

PLATE LOADING TESTS ON WEAKLY CEMENTED SURFACE DESERT SANDS

N.F. Ismael¹ and H.A. Al-Sanad²

ABSTRACT

The bearing capacity and deformation characteristics of weakly cemented surface sands were examined by a field testing program. The program consisted of plate loading tests carried out to failure employing three plate sizes: 0.3 m, 0.46 m, and 0.61 m diameter. Three ground conditions were tested including natural ground, soaked or saturated ground, and compacted ground. It has been found that the principal effect of cementation is to induce a cohesion intercept, and to reduce compressibility. The failure mode observed was punching shear with the failure load not well defined. Soaking resulted in a reduction of the bearing capacity and the deformation modulus. The decrease in bearing capacity and deformation modulus depends on the method and duration of soaking, degree of cementation and size of footing. Tests on recompacted ground indicated a 70% reduction of bearing capacity and deformation modulus due to the breaking of cementation bonds and the disturbance of the soil fabric. Settlement increased with plate diameter, however, the increase was less than the predicted settlement values for cohesionless soils.

INTRODUCTION

Cemented sands exist in many places of the world where arid or semi-arid environments prevail. In Kuwait extensive deposits of cemented sands occur at or near the ground level. This competent deposit, known locally as "gatch", extends to a great depth over limestone bedrock, it has varying degrees of cementation depending on the location and the depth below ground level. The excess of evaporation over rainfall leads to the precipitation of carbonates, gypsum, and iron oxides in the soil matrix and the formation of crusts of cemented sands.

With major construction in Kuwait over the past 10 years, many multistory buildings, parking garages, housing projects, power stations and other structures were founded on this deposit and interest in the properties and behavior of cemented sands has grown. Of importance are the bearing capacity, the failure mode of shallow foundations, the allowable soil pressure, deformation characteristics, and the effect of load cycling on cementation. The effect of saturation on the cementation bonds and bearing capacity is not known. The influence of soil disturbance caused by excavation and compaction of cemented sands has not been examined. Recently

¹ Professor, Dept. of Civil Engineering, Kuwait University, P.O. Box 5969, 13060 Safat, Kuwait.

² Associate Professor, Department of Civil Engineering, Kuwait University, P.O. Box 5969, 13060 Safat, Kuwait.

published research has dealt only with laboratory determination of the geotechnical properties of cemented sands and the role of cementation in inducing a cohesion intercept c and slightly increasing the angle of shearing resistance ϕ (Clough et al 1981, Saxena and Lastrico 1978, and Ismael et al 1986). The pronounced effect of artificial cementation on the cone penetration resistance of sand was investigated recently by laboratory tests (Rad and Tumay 1986).

In the absence of adequate information and field test results on cemented sands, local foundation design is currently based on penetration tests and the theories and empirical correlations developed for noncemented sands. This leads to uncertainties, and conservative designs. In the few instances where plate load tests were performed, they were intended to confirm the design values and were terminated at low stress levels in the elastic range.

To examine the bearing capacity and compressibility of cemented sands, plate loading tests were carried out to failure at one site where cemented sands exist at ground surface. Three plate sizes were employed: 0.3 m, 0.45 m, and 0.61 m diameter. Tests were carried out on natural ground at insitu moisture conditions and on saturated or soaked ground. The effect of soil disturbance was examined by tests carried out on soils excavated from a test pit and replaced at their original density. Laboratory tests were carried out to determine the soil physical properties and strength parameters.

This paper presents and analyzes the field test results with respect to the deformation characteristics and the bearing capacity. The effect of saturation and soil disturbance are assessed. Previous plate loading tests carried out locally within the elastic range are included to supplement the findings of the present testing program.

SOIL CONDITIONS

Two auger boreholes were drilled at the test site to a depth of 5.5 m. Sampling and Standard Penetration Tests (SPT) were carried out at 1 m intervals. The samples for classification tests were prepared by breaking the cementation bonds using a rubber pestle. Atterberg limit tests were conducted on fractions passing the No. 40 U.S. sieve. Data from the two boreholes were nearly identical and indicated similarity of the soil properties across the site. A summary of the soil conditions is given in Figure 1. The soil profile consists of a medium dense weakly cemented silty sand layer to a depth of 3 m. This is underlain by medium dense to very dense, silty sand with cemented lumps to the bottom of the borehole. The relative density of the surface samples was determined as 73%. Considering the largest plate employed herein which was 0.61 m, the cemented sand layer was considered to be of sufficient thickness and homogeneity that the pressure bulbs under the loading plates will be almost entirely within the same material. Chemical analysis was carried out on three samples taken from depths of 0.5 m, 2 m, and 3 m to determine their chemical

PLATE LOADING TESTS

composition. A summary of the results is given in Table 1. As may be noted the three samples consist mainly of quartz. However, the near surface sample contains approximately 10% carbonates and sulphates. This ratio decreases with depth becoming 1.88% for the sample at 3 m depth.

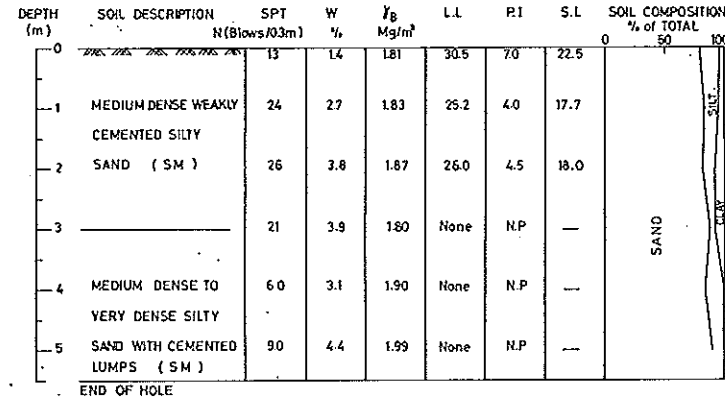


Fig. 1 Soil Conditions at the Test Site-Borehole 1

Table 1 Chemical Composition of the Soil Samples (Depth = 0.5 to 3 m)

Component Oxides	% Composition		
	0.5 m	2 m	3 m
SiO ₂	74.7	81.02	87.36
Fe ₂ O ₃	0.4	1.2	0.48
Al ₂ O ₃	6.00	3.98	5.30
CaO	10.1	4.5	1.42
MgO	1.73	2.94	2.94
CO ₂	3.73	3.14	0.70
SO ₃	0.37	0.48	0.17
CL	0.03	0.09	0.07
Organic Matter	2.26	2.09	0.92

Compounds	% Composition		
	0.5 m	2 m	3 m
CaCO ₃	8.48	7.12	1.59
CaSO ₄	0.63	0.82	0.29

To determine the strength parameters of the soil beneath the plates three series of drained direct shear tests were carried out in accordance with ASTM D 3080 (1982) to simulate field conditions. Tests of the first series were carried out on undisturbed samples trimmed from block samples taken at a depth of 0.5 m and tested at their natural moisture content. The second series were conducted on undisturbed samples saturated prior to testing. Disturbed samples which were remolded to the original density and moisture content were tested in the third series. The failure envelopes based on the peak and residual strengths are shown in Figure 2. The cohesion (c) and angle of shearing resistance (ϕ) determined based on peak strength were 15 kPa and 35° for the natural specimens, 0 kPa and 35° for saturated specimens and 2 kPa, 35° for remolded specimens. This indicates that saturation leads to the loss of the cohesion with no change in the angle of shearing resistance. Similar results were obtained from the remolded specimens except that a residual cohesion of 2 kPa was recorded which suggests that not all cementation bonds were broken in the remolded specimens tested in the direct shear apparatus. The residual parameters given in Figure 2 indicate similarity for the different sample conditions with no cohesion and a slight decrease of 1 to 2 degrees in the angle ϕ from peak values.

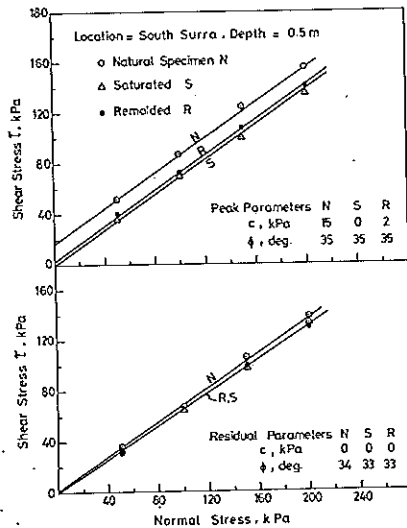


Fig. 2 Failure Envelopes for Different Sample Conditions

The shear stress versus shear displacement curves at two normal pressures are plotted in Figure 3. Examination of these curves reveals stiff behavior for the natural cemented samples followed by brittle failure after the peak strength was reached. The saturated and remolded samples are more compressible and less brittle after failure with little difference between them.

PLATE LOADING TESTS

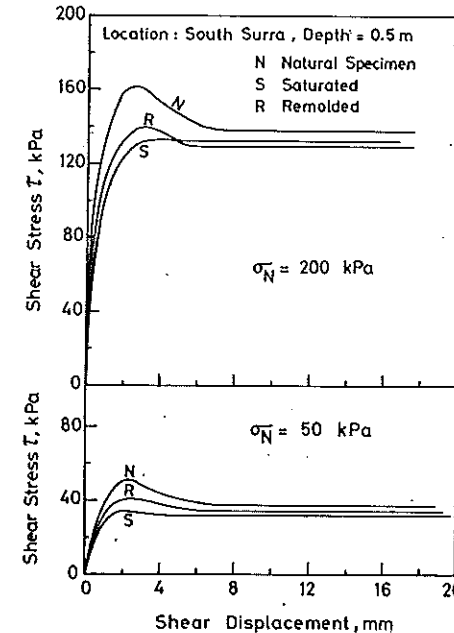


Fig. 3 Shear Stress vs. Shear Displacement Curves

The preceding results based on direct shear tests were employed in the analysis of the plate loading test data. Attempts to trim cylindrical samples for drained triaxial tests were not successful for samples taken at 0.5 m depth. The samples cracked during preparation due to the breaking of weak cementation bonds.

THE TESTING PROGRAM

Table 2 is a summary of the plate loading test program. A total of thirteen tests were conducted employing three plate sizes and three soil conditions. To carry out these tests twelve 0.3 m diameter bored piles were installed to provide reaction. A layout plan of these piles and the borehole locations is shown in Figure 4. Piles 1 to 4 were drilled to a depth of 3 m and were reinforced with a cage made of 4 - 22 mm bars. Piles 5 to 12 extended to a depth of 5 m and had reinforcing cages 0.25 m diameter consisting of six - 22 mm, reinforcing bars. All piles protruded 0.3 m above ground level and a central 36 mm reinforcing bar was positioned in every pile. This rod which was 3 m long was welded to a 0.5 m threaded ready made section of the same diameter. Of this length 2.7 m was embedded in the piles and 0.8 m (including the threaded section) projected above the top of the piles. The piles were installed in

Table 2 Summary of the Field Testing Program

Plate Size (m)	No. of Tests for Different Ground Conditions		
	Natural	Soaked	Compacted
0.3	2	1	2
0.46	2	1	1
0.61	2	1	1

Total No of tests = 13
 Depth of Tests = 0.2 m

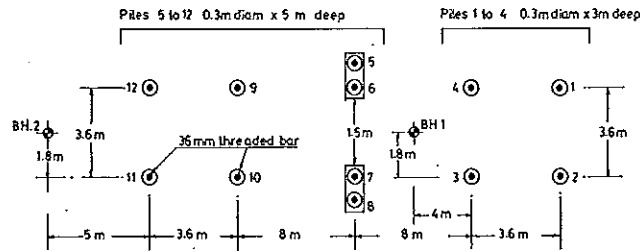


Fig. 4 Layout Plan of the Reaction Piles

a square pattern with a span of 3.6 m except for piles 5 to 8 which were installed along a straight line to form two small groups of two piles capped with a rigid cap and having a clear span of 1.5 m. The plate loading tests were conducted in the center of the spans forming the edges of the squares and in between the two small groups. As such a total of nine locations were available for testing which were sufficient for the tests on natural and saturated ground (Table 2). The remaining four tests on compacted ground were conducted by excavating test pits in between piles 1, 2, 3 and 4, replacing the soil to the prevailing insitu density and testing the plates on the compacted soils.

The installation of the reaction piles was carried out under favorable ground conditions. The holes were dry upon augering with no caving or collapse and no ground water was encountered. The steel reinforcing cage was lowered down in position and concrete was poured to the level of the tip of the central rod which was positioned in place and concrete poured to the top of the piles. The threaded section and the welded connection were tested in the laboratory to ensure adequate strength. Test results indicated that the threaded section will break first at a tensile load of 490 kN (50 tons). Thus a total reaction of 980 kN (100 tons) could be reached in each test which was considered satisfactory for the largest plate of the present tests.

PLATE LOADING TESTS

The installation of the piles served a dual purpose. First they were loaded laterally against each other at ground level and the lateral behavior of bored piles in cemented sand was investigated. Then they were employed to provide reaction for the present tests. The results of the lateral load tests are outside the scope of the present study and has been included in a separate paper.

EQUIPMENT AND PROCEDURE OF FIELD TESTING

Two rigid steel channels 4 m long × 0.3 m height × 0.1 m thick were employed as the test beams. To expedite the work two additional identical channels were brought to the site. The channels were reinforced with stiffeners and were placed back to back on top of the reaction piles. Figure 5 shows a photograph of the test set up prior to a plate loading test. The beams were tied to the reaction piles by means of 0.3 m × 0.5 m × 25 mm thick steel plates with a 38 mm central hole through which the threaded rod passed. The loads were applied using a 510 kN (52 ton) hydraulic jack having a long stroke of 250 mm. The jack was connected to a hand-operated pump equipped with a calibrated pressure gauge which read to an accuracy of 9.8 kN (1 ton). Three dial gauges having a range of 51 mm were attached from the reference beams to the test plates. The gauges were read at each load increment to an accuracy of 0.01 mm.

To carry out a test the upper 0.2 m of soil was removed, the ground was levelled and the test plate was positioned followed by smaller plates. The testing procedure followed ASTM D 1194-72 (1982). The load was applied in cumulative equal increments of not more than one tenth of the estimated bearing capacity. After the application of each load increment, the cumulative load was maintained for a

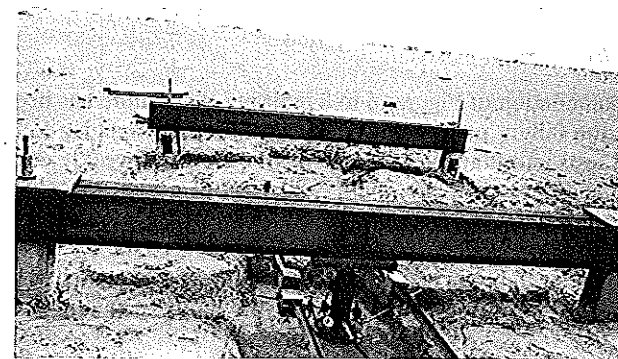


Fig. 5 Plate Load Test Setup

time interval of not less than 15 minutes and until all initial settlements had ceased. Each test was continued until the ratio of load increment to settlement increment reached a minimum steady magnitude or until failure was reached. In a few tests, the load was cycled in the elastic range to obtain the magnitude of the reloading soil deformation modulus.

Tests on soaked ground were conducted by prewetting the test area for a period of two hours prior to the test. Prior to loading, additional water was poured slowly in the test area to ensure saturation to a depth not less than twice the diameter of the largest bearing plate. The method of ground wetting was intended to duplicate field conditions resulting from heavy rain or leaking pipes. The volume of water seeping through the ground, and the type and composition of the surface soils, Fig. 1, indicated that the tests were conducted under drained conditions.

For tests on compacted ground an area approximately 1.5 m × 1.5 m × 1.2 m depth was excavated and subsequently back-filled with the same soil after breaking the cementation bonds. Backfilling was done in four layers with each layer compacted with a vibratory plate compactor. Water was employed in the amount of 10% to facilitate compaction to the initial insitu density as confirmed by the sand cone test. This moisture content is the optimum value determined from the Standard Proctor test. A relative compaction of 95% was achieved for all layers. Tests on compacted ground was performed 48 hours after completion of compaction.

ANALYSIS OF TEST RESULTS

Test results are plotted in Figures 6 to 8 in the form of pressure vs. settlement curves. In each figure the results of tests on natural ground, soaked ground and compacted ground are superimposed to show the effect of soaking and soil disturbance on the settlement and bearing capacity. Duplicate tests on natural ground revealed close similarity. In the next two sections the bearing capacity is first examined followed by an evaluation of the soil modulus and settlement under working loads.

Bearing Capacity

A close examination of Figures 6 to 8 indicates that failure has been progressive in nature with the failure point not well defined. Indeed punching failure has been observed in all tests and it appears to be a characteristic of weakly cemented sands. Figures 9a, 9b show, respectively, a close up view of a plate punching in the soil at failure and the failure zone after removal of the test plate. The type of failure observed is affected by the cementation level and other characteristics of the soil fabric. Further tests will be carried out later on a strongly cemented sand to examine the influence of the degree of cementation on the mode of failure and the bearing capacity.

For the determination of the failure pressure the slope tangent method was employed. The failure pressure is taken at the intersection of the tangents to the

PLATE LOADING TESTS

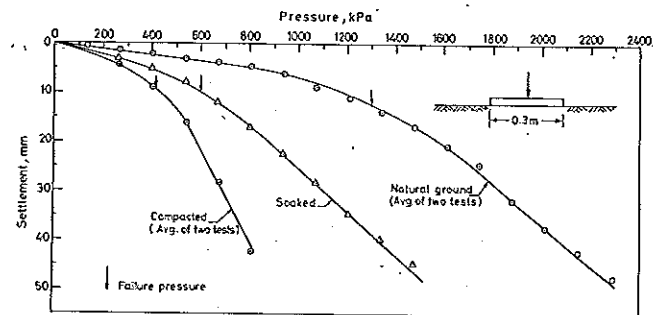


Fig. 6 Pressure-Settlement Curves, 0.3 m Plate

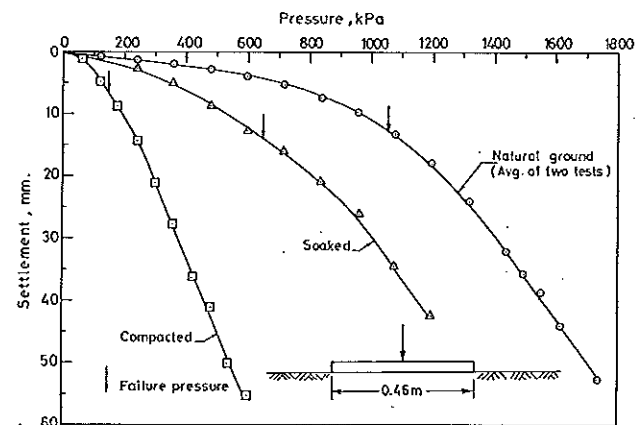


Fig. 7 Pressure-Settlement Curves, 0.46 m Plate

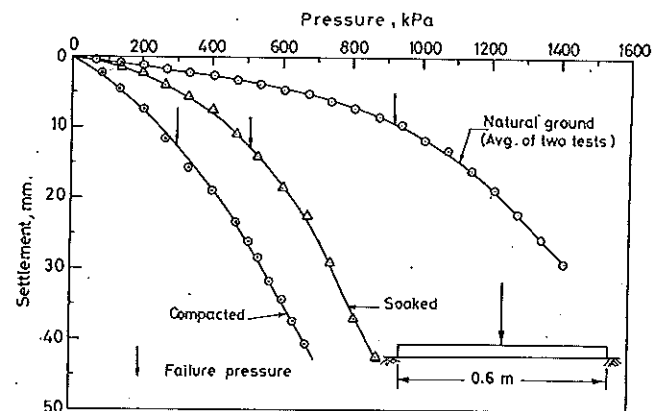


Fig. 8 Pressure-Settlement Curves, 0.61 m Plate

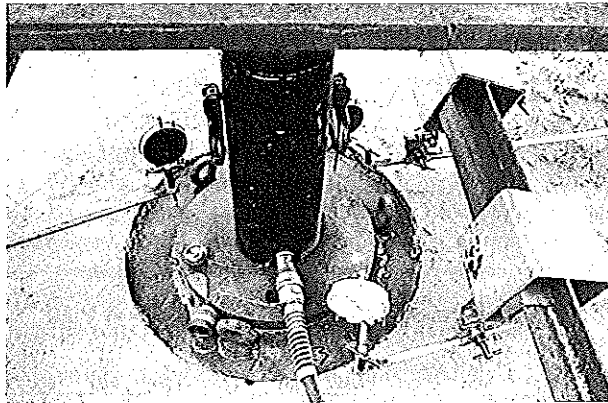


Fig. 9a A Close-up View of a Plate Punching in Cemented Sand at Failure

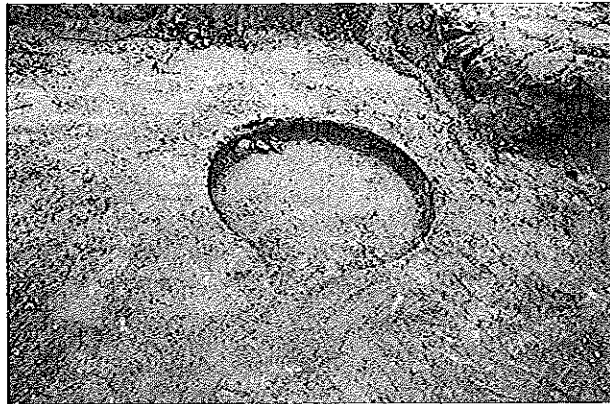


Fig. 9b Failure Zone After Removal of the Test Plate

initial and final portions of the pressure settlement curves as shown by vertical arrows in Figures 6 to 8. The failure settlement was taken as that corresponding to the failure pressure. A summary of the measured values and the corresponding settlement is given in Table 3. An examination of Figs. 6 to 8 reveals that soaking and ground disturbance lead to a significant reduction in bearing capacity and to increased compressibility. The bearing capacity under soaked ground is 55%, on average, of the corresponding value under natural moisture conditions. For the compacted ground this ratio decreases further to nearly 32% if the results on the 0.46 m plate, which are unusually low, are excluded.

Table 3 Summary of Bearing Capacity Values

Plate Diam m	Natural Ground				Soaked Ground				Compacted Ground			
	B.C ₁ kPa	S _r mm	S _r /B %	B.C ₂ kPa	S _r mm	S _r /B %	B.C ₂ /B.C ₁ %	B.C ₁ -B.C ₂ kPa	B.C ₃ kPa	S _r mm	S _r /B %	B.C ₃ /B.C ₁ %
0.3	1300	12.5	4.1	600	10	3.3	46.2	700	420	9.3	3.01	32
0.46	1060	12.5	2.7	650	14	3.1	61.3	410	150	7	1.5	14
0.61	920	9.1	1.5	520	13	2.1	56.5	400	300	12.5	2.1	32.6

B.C = Bearing Capacity

To analyze the test results the ultimate bearing capacity based on plasticity theory is employed. For circular foundations at ground level and ignoring the overburden pressure term, the bearing capacity is given by

$$q_u = c N_c \xi_c + 0.5 \gamma B N_\gamma \xi_\gamma \quad (1)$$

where c is the cohesion of soil, γ = unit weight of soil, B is the footing width or diameter, N_c and N_γ are dimensionless bearing capacity factors and ξ_c , ξ_γ are dimensionless parameters called shape factors.

For weightless soil, Prandtl (1921) and Reissner (1924) have found that

$$N_c = (N_q - 1) \cot \phi \quad (2)$$

$$N_q = e^{\pi \tan \phi} \tan^2 (\pi/4 + \phi/2) \quad (3)$$

The values of N_γ have been approximated by Vesic (1973) by the analytical expression

$$N_\gamma = 2(N_q + 1) \tan \phi \quad (4)$$

The shape factors for circular footings (De Beer 1970) are given by

$$\begin{aligned} \xi_c &= 1 + N_q/N_c \\ \xi_\gamma &= 0.60 \end{aligned} \quad (5)$$

For the measured value of $\phi = 35^\circ$, the values of N_c , N_q and N_γ as determined by the above relationships are 46.12, 33.30, and 48.03 respectively. The calculated shape factor ξ_c is 1.72. Substituting these values and the measured values of $c = 15$ kPa and $\gamma = 17.75$ kN/m³ into Eq.1 yields

$$q_u = 1190 + 255.76 B \quad (6)$$

For the values of $B = 0.3$ m, 0.46 m, 0.61 m, the calculated bearing capacities are 1268, 1309, 1347 kPa, respectively. Compared to the measured values of 1300,

1060, 920 kPa, agreement exists for the 0.3 m plate where the measured bearing capacity is 2% smaller than the calculated value. However, in terms of increasing plate size the calculated bearing capacity increases whereas the measured values decrease. To explain this evident contradiction, it is known that the bearing capacity factors particularly N_γ decrease with footing size (De Beer 1965; Hettler and Gudehus 1988).

Moreover, it can be seen that the contribution of the cohesion term to q_u is around 90% for the plates tested and hence q_u is highly dependent on c . It may thus be that as the stressed soil bulb increased with increased plate diameter, the average cohesion of the soil decreased as more imperfections, discontinuities, etc. are encompassed by the enlarged soil bulb. This is very likely because of the usual desiccation and higher strength near the ground level in a desert environment. Unfortunately, it could not be verified by test results since attempts to trim undisturbed samples from lower depths for direct shear tests failed as the samples crumbled due to their low strength.

For the soaked ground, it has been expected that the bearing capacity of all plates will decrease by the component due to soil cohesion which is 1190 kPa. However, examination of Table 3 shows that soaking has been partially effective with a reduction in bearing capacity of 700 kPa for the 0.3 m plate. For the 0.46 m and 0.61 m diameter plates the reduction in bearing capacity has been 410 kPa and 400 kPa, respectively.

The preceding results can be explained readily by the fact that the conditions in the field are different somewhat from those in the laboratory. In the laboratory full saturation of the sample ensures loss of cementation and simultaneous loss of the cohesion intercept. However, in the field, the degree of saturation within the entire stressed zone beneath the plates cannot remain 100% prior to or during the test. This is especially true for the larger size plates where the pressure bulb extends deeper, and hence the effect of soaking at the ground surface is less pronounced. Thus it can be stated that soaking or ground wetting in the field leads to partial loss of the cohesion intercept of cemented sands in lower depths beneath the plates which depends on the method and duration of soaking, the plate size, and the degree of cementation insitu.

The above results are qualitative in nature and illustrate the significant effect of soaking on weakly cemented surface desert sands. For more stable deeper deposits of moderately to strongly cemented silty sands the effect of soaking on settlement and bearing capacity is expected to be less significant. This will be examined shortly in a separate field testing program.

Tests on compacted soil indicated a significant reduction of the bearing capacity to only 32% of the corresponding values of undisturbed cemented sands (Table 3). It should be mentioned, that the SPT N values and dynamic cone tests in the compacted sands resulted in N values of 4 to 5 throughout the compacted zone. These values are 30-38% of the measured values for undisturbed ground (Fig. 1). Employing the laboratory test results, $c = 2$ kPa, $\phi = 35^\circ$ for compacted sands, the

bearing capacity values based on Eq. 1 are 235, 276, and 315 kPa compared to the measured values of 420, 150, and 300 kPa for the 0.3, 0.46, and 0.61 m plates, respectively. Good agreement is evident for the 0.61 m diameter plate. Large differences exist for the smaller plates which may be attributed to sensitivity of test results to some variations or nonuniformity of the soil density achieved by compaction. The presence of a small cohesion intercept in the laboratory samples is not surprising and indicates that minor cementation should be expected even after excavation and replacement. Re-cementation, although weaker than the original bonds can also occur in recompacted cemented soils.

Evaluation of the Soil Modulus.

The elastic modulus of cemented sands from plate loading tests is of interest in connection with settlement calculations. It can be computed from the theory of elasticity as

$$E = \frac{q \cdot B}{S} (1 - \nu^2) \cdot I_s \quad (7)$$

where q = intensity of contact pressure, S = settlement, and I_s is an influence factor which depends on the shape of the plate and its rigidity.

For a rigid circular plate $I_s = \pi/4 = 0.785$. Poisson's ratio was calculated from a combination of the equations

$$\nu = \frac{k_o}{1+k_o} \text{ and } k_o = 1 - \sin \phi \quad (\text{Jaky, 1948}) \quad (8)$$

where k_o is the coefficient of earth pressure at rest. For $\phi = 35^\circ$, the values of k_o and ν determined from Eq. 8 are 0.426 and 0.3, respectively.

Substituting the above values into Eq. 7 yields

$$E = \frac{q \cdot B}{S} (0.7147) \quad (9)$$

From the slope of the initial linear segment of the pressure settlement curves. Figs. 6 to 8, $\frac{q}{S}$ is determined. Employing Eq. 9 the modulus E is calculated for each plate size. A summary of the modulus values is given in Table 4. In tests on natural ground, two tests were carried out with an unloading-reloading cycle in the elastic range. From these tests the reloading modulus was determined as the secant modulus between zero stress after unloading and the point where the previous maximum stress had been reached again. Several interesting observations can be made with respect to Table 4. First, the ratio of the reloading to the initial moduli, E_R/E_N , is quite high being 2.73 to 3. Second, soaking resulted in a reduction of the soil modulus to nearly 50% of its natural insitu value for the 0.3 m and 0.46 m plates and

to 59% for the 0.61 m plate. If the ground is disturbed and cementation bonds are destroyed the soil modulus is further reduced to a value averaging 30% of the original natural soil modulus. It can be seen that E increases with increasing plate size, but the increase is not linear as predicted by Eq. 7 because the stiffness of the soil increases with depth.

Comparison of E Values with SPT Correlations

Table 4 Soil Modulus Values For Different Ground Conditions

Plate Size, m	Soil Modulus, E , kPa						
	Natural Ground			Soaked		Compacted	
	First Loading E_N	Reloading E_R	E_R/E_N	E_s	E_s/E_N	E_c	E_c/E_N
0.3	36300	108940	3	17430	0.48	13070	0.36
0.46	54460	-	-	26140	0.48	15080	0.28
0.61	62280	169840	2.73	37100	0.59	14530	0.23

- Not measured

Although the SPT test is not considered a suitable test for determining the strength and compressibility of cemented sands, it is of interest to compare the modulus values obtained herein with predictions based on empirical correlations with the SPT N values for cohesionless soils. Figure 10 presents relationships between the modulus and N -value established by D'Appolonia et al. (1970) and Trofimenkov (1974). The N -value corresponds to the average measured SPT in the zone of influence below the base of the footing. This zone was taken equal to the width of the footing.

Test results from Table 4 are superimposed on Figure 10 for the plate sizes 0.46 m and 0.61 m under natural and soaked ground conditions. As shown the results of the tests on natural ground nearly coincide with the correlation for overconsolidated sand whereas the results on soaked ground are in agreement with the correlations for normally consolidated sand. Thus it may be concluded that the cohesion intercept of cemented sands causes increased stiffness and a much higher modulus. However, if the ground is wet the modulus will decrease depending on the degree of saturation and the size of the plate or footing under consideration.

Test results from two other sites in Kuwait where plate load tests were carried out on natural cemented sands are shown in Figure 10. The tests were carried out on the natural ground without saturation. These results are in reasonable agreement with the upper bound curves given by D'Appolonia et al. (1970) and Trofimenkov (1974).

PLATE LOADING TESTS

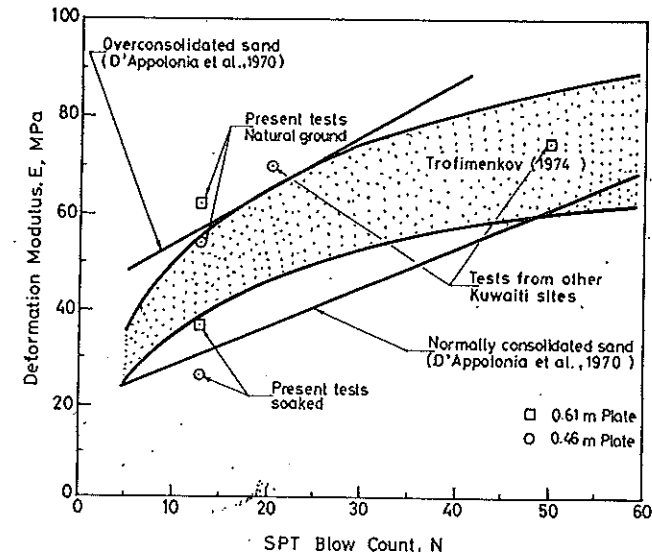


Fig. 10 Empirical Correlations between the Soil Modulus and SPT Blow Count

Effect of Plate Size on Settlement of Cemented Sands

Since the maximum size employed in the present study was 0.61 m, it is rather difficult to draw definitive conclusions regarding the increase of settlement with plate size. However, a clear trend may be observed. Fig. 11 shows the pressure settlement curves for both the 0.3 m and 0.61 m plates for natural and soaked ground conditions. As shown the settlement increases with size for all pressures increments. The settlement ratios for the different width ratios are given in Table 5 at several applied pressures within the initial linear segment of the pressure settlement curves.

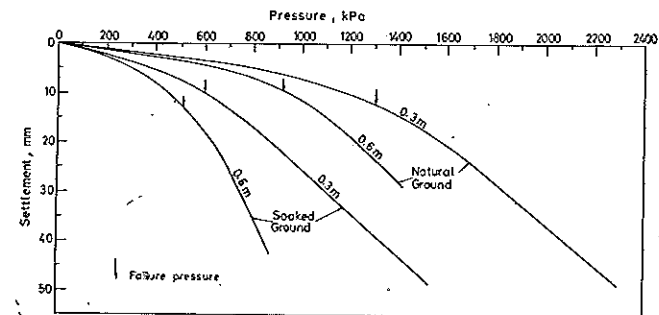


Fig. 11 Comparison of the Pressure - Settlement Curves for the 0.3 m and 0.61 m Plates

The average value of the settlement ratios is much smaller than what might be predicted using the well known Terzaghi and Peck relationship (1967) which has the form

$$S_B = S_1 \left(\frac{2B}{B+0.3} \right)^2 \quad (10)$$

where S_B is the settlement of a footing of width B m and S_1 is the settlement of a 0.3 m square steel plate, both having the same load per unit area. According to this equation $S_{0.46}/S_{0.3} = 1.46$ and $S_{0.61}/S_{0.3} = 1.78$ compared to the measured values of 1.02 and 1.18 for weakly cemented sands at natural conditions and 1.16 and 1.46 for soaked ground conditions (Table 5). It appears that as the cohesion component in cemented sand decreases, the settlement behavior moves closer to that of cohesionless sands.

The foregoing analysis is based on very limited tests. Additional tests employing larger size plates on soils with varying degrees of cementation are needed to explore the variation of settlement with the foundation width or diameter. While Eq. 10 is applicable for cohesionless desert sands (Ismael, 1985) it yields larger settlement than actual measurements for weakly cemented sands.

Table 5 Settlement Ratios for Different Width Ratios

Pressure q (kPa)	Natural Ground		Soaked Ground	
	$S_{0.46}/S_{0.3}$	$S_{0.61}/S_{0.3}$	$S_{0.46}/S_{0.3}$	$S_{0.61}/S_{0.3}$
300	1.0	1.143	1.08	1.33
400	1.06	1.22	1.25	1.60
500	1	1.166	-	-
600	1.02	1.181	-	-
Avg.	1.02	1.18	1.16	1.46

Not considered. It lies outside the linear elastic line.

CONCLUSIONS AND RECOMMENDATIONS

To examine the bearing capacity and deformation characteristics of cemented surface sands, a program of plate loading tests employing three plate sizes was carried out in the field. Based on the field and laboratory test results the following conclusions and recommendations are made:

1. Cemented sands are stiffer and less compressible than uncemented sands. Cementation leads to the existence of a cohesion intercept c while the angle of shearing resistance ϕ remains relatively unchanged.

2. Failure of weakly cemented sands under foundation loads is progressive and occurs in punching shear.
3. Soaking causes a reduction of the bearing capacity due to the partial loss of the soil cohesion. The reduction will depend on the degree of cementation, extent of soaking, and the size of footing. A reduction of 45% was measured in the present tests.
4. The deformation modulus of cemented sands has a magnitude similar to that of overconsolidated sands. However, soaking leads to up to 50% reduction in the modulus and increased compressibility. The reduction in modulus may be smaller for larger foundation sizes and denser deposits.
5. Ground disturbance, by excavation and replacement of cemented sand at the original insitu density, results in great reduction of bearing capacity and the soil modulus due to the loss of the cohesion intercept and the disturbance of the soil fabric by breaking the cementation bonds. The measured bearing capacity and soil modulus of compacted ground was 30% of the values of natural undisturbed ground.
6. Settlement increased at working loads with the plate diameter. However, the increase is smaller than the predicted values for noncemented sands. The settlement ratios for different width ratios are larger for soaked ground compared with natural ground conditions.
7. It is recommended to carry out additional tests employing larger plate sizes and larger range of width ratios at several sites possessing varying degrees of cementation to examine in more detail the effect of foundation width on the settlement of cemented sands.

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