure 7) but friction angles decrease (Figure 3), the amount of net volumetric compression increases (Figure 5), and higher strains were necessary to establish the peak strengths (Figure 4) (Semple 1988, Yamamuro & Lade 1996, Maeda & Miura 1999, Coop et al. 2004), such that the dry test at 750 kPa was stopped before the peak was reached.

Concerning the difference between dry and wet sands at all confining stress levels the wet samples show a lower peak deviator stress and peak friction angle (Figures 3, 7). Apart from the test at 750 kPa the peak strengths for wet S2 samples are reached at higher axial strains (Figure 4).

The measured volume changes during drained shearing of the wet samples were interpreted in terms of dilation and contraction. Under low confining pressure, until 100 kPa, the sand shows clear shear-dilation (Figure 6) with a net volumetric dilation at the end of the test, but at higher stresses there is a lot of contraction before the peak strength is reached.

All samples bulged during shearing yet at peak stresses an obvious shear band did develop for the tests at stresses below 500 kPa. On the contrary, no definite slip surface was observed for the drained tests at 500 and 750 kPa. As can be seen from the volume change measurements (Figure 6), these samples were still contracting towards the end of the test. The volumetric strains from the 500 and 750 kPa tests are very close to each other. It is likely that the specimen will not contract much further during shearing despite continued loading. This is supported by the zero rate of volume change at failure for these stress levels (Yamamuro & Lade 1996). The volume change of dry S2 was not recorded, but is less compressible (Wils et al., 2014), and will likely contract less during shearing than wet S2. Even without ΔV data, some volume change characteristics can be derived, since the maximum dilation rate is associated with the peak strength (Bolton, 1986). Taking for instance the strain level at the peak of dry S2 for the tests at 400 kPa (Figure 4), and looking at the corresponding (ΔV, ε_a) graph for wet S2 at this stress level (Figure 6), it is seen that by the time the dry sand has reached its peak strength and therefore its maximum dilation rate, the wet sand is still contracting.

After the peak, the deviator stress drops for all samples (Figure 7), but without evolving into a clear constant stress and volume state and without maintaining a clear distinction between dry and wet tests.

Crushability was evaluated from the shift in grain size distribution and quantified using Hardin's breakage factor B_r, the values of which are depicted in Figure 5 and it is obvious that the wet S2 underwent more crushing than dry S2 at all confining stresses. This was the combined result of isotropic compression and shearing (Bishop 1966, Yamamuro & Lade 1996), as was verified by sieving after consolidation of dry and wet S2 until 200 kPa. The B_r factors were 0 for the dry test and 0.018 for the wet test, both much smaller than the respective values obtained after consolidation and shearing, i.e. 0.035 and 0.050.

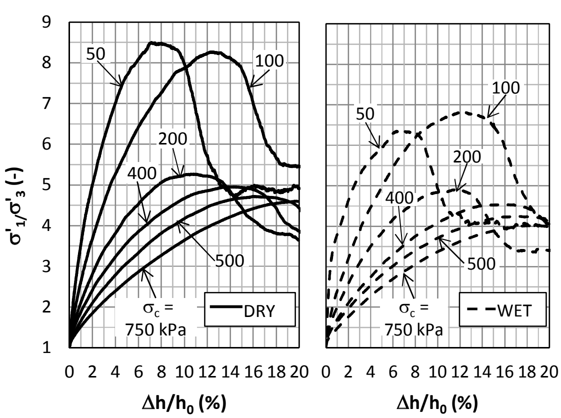

Figure 2. Normalized stress vs. axial strain curves.

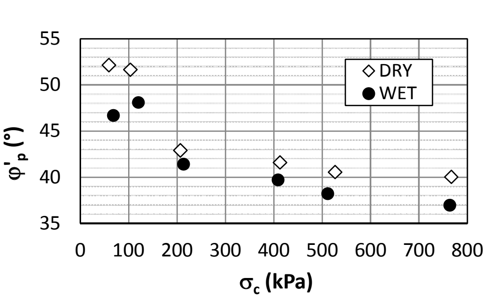

Figure 3. Peak secant friction angles $\varphi'_p = \text{asin}(\sigma_1\text{-}\sigma_3)_p/(\sigma'_1+\sigma'_3)_p$ at different confining stresses.

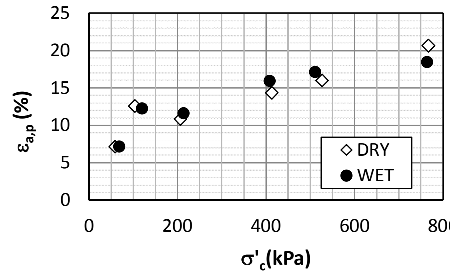

Figure 4. Axial strain at failure for different confining pressures.

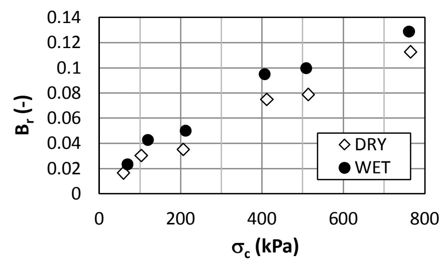

Figure 5. Relative breakage factors.

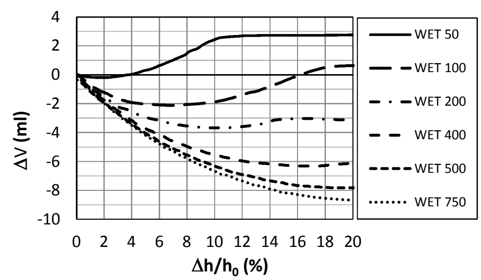

Figure 6. Volumetric strain – axial strain curves from wet tests.

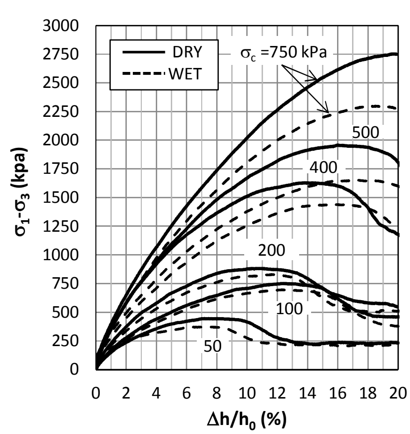

Figure 7. Deviatoric stress – axial strain curves.

5 INTERPRETATION

It is widely appreciated that increasing the confining pressure on any sand increases the compressibility, the peak deviator stress, the axial strain at failure and the initial stiffness (Semple 1988). Peak friction angle reduces with effective stress both for quartz sand (Bishop 1966, Bolton 1896) and bioclastic soils (Golightly and Hyde 1988). Calcareous sand generally has a higher mobilized friction angle than quartz sand, but it decreases faster with increasing pressure due to the high compressibility which suppresses and delays dilation. The increased compression is mainly caused by grain crushing and fine particles infilling the space between grains (Miura & O-Hara 1979).

The tests presented here demonstrate that water accelerates particle-crushing and intensifies contractile shear response; hence the dilatancy effect is decreased and the strength of a saturated calcareous sand decreases even faster than that of a dry sample.

The stress-strain curves of dry and wet S2 change considerably through particle crushing; the peak of the stress-strain curve becomes less pronounced and the sand changes from a strain hardening-softening into a strain hardening material (Miura et al 1984). This is obvious from Figure 2, although the water condition does not affect the general shape of stress-strain curves at identical stress levels.

Increasing the confining pressure from 500 to 750 kPa does not affect the maximum effective stress ratio (Figure 2) neither for dry nor wet conditions. Some researchers found that the decrease of friction angle with increasing pressure achieves a stable value at high pressures (Miura & Yamanouchi 1975); indeed Figure 3 suggests a cessation of friction angle decrease for wet and dry S2 sand from 500 kPa on.

The evolution of volumetric strain of wet tests at 500 and 750 kPa are very similar (Figure 6). The rate of volume change at failure is close to zero for both stress levels. It is likely that these specimens have reached the maximum volumetric compression, and shearing at higher stresses will hardly cause further contraction or crushing (Yamamuro & Lade 1996). The increase of strain level at failure is also slowing down at this point (Figure 4). The reason breakage doesn't show similar signs of stabilizing (Figure 5), is believed to lie in the choice of preparing the samples at different densities, so as to obtain equally dense samples after consolidation. Even though all samples had a similar density at the start of shearing, more breakage occurred prior, during isotropic compression of initially looser samples that were consolidated to higher stress levels. This additional breakage in the consolidation stage keeps increasing the breakage with confining pressures, although the volumetric contraction during the subsequent shear phase no longer increases at stress levels beyond 750 kPa.

The apparent stabilization of the stress that is observed after peak for tests at low pressures is not an effect of critical state occurring in the shear band. Firstly, it impossible to deduce the ultimate conditions in the shear plane from measurements of boundary displacements and volume changes (Bolton 1986) because of the non-uniformity of strains in the sample. Secondly, the triaxial test device is unable to achieve the critical state (Coop 1988), as mobilization of this state requires stabilization in the grading and much higher strains (Coop 2004). The apparent constant volume state is actually a transient state where simultaneous crushing and dilation counteract.

Particle damage during shearing occurs already at low stresses (Figure 5). However, only sieving was done and Sadrekarimi & Olson (2010) found that the full increase of fines content during shearing can only be demonstrated when the soil is dispersed before sieving. During shearing, the coarser particles are in contact primarily through the fines. Regarding the perceived differences in the crushability of wet and dry S2-sand during shearing, it is suggested that these fines, which are important for the reduction of the contact stresses, might have been released from the coarse particles in a saturated environment and are floating in the water with no structural purpose.

6 CONCLUSION

The authors investigated the mechanical behaviour of crushable sands in saturated environments, in regards to such matters as the calculation of the point resistance of offshore piles.

At high stress particle crushing is the most important factor affecting the behaviour. If particles crush before they override (Bolton 1986) the sand contracts when it would otherwise expand. Crushing can even be expected at low to moderate stress levels during shearing of crushable sands, and more so for wet samples. At similar stress levels, the breakage of wet calcareous S2 sand is higher, the axial strain at failure is higher and the friction angle is lower.

The presented results are similar to those of dry and wet triaxial tests on quartz sand at high stresses (Miura & Yamanouchi, 1975), but the stress levels used in this study are more commonly encountered in and around geotechnical constructions. Therefore, design and data interpretation for calcareous soils that are susceptible to dry-wet cycles are particularly advised to address the effect of water.

The absence of dilation in bioclastic sand is critical for its field performance and is worse in wet environments. Semple (1988) also cautioned for the translation of shear test results to in situ performance of bioclastic sand. In tests the confining stress is kept constant, whereas on site contraction of calcareous sand induces a collapse of the mean stress field, thus reducing the mobilized shear resistance. The larger strains necessary for saturated calcareous sand to develop its full frictional resistance, should also be considered in the design of engineering structures (Coop 1988). Since crushing is an irrecoverable strain, wet calcareous sand might also perform even more poorly than dry calcareous sand in terms of skin friction on driven piles and cyclic loading, or other conditions where soil rebound is essential (Semple 1988).

The reason for the influence of water on the decrease in friction angle is not established in this paper but should be found by looking for the reason that its presence facilitates crushing. The measured friction angle is the result of mineral sliding, dilatancy, particle rearrangement and crushing (Rowe 1962). Miura and Yamanouchi (1975) explained from a mechanochemical viewpoint that water decreases the crushing strength of particles. This would indeed reduce the effective stress at which dilatancy is suppressed (Bolton 1986). Salezadeh et al. (2005) noted the decreased hydraulic conductivity in the shear band caused by the increase of fines, which would slow down pore pressure dissipation and may account for a decrease in shear strength. Kim et al. (2014) reasoned that crushing exposes intraparticle voids, causing a pore pressure decrease, an effective stress increase and hence crushing. It remains to be evaluated whether, when, and to what extent water promotes structural collapse, weakening of particles or interface lubrication, and which aspects – e.g. contact physics, mineral friction, cementation – contribute to the perceived differences between the geotechnical characteristics of wet and dry calcareous sand.

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