BEARING CAPACITY OF FOOTINGS ON CALCAREOUS SANDS

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ABSTRACT

The bearing capacity and compressibility of a calcareous sand were examined by drained triaxial compression tests and laboratory model tests on strip footings 51mm by 305mm. The tests were carried out on two sands—a calcareous sand having a carbonate content of 18% and a clean sand obtained by washing the calcareous sand with Hydrochloric Acid (HCl) and water. All tests were performed at a relative density of 66%. The results indicate increased compressibility and no change in the peak strength parameters due to the presence of carbonates. The bearing capacity of the calcareous sand was smaller than that of the clean silica sand. Various methods of analysis were discussed to account for the effect of compressibility on bearing capacity.

Key words: bearing capacity, compression, failure, foundation, model test, shear strength, special soil, stress-strain curve, triaxial compression test (IGC: D6/E3/E14)

INTRODUCTION

Calcareous soils are characterized by the presence of carbonates in the soil matrix. These carbonates consisting predominantly of calcium carbonates are deposited around the particles and in the voids of the soil matrix. They cover a great part of seabeds where oil and gas platforms are built. The areas concerned are near Australia, India, North Atlantic, or in the gulf of Mexico, and the Arabian Gulf (Levacher, 1988). Calcareous sands exist also on land in arid and semiarid environments, Beckwith and Hansen (1982), Netterberg (1982), Oweis and Bowman (1981), Ismael and Al-Sanad (1986). The carbonate content varies generally from 10 to 30% for land deposits, Ismael et al. (1986), to 90% or more

for offshore deposits, Golightly and Hyde (1988), Hull et al. (1988), Morrison et al. (1988). Because these soils behave differently from clean silica sand, being particularly more compressible, interest has grown into their fundamental behavior, and load transfer mechanism. The first international conference on calcareous sediments was held in Perth, Australia (Jewell et al., 1988) to discuss the properties and behavior of these soils.

The performance of pile foundations in granular carbonate sediments has been extensively investigated in recent years in connection with the construction of oil platforms and some structures on land. It has been demonstrated that these sediments offer inferior skin friction for driven piles due to significant volume change during shear and reduced lateral stress against

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the pile shaft and particle crushing, McGlelland (1974), Murff (1987). However, the skin friction for drilled and grouted piles in calcareous sands is always greater than that of driven piles in the same material (Nauroy et al., 1988). In contrast, there is a lack of information in the literature on the performance of shallow foundations on these deposits, Poulos and Chua (1985), Dutt and Ingram (1988). The reason for this lack of concern is that, granular deposits whether siliceous or calcareous provide large bearing capacity compared with the allowable soil pressure which is usually governed by the permissible settlement. This is even when the bearing capacity is reduced due to soil compressibility.

With increased interest in the bearing capacity of shallow foundations on calcareous sands, both offshore for jack-up footings of mobile drilling units, and on land for buildings and marine structures, a laboratory testing program consisting of isotropically consolidated drained triaxial tests and model tests on strip footings was carried out on a calcareous fine sand from Kuwait having a carbonate content of 18% and on the same soil after removing the carbonates. To determine the effect of carbonates on the fundamental soil properties including strength and compressibility, both soils were tested at the same relative density. This paper analyzes test results and compares the measured bearing capacity factors with the theoretical values. Various methods to account for soil compressibility are discussed including Terzaghi (1943) suggestion to use reduced strength parameters, and Vesic (1973) recommendations to employ compressibility factors in the bearing capacity equation based on the cavity expansion theory. A simple empirical method is proposed to account for soil compressibility.

BASIC PROPERTIES AND TRIAXIAL TESTS

The sand used in this program is a calcareous fine to medium windblown sand taken from Jabriya, Kuwait. A large bulk sample weighing 1 ton was recovered from a depth of 1 m for use in the triaxial and model tests. Chemi-

cal tests on representative samples indicated that the silica content is 75% and the amount of carbonates is 18.21% mostly in the form of calcium carbonate, Ismael et al. (1986). The cleaning process was carried out by washing with predeterminate amounts of conc. Hydrochloric Acid (HCl) and then with water to complete the reaction, and replace the carbonates with chlorides which are soluble in water. The chemical reaction is as follows:

Chemical analysis of the washed samples using a calcimeter revealed a 4% residual amount of carbonates remaining in the sand which is designated herein as clean sand. The physical properties of both the calcareous and the clean sand are summarized in Table 1.

As in Fig. 1, the two soils have almost identical grain size distribution curves. Isotropically consolidated drained triaxial compression tests with volume change measurements were performed on both soils at five confining pressures and initial relative density of 66%.

Table 1. Classification properties of calcareous and clean sands

Property	Calcareous Sand	Clean Sand
Coefficient of Uniformity	4.4	4.0
Mean Diameter (mm)	0.21	0.20
Specific Gravity	2.71	2.67
Maximum Density (Mg/m ³)	1.846	1.816
Minimum Density (Mg/m ³)	1.596	1.555

Fig. 1. Grain size distribution curves

This is a representative value of field conditions which was also employed in the model footing tests described in the next section. Fig. 2 is a plot of $(\sigma_1 - \sigma_3)/2$ vs. $(\sigma_1 + \sigma_3)/2$. As shown, both soils have identical strength parameters c , $\phi = (0, 34^\circ)$, where c is the cohesion and ϕ is the angle of shearing resistance.

While Fig. 2 shows identical shear strength for the two cohesionless soils, their deformation characteristics varied considerably with the presence of carbonates. Fig. 3 compares the axial stress difference $\sigma_1 - \sigma_3$ versus axial strain ϵ , and the volumetric strain $\Delta V/V$ versus ϵ for the calcareous and the clean sand at confining pressures of 140 and 630 kPa. The presence of carbonates has clearly resulted in a decrease in the soil modulus and an increase in the axial and volumetric strain at all stress levels. Both sands exhibited volume reduction.

Fig. 2. Failure envelope and strength parameters for calcareous and clean sands

Fig. 3. Stress-strain & volume change curves from drained triaxial tests

On account of the low relative density employed herein, the clean sand did not display volume increase or dilatant behavior during shear.

Table 2. Results of drained triaxial compression tests

Soil Type	σ_3 , in (kN/m ²)	Axial Strain at failure ϵ_f (%)	$(\sigma_1 - \sigma_3)$ (kN/m ²)	E_t (kN/m ²)	c kPa	* ϕ , in degrees
Calcareous	140	14.0	171	20800	0	~ 34
	280	12.8	333	33000		
	420	13.0	511	37000		
	560	12.8	706	33300		
	630	10.0	736	40000		
Clean	140	7.8	160	33300	0	~ 34
	280	11.0	340	41000		
	420	11.8	506	50000		
	560	11.6	662	60000		
	630	7.8	741	76200		

$$*\sin \phi = (\sigma_1 - \sigma_3)_f / (\sigma_1 + \sigma_3)_f$$

Fig. 4. Initial tangent modulus, E_t , versus confining stress, σ_3

The peak stress reached is almost the same for both soils at all confining pressures. Table 2 is a summary of the strength and modulus values determined from the triaxial tests. Values of the Poisson's ratio were calculated by

$$\nu = \frac{1}{2} \left(1 - \frac{\delta \Delta}{\delta \varepsilon} \right) \quad (1)$$

in which ν = Poisson's ratio; Δ is the volumetric strain, and δ = incremental change. Values calculated from the initial portion of the stress strain curves varied in a narrow range of 0.25–0.34.

The initial tangent modulus E_t determined from $(\sigma_1 - \sigma_3)$ vs. axial strain (ε) curves was plotted versus σ_3 in Fig. 4. The points fit approximately the following relationship.

$$E_t = K(\sigma_3)^{0.5} \quad (2)$$

where K is a constant which depends on the soil density and compressibility. The value of K determined herein is 2700 for clean sand and 1750 for calcareous sand. Similar relationship was employed by Duncan and Chang (1970), and Clough and Duncan (1971) in nonlinear analysis of stress and strain in soils.

MODEL FOOTING TESTS

A steel sand box measuring 1*1*0.5 m depth was employed in the tests. A rigid frame mounted on the box provided reaction. The loads were applied by a manually operated hydraulic jack connected to a proving ring for

Fig. 5. Test set-up for strip footing

measuring the load. A rigid strip footing 51*305*51mm thick with rough base was attached to the proving ring. Fig. 5 shows a general view prior to a test on calcareous sand. Displacements were recorded by two dial gauges on both sides of the footing. Density measurements were made after each test using the sand cone test in accordance with ASTM Designation D 1556 (1987). The tests were conducted at the surface and at a depth of 0.2 m at a corner of the box not affected by the failure zone. The results compared well with the overall density obtained from the weight of sand placed in the box in each model test.

To carry out a test, the box was first filled with sand having the desired relative density of 66%. To do this equal weights of sand were placed, levelled, and tamped to form 76 mm layers. Equal increments of stress were applied and maintained for at least 5 minutes and until all movements had ceased. Each loading was approximately equal to 10% of the expected ultimate pressure. Three identical tests were carried out on calcareous sand followed by three tests on clean sand. For each model test the sand was removed from the box and recompacted to the same relative density. The effect of particle breakage during recompaction was negligible since the sand consisted mainly of quartz having high mineral hardness. This was confirmed by comparing the gradation characteristics of both sands before and after completion of the tests.

The amount of sand required for a model tests was nearly 800 kg. Cleaning carbonates

from such a quantity of sand required a lot of effort. First, the soil was placed in plastic containers and HCl was added gradually in precalculated amounts and mixed with the soil. After the complete chemical reaction has occurred the soil was washed with water with care taken not to remove fine particles. The soil was subsequently dried prior to its use in the model tests.

The preceding method was preferred in order to maintain the origin and the grading characteristics of the soil. Many investigators have compared the behavior of calcareous sediments with those of silica sand taken from other locations and having different basic properties and gradation characteristics, Poulos and Chua (1985), Golightly and Hyde (1988), Hull et al. (1988). As such, the difference in behavior can not be exclusively attributed to the presence of carbonates in the soil matrix.

ANALYSIS AND DISCUSSION OF TEST RESULTS

Fig. 6 shows the pressure-settlement diagrams for clean and calcareous sands. It is evident that the presence of carbonates has reduced the average bearing capacity to 130 kN/m^2 or 56% of the corresponding value of 231 kN/m^2 for clean sands. The measured values obtained from three identical tests ranged from 229 to 236 kN/m^2 for clean sand and 120 to 136 kN/m^2 for calcareous sand. An increase in settlement for calcareous sand is also visible at

working loads indicating a more compressible behavior.

The results of the drained triaxial tests and model footing tests appear contradictory. In the triaxial tests, identical strength parameters were obtained for both materials, but the calcareous sand had a lower modulus and higher strain at failure. However, in the model footing tests, both the bearing pressure and the strain at failure are lower for the calcareous sand than for the clean sand. To explain this contradiction, it has been argued, Vesic (1973), Ismael and Vesic (1981) that the bearing capacity is not exclusively dependent on the strength parameters c , ϕ of the soil but is also influenced by the deformation parameters E , ν .

To arrive at an adequate assessment of the influence of soil compressibility and related scale effects, it would be necessary to have a bearing capacity theory based on a more realistic soil model. Such model would allow elastic deformation prior to shear failure (elastic plastic solid). It has been observed using X ray radiography (Ismael, 1974) that failure in compressible soils is progressive and occurs after initial elastic settlement. At the instant of failure, only the elastic wedge and small zones of radial shear are formed beneath the footing. With continuous loading all zones characterizing general shear failure occur.

To account for soil compressibility, Terzaghi (1943) proposed to use reduced strength parameters in the bearing capacity formula. Vesic (1973) introduced the compressibility factors

Fig. 6. Pressure-settlement diagrams for strip footings

ξ_{qc} , ξ_{rc} in the general shear equation based on the rigidity index I_r defined as

$$I_r = \frac{G}{\tau} = \frac{E}{2(1+\nu)(c+q \tan \phi)} \quad (3)$$

where q is the initial effective overburden pressure, G is the shear modulus, τ is the shear stress.

The compressibility factors ξ_{qc} , ξ_{rc} are defined as

$$\begin{aligned} \xi_{qc} &= \frac{N_q(\text{compressible})}{N_q(\text{incompressible})}, \\ \xi_{rc} &= \frac{N_r(\text{compressible})}{N_r(\text{incompressible})} \end{aligned} \quad (4)$$

where N_q and N_r are the bearing capacity factors.

The solution giving the compressibility factors is based on the assumption that the ultimate normal pressure on the sides of the wedge under the footing is equal to the pressure needed to expand a cavity in the same soil mass. The analytical values of the compressibility factors are determined from

$$\begin{aligned} \xi_{rc} = \xi_{qc} = \exp \left\{ \left[-4.4 + 0.6 \frac{B}{L} \right] \tan \phi \right. \\ \left. + \left[\frac{(3.07 \sin \phi)(\log_{10} 2 I_r)}{1 + \sin \phi} \right] \right\} \end{aligned} \quad (5)$$

The rigidity index I_r defined above is the inverse of the shear strain in the failure zone below the footing. It is the critical parameter controlling the compressibility factors for a particular footing shape and soil conditions. Vesic (1973) presented the numerical values of the compressibility factors for both strip and square or circular footings as a function of ϕ and I_r .

From Eq. (5), the magnitude of the rigidity index can be found for any angle, ϕ , below which it becomes necessary to reduce the bearing capacity because of compressibility effects. This critical rigidity index is given by

$$\begin{aligned} I_r(\text{critical}) &= 0.5 \exp \left[\left(3.3 - 0.5 \frac{B}{L} \right) \right. \\ &\quad \left. \times \cot \left(45 - \frac{\phi}{2} \right) \right] \end{aligned} \quad (6)$$

Numerical values of the critical rigidity index which separates compressible soils from incom-

pressible soils are obtained by direct substitution in Eq. (6). For the footing employed herein where $L/B=0.167$, $\phi=34^\circ$, $I_r(\text{critical})=215.3$.

The bearing capacity factors N_r and N_q were calculated from test results using the following shape factors proposed by De Beer (1970).

$$\zeta_r = 1 - 0.4 \frac{B}{L} \quad (7)$$

$$\zeta_q = 1 + \frac{B}{L} \tan \phi \quad (8)$$

in which ζ_r , ζ_q are shape factors; B =width of footing and L =Length of footing.

To obtain the value of N_r , N_q the bearing capacity equation is written in the form

$$q_0 = \gamma \cdot w \cdot N_q \zeta_q \zeta_{qd} + \frac{1}{2} \gamma B N_r \zeta_r \quad (9)$$

in which, q_0 =the ultimate pressure, γ =unit weight of the soil, w =the settlement of the footing at the moment of failure, ζ_{qd} is a depth factor defined by:

$$\zeta_{qd} = 1 + 0.35 \frac{w}{B} \quad (10)$$

Solving for N_q yields:

$$N_q = \frac{q_0}{\gamma B \left(\frac{w}{B} \zeta_q \zeta_{qd} + \frac{N_r}{2 N_q} \zeta_r \right)} \quad (11)$$

To determine N_q from Eq. (11), Eqs. (7), (8), (10) are used with assumed factor N_r in the form:

$$N_r = 2(N_q + 1) \tan \phi \quad (12)$$

The values of N_q and N_r were computed from Eqs. (11), (12) for calcareous and clean sands using the present experimental results. These were $N_q=309$ for clean sand and 190 for calcareous sand, $N_r=418$ for clean sand and 258 for calcareous sand. The compressibility factors defined by Eq. (4) as the ratio of these values are $\xi_{rc}=\xi_{qc}=0.61$, considering the clean sand as incompressible.

The theoretical values of N_q were determined from

$$N_q = e^{\pi \tan \phi} \tan^2 \left(\frac{\pi}{4} + \frac{\phi}{2} \right) \quad (13)$$

and N_r was determined from Eq. (12). On this basis, for $\phi=34^\circ$, N_q and N_r were calculated as 29.44 and 41.06 respectively.

Fig. 7. Effect of size on bearing capacity of surface footings in sand (after De Beer, 1965)

Comparing the computed values of N_q and N_r to the theoretical values, it is noted that the computed values are 6 to 10 times greater than the theoretical values for calcareous and clean sands respectively. These important differences which may appear surprising is a direct result of the effect of footing width on the bearing capacity factors of surface footings and has been observed by many investigators as noted by De Beer (1965). It has been demonstrated recently by Hettles and Gudehus (1988) that this scale effect can largely be attributed to the influence of pressure level on the angle of shearing resistance ϕ .

The computed values of N_r are superimposed on Fig. 7 taken from De Beer (1965). As evident the test results lie in the range of measured values by other investigators. Fig. 7 also indicates that as the width of the footing increases the bearing capacity factor N_r decreases and it is anticipated that the theoretical values are only reached for larger footing sizes. It also appears that the values of N_r tend asymptotically to some minimum.

The preceding discussion revealed the important effect of footing width on the bearing capacity factors for surface footings. The computed compressibility factors are, however not affected by the footing width since they are ratios of the bearing capacity factors for footings of the same width. Therefore the compressibility factor determined herein as 0.61

remains a reliable estimate of the effect of mineral compressibility on the bearing capacity of strip footings.

To examine the proposal by Terzaghi (1943) to use reduced strength parameters for compressible soils defined by

$$\begin{aligned}\bar{c} &= 0.67c \\ \bar{\phi} &= \tan^{-1}(0.67 \tan \phi)\end{aligned}\quad (14)$$

Using ϕ determined from Eq. (14) as 24.3, the theoretical bearing capacity factors are: $N_q = 9.9$, $N_r = 9.88$. Dividing these factors by the corresponding values for incompressible soils, the compressibility factors were determined as $\xi_{qc} = 0.34$, $\xi_{rc} = 0.24$. This is nearly half the measured value of 0.61.

The preceding analysis indicated that the Terzaghi (1943) proposal is conservative for surface footings on compressible calcareous soils with low carbonate content. Similar findings were reached by Ismael and Vesic (1981) where the compressibility was induced by adding mica to silica sand. In contrast, for very compressible deposits having carbonate contents of about 90%, and existing usually offshore, Poulos and Chua (1985) and Dutt and Ingram (1988) have found that this method will over predict the bearing capacity.

The cavity expansion model takes the soil deformation parameters into account and provides a closer prediction of the bearing capacity, Poulos and Chua (1985). However the accuracy

of the compressibility factors is dependent on the selection of the parameters E , ν from laboratory tests where repeatability is questionable, Dutt and Ingram (1988). This is especially true for surface and near surface model footing where the values of E and q are very small, and require further triaxial tests at very low confining pressure. This is to take the curvature of failure envelope in the low pressure range into consideration. This curvature causes sharp increase in the angle ϕ at low confining pressures. For these reasons estimation of reliable I_r values was not possible and the method was not applied to the tests of this study. It will be extended, however, to future tests with side overburden pressure.

Considering the constant K appearing in Eq. (2) as an index of compressibility, the values of this constant were determined as 1750 and 2700 for calcareous and clean sands respectively. The ratio between the K values of 0.64 is close to the compressibility factors $\xi_{qc} = \xi_{rc}$ calculated herein as 0.61. Thus the reduction in the bearing capacity factors may be approximated by the reduction in the deformation modulus value at a confining pressure of 100 kN/m² (1 tsf) for compressible vs. incompressible soil at the same relative density. This simple method needs confirmation by other case studies.

Poulos and Chua (1985) carried out loading tests of circular footings 25 mm diameter in a vessel 305 mm diameter and 229 mm deep. They used a silica sand from Sydney, and calcareous sand from Bass Strait, Australia which has a carbonate content of 89%. Test results on both sands at different relative densities and an initial overburden pressure 103 kN/m² is reproduced in Fig. 8. The bearing capacity for the calcareous sand decreased to approximately 58% of the value for silica sand. Had it not been for the many differences between the present tests and the tests by Poulos and Chua (1985), it would have been possible to conclude that compressibility does not increase linearly with carbonate content, since the same reduction in bearing capacity was approximately obtained in both cases despite the large difference in the carbonate content and test procedure. Poulos and Chua carried

Fig. 8. Influence of initial relative density and footing embedment on ultimate bearing capacity (after poulos and Chua, 1985)

out strain controlled rather than stress controlled tests on two sands from different origin and different grading characteristics, and defined failure arbitrarily as corresponding to 10% of the footing diameter. Moreover, they ran the test on saturated sand as compared to the dry sands employed herein. Due to these differences the similarity in the effect of the presence of carbonates on the bearing capacity appears to be accidental.

Practical Application.— Despite lower bearing capacity for calcareous soils as shown in Fig. 6, the allowable design soil pressure is not significantly affect since it is usually based on a limited permissible settlement. The actual bearing capacity of the full size foundation will be much greater than those obtained from the model tests and the allowable pressure will be in the initial elastic portion of the pressure-settlement diagram. The difference in settlement between calcareous and clean soils at low stresses is not significant to justify any change in design recommendations.

The foregoing recommendations, however, applies only to the granular soils tested herein with low carbonate content (18.2%) possessing some compressibility and where permissible settlement rather than bearing capacity controls the magnitude of the allowable soil pressures. However, for highly compressible deposits such as offshore marine deposits consisting pre-

dominantly of carbonates, the bearing capacity may be very low. Therefore, it must be calculated by one of the aforementioned methods and a reasonable factor of safety must be ensured against bearing capacity failure. Furthermore, the additional settlement due to compressibility must be considered in the analysis. In these cases, it is recommended to carry out plate load tests to failure, and insitu determination of the soil parameters preferably using the pressuremeter test.

Since it is already known that with increased carbonate content increased compressibility and decreased interparticle friction occur (Murff, 1987), the bearing capacity is expected to decrease with an increase in the carbonate content. Also the mode of failure of shallow footing will also change from general to punching shear failure. Other characteristics of the soil fabric affecting compressibility include the particle nature, and the void ratio of the soil matrix.

CONCLUSIONS AND RECOMMENDATIONS

An experimental laboratory testing program was carried out to investigate fundamental properties of land deposits of calcareous sands with low carbonate content and the load carrying capacity of strip footings on these soils. Based on test results the following conclusions are reached:

1. Calcareous sands are compressible compared to clean silica sands. This is evident from the decrease in the modulus of deformation and the increase in the volume reduction during shear in triaxial compression tests.
2. The peak strength parameters c and ϕ were found to be the same for calcareous and clean sands with no effect due to compressibility.
3. The bearing capacity of footings on calcareous sands is lower than the corresponding values for clean sands. The effect on footing design will be significant for highly calcareous soils such as offshore deposits where the ultimate bearing capacity is very low. In such cases the bearing capacity rather than the permissible settlement may govern the magnitude

of the allowable soil pressure for foundation design.

4. The proposed method by Terzaghi to use reduced strength parameters to calculate the bearing capacity of compressible soils is conservative with respect to test results on surface footings. A simple method based on the ratio of the soil deformation modulus values is recommended.

5. It is recommended to examine the effect of increasing the carbonate content gradually up to 95% on the strength and deformation parameters and on the bearing capacity of foundations.

6. It is recommended to develop a bearing capacity theory for compressible soils based on initial elastic compression followed by plastic flow and shear failure.

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