Saturation Effects on Calcareous Desert Sands

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Saturation of the surface desert sands in Kuwait has caused some problems of settlement and local slope failures. The surface soils, consisting of calcareous windblown fine sand or silty sand, are sensitive to saturation. Laboratory and field testing programs were conducted to examine the effect of saturation on the shear strength parameters, settlement under load, and the ultimate bearing capacity. Laboratory testing consisted of basic properties, direct shear tests, and consolidation tests on undisturbed samples under unsoaked and soaked conditions. The samples were trimmed from block samples taken from five sites where slight interparticle cementations exist. Field tests included standard penetration, static and dynamic cone penetration tests, and plate load tests. All field tests were performed at one site on samples under unsoaked and soaked conditions. The laboratory test results indicate a reduction of the shear strength parameters due to saturation and increased compressibility or settlement under load. The field tests indicate a loss of 25 percent in both the ultimate bearing capacity and the allowable soil pressure for a given settlement criterion at the site most sensitive to saturation along a 35-km (22-mi) corridor. An average ratio of 4 was calculated between the cone penetration and the standard penetration resistance of the surface sands.

Calcareous sands exist in many parts of the world where arid or semiarid conditions prevail. These include the Arabian Peninsula *(1-3)*, southwestern United States and Mexico (*4*,pp.16–35), the Indian continental shelf (*5*,pp.113–140), South and South-West Africa (*6*,pp.296–309), and western Australia (*7*,pp.179–209). They are characterized by the presence of carbonates deposited at the points of contact between the particles *(8)* at a rate depending on local conditions and the geologic history of the deposit. As a result and depending on the grading characteristics, various degrees of cementation are produced at different locations and at different elevations at the same location. Calcareous sands encountered may be uncemented or weakly to strongly cemented *(5)*. The surface soil of Kuwait and major areas of the Arabian Peninsula usually consists of an uncemented calcareous windblown dune sand or silty fine sand to varying depths *(9, 10)*. This sand, although cohesionless, may have slight interparticle cementations in some areas, which facilitates excavation of shallow vertical cuts for temporary construction purposes.

With development and major construction in desert areas, particularly the Arabian Peninsula, interest in the properties and behavior of calcareous soils has grown. Recently, many problems of foundation and ground-floor settlement, deterioration, and local failures of slopes have occurred in Kuwait following saturation of the ground soils from heavy rain in the winter season or from the irrigation water and water leaking from underground pipelines. This pointed out the sensitivity of the local, moisture-deficient soils and the possible loss of strength and increased compressibility following saturation or soaking with water.

To examine the effect of saturation on the soil properties, a program of laboratory and field tests was carried out. Laboratory tests included basic physical properties and direct shear and consolidation tests on unsoaked and soaked undisturbed samples. All samples were trimmed from large blocks taken from five sites along a corridor 35 km (22 mi) long by 2 km (1.25 mi) wide where slight interparticle cementations existed. On the basis of the laboratory test results, a site was chosen for penetration and plate bearing tests, which were conducted at the in situ moisture conditions and after prewetting. Penetration tests included the standard penetration test (SPT) and dynamic and static cone penetration tests. All field tests were carried out at three locations within the site.

The results of the laboratory and the field tests are presented. The effect of saturation on the strength parameters, consolidation characteristics, and settlement under load is determined from laboratory test results. The reductions in both the ultimate bearing capacity and the allowable soil pressure based on a settlement criterion were determined from the results of plate bearing and penetration tests. A comparison is made between the laboratory and the field test findings.

BASIC SOIL PROPERTIES

Kuwait is located at the tip of the Arabian Gulf, as shown in Figure 1. It has an area of 17 800 km^2 (7,000 mi^2). The ground is in general a flat, gently undulating desert plain with occasional low hills, escarpments, and depressions *(11)*. A detailed examination of the soil conditions along the corridor shown in Figure 1 was carried out *(9, 10)*. The soil profile consists of a surface layer of calcareous windblown fine dune sand to a depth of 3 to 7 m (10 to 23 ft). This is underlain by a marine-deposited weakly to strongly cemented calcareous silty sand known locally as "gatch" and extending to a great depth over limestone bedrock. The Unified Soil Classification of the majority of samples in the upper layer is SP → SM, with the amount of fines passing the No. 200 sieve rarely exceeding 12 percent. The SPT values range between 15 and 35, and they generally increase in magnitude with depth. For the lower layer classified as SM → SC the SPT values generally exceed 50. In the majority of the boreholes drilled along the corridor, groundwater was encountered below the bottom of the upper layer *(9, 10)*.

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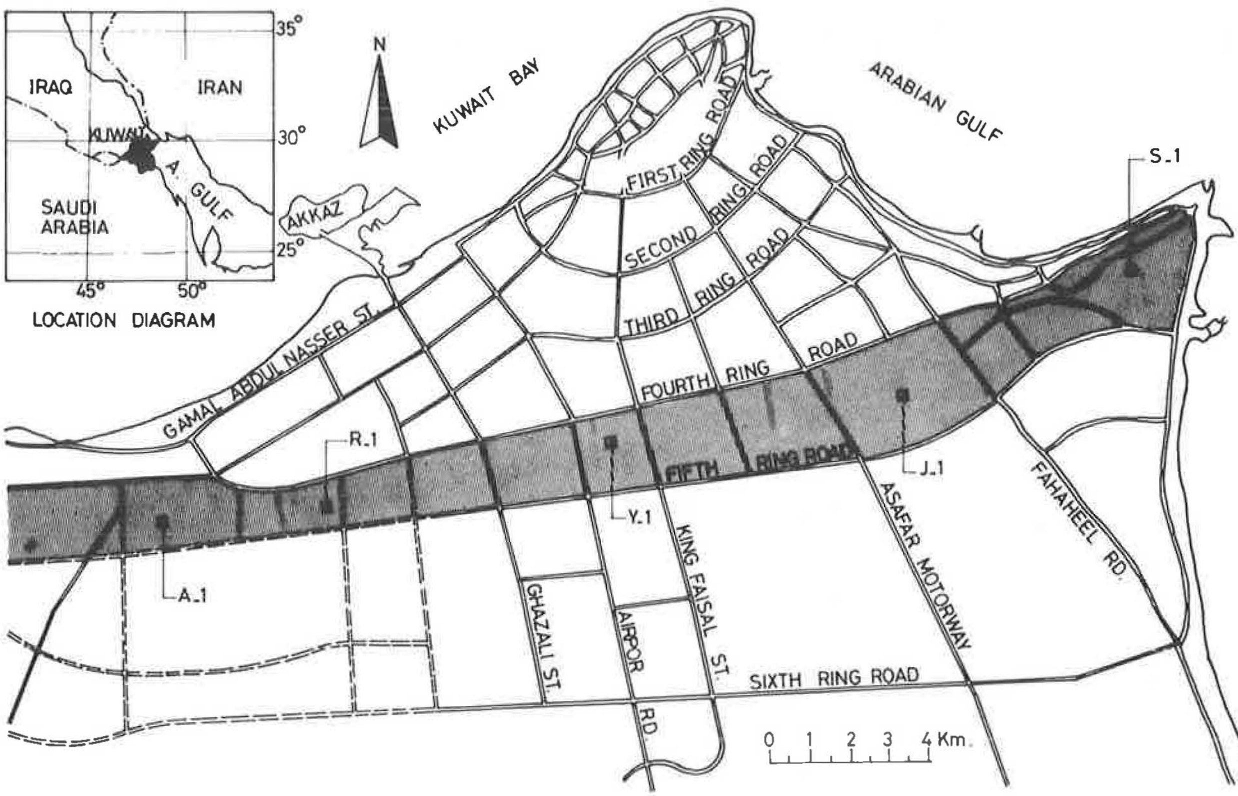

FIGURE 1 Site locations along corridor.

Five 0.5-m (1.6-ft) cubes of undisturbed block samples were cut from the bottom of excavation pits at a depth of 1.0 to 1.5 m (3.3 to 5 ft) using long knives and saws. The samples were taken from five selected sites, namely, A-1, R-1, Y-1, J-1, and S-1 (Figure 1). These samples were carefully wrapped in large plastic bags and transported to the laboratory for testing. The physical properties were determined first; they are summarized in Table 1. Indicated are the location, natural moisture content (w), bulk and dry unit weights (γ_b, γ_d), specific gravity (G_s), mean diameter (D_{50}), void ratio (e), degree of saturation (S_r), minimum and maximum dry densities ($\gamma_{d\ min}$ and $\gamma_{d\ max}$), relative density (R_d), the coefficient of permeability (K), and the percent of fines. Examination of Table 1 reveals the similarity of the soil properties at the different sites. The subsoils are in a relatively dry condition with the natural moisture content less than 2 percent. The mean diameter is nearly constant at 0.15 to 0.16 mm and the specific gravity varies in a narrow range between 2.67 and 2.72. The relative density, averaging nearly 70 percent, indicates a compact or medium dense sand. The values of the coefficient of permeability are in the range 10^{-3} to 10^{-4} cm/sec (ft/min), indicating a free-draining sandy soil.

The complete chemical analysis on bulk samples from the different sites is given in Table 2. As indicated, quartz constitutes the principal component. The amount of carbonates varies between 10 and 20 percent, mostly in the form of calcite (calcium carbonate). The arid environment of Kuwait and the excess of evaporation over rainfall lead to upward movement of groundwater and the concentration of soluble materials at or near the ground surface, enriching the soil with carbonate and gypsum and often leading to the formation of crusts of cemented soils *(1)*.

TABLE 1 PHYSICAL PROPERTIES

Site	Natural Moisture Content, w (%)	Bulk Unit Weight, γ_b (kg/m^3)	Dry Unit Weight, γ_d (kg/m^3)	Specific Gravity, G_s	Mean Diameter, D_{50} (mm)	Void Ratio, e	Degree of Saturation, S_r (%)	Dry Density (kg/m^3) $\gamma_{d\ min}$	Dry Density (kg/m^3) $\gamma_{d\ max}$	Relative Density, R_d (%)	$K \times 10^{-3}$ (cm/sec)	Percent Passing No. 200 Sieve
A-1	1.8	1717	1687	2.72	0.15	0.612	8.0	1521	1789	65.7	2.0	15.2
R-1	1.0	1727	1710	2.69	0.14	0.591	4.6	1541	1769	76.7	1.73	6.1
Y-1	1.1	1734	1715	2.72	0.15	0.586	5.1	1574	1783	70.1	0.93	8.9
J-1	0.9	1719	1704	2.72	0.16	0.596	4.1	1564	1792	64.6	0.26	5.2
S-1	1.5	1766	1740	2.67	0.16	0.563	7.2	1577	1779	72.4	0.64	9.7

Note: Samples were from a depth of 1.0 to 1.5 m.

TABLE 2 CHEMICAL ANALYSIS OF SOIL SAMPLES

Site	pH Value	Composition (%)										
		SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	CO_3	$CaSO_4$	$CaCO_3$	MgCO3	$\overline{SO}_3$	$\overline{C}1$
A-1	7.65	71.86	9.30	1.04	5.62	2.82	11.46	1.16	10.03	6.81	0.850	0.018
R-1	8.0	63.56	11.44	0.80	11.76	0.25	10.62	0.75	17.70	–[a]	0.44	0.021
Y-1	8.2	71.00	9.32	0.88	8.61	1.25	10.39	–[a]	15.38	2.1	0.052	0.018
J-1	–[a]	73.12	6.48	0.72	7.98	1.50	8.47	–[a]	14.26	–[a]	–[a]	0.018
S-1	8.7	76.90	5.62	0.52	6.30	1.08	9.45	1.01	11.25	3.06	0.025	0.021

Note: Samples were from a depth of 1.0 to 1.5 m.
[a]Not measured.

DIRECT SHEAR TESTS

Two sets of drained direct shear tests were conducted on samples both at in situ moisture content and after soaking (S_r = 100 percent) for 24 hr (ASTM D-3080). These tests were performed at a small rate of strain to ensure total dissipation of pore-water pressure during shear. As previously stated, all samples tested were undisturbed samples trimmed from the block samples possessing very weak cementations or bonding that could be easily crushed by a slight finger pressure.

Stress displacement curves and volume change during shear are plotted for R-1 sand in Figure 2 for samples tested under in situ moisture conditions. As shown, the stiffness and peak strength increase with an increase in the normal pressure. Volume change data indicate the development of strong dilation at small displacement, particularly with small normal pressures. The stress displacement data in Figure 2 show a ductile failure mode that has no significant drop after it reaches peak strength. However, by a comparison with the stress displacement data for soaked specimens at the same normal pressure, it will appear that soaked specimens have a much more ductile failure than the samples tested at in situ moisture content. This is shown in Figure 3, which indicates that the shear strength occurring at smaller displacement is higher for the unsoaked samples. The results are similar for other sites as well.

A summary of the peak-strength parameters under unsoaked and soaked conditions is given in Table 3 along with the residual unsoaked parameters and the predicted values based on the empirical relation between the SPT *N*-values and ϕ given by Peck et al. (*12*, p.310) The presence of a cohesion intercept *(C)* of about 4 to 24 kPa (0.6 to 3.5 psi) is due to the slight bond or interparticle cementation that exists at some locations. This may explain why windblown sand may stand steeply or even vertically in excavations for temporary con-

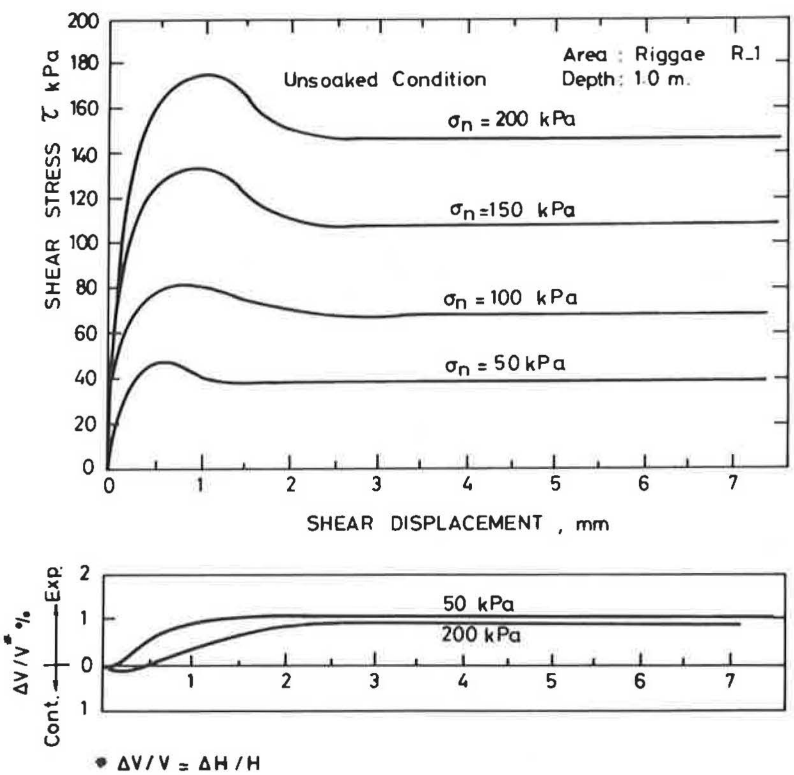

FIGURE 2 Stress displacement and volume change from unsoaked direct shear tests on R-1 soil.

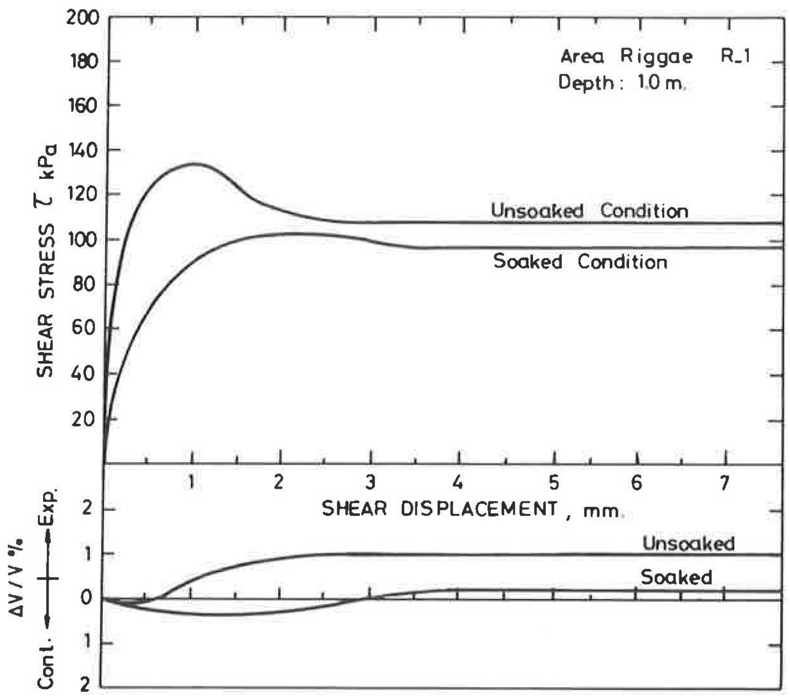

FIGURE 3 Comparison of stress displacement and volume change for unsoaked and soaked specimens (σ_n = 150 kPa).

struction. The soaked specimens have no cohesion and an angle of shearing resistance (ϕ) of 35.5 degrees compared with 40 degrees under unsoaked conditions. Plots of the shear stress versus normal stress showing the Mohr Coulomb envelopes for four of the test soils under unsoaked and soaked conditions are given in Figure 4. The fifth soil (A-1) could not be trimmed for testing because the samples crushed easily during preparation despite repeated trials. The loss of some strength because of saturation implies loss of bearing capacity and reveals the sensitivity of surface soils to excess moisture and the possible deterioration of slopes if subjected to heavy rain during the winter season.

A comparison of the residual strength parameters under unsoaked conditions with the peak soaked parameters indicates great similarity. This may be explained by noting that small interparticle cementations are destroyed once the peak strength has been reached *(13, 14)*, usually at small displacement levels, and soaking will have little extra effect, if any, in further reducing the strength.

The values of ϕ *(12)* range between 33 and 34 degrees and average 33.5 degrees. They are smaller than the measured soaked values by 2 degrees, and, as such, they are conservative lower-bound values.

CONSOLIDATION TESTS

Consolidation tests were carried out on 63.5-mm (2.5-in.) undisturbed samples. Initially the effect of soaking and the possible collapse potential were examined in a manner similar to that suggested by Knight *(15)*. A sample was loaded in the consolidation apparatus at its natural moisture content until a pressure of 200 kPa (29 psi) was reached. At the end of this loading, the specimen was flooded with water and left for a day, and the test was then continued to its maximum loading limit. The resulting curves for the test soils are shown in Figure 5. The collapse potential *(CP)* is defined as

$$CP = \Delta e_c/(1 + e_0) = \Delta H_c/H_0 \quad (1)$$

where

Δe_c = change in void ratio upon wetting,

TABLE 3 DIRECT SHEAR STRENGTH PARAMETERS FOR SURFACE SOILS

		Peak Parameters							
		Unsoaked		Soaked		Residual Unsoaked		
Site	SPT (*N* blows/0.3 m)	*C* (kPa)	ϕ (degrees)	*C* (kPa)	ϕ (degrees)	*C* (kPa)	ϕ (degrees)	Predicted ϕ' *(12)*
R-1	20	4	40.5	0	35.5	0	36	33.5
Y-1	20	0	40.9	0	35.5	0	37	33.5
J-1	18	4	41.7	0	35.8	0	36	33.0
S-1	23	24	40.4	0	35.5	0	35	34.0

e_0 = natural void ratio,
ΔH_c = change in height upon wetting, and
H_0 = initial height.

Jennings and Knight *(16)* have suggested some severity ratings for different values of the collapse potential, as follows:

CP (%)	*Severity of Problem*
0-1	None
1-5	Moderate
5-10	Normal
10-20	Severe
> 20	Very severe

From Figure 5 the collapse potential was calculated as 0.9, 1.0, 0.7, and 3.6 for Y-1, J-1, R-1, and S-1 soils, respectively. Comparing these values, which are essentially the strain caused by soaking, with the ratings of Jennings and Knight, it is concluded that there will be no problem with collapse in the first three soils, whereas the last soil (S-1) is considered to have moderate trouble with collapse. This may be explained by the fact that the surface soils consist of a relatively dry windblown dune sand—an aeolian deposit that is known to collapse *(17, 18)*. The possibility of collapse apparently increases with the presence of weak interparticle calcareous cementation bonds, which dissolve upon soaking and which vary from site to site despite the similarity of the grading characteristics.

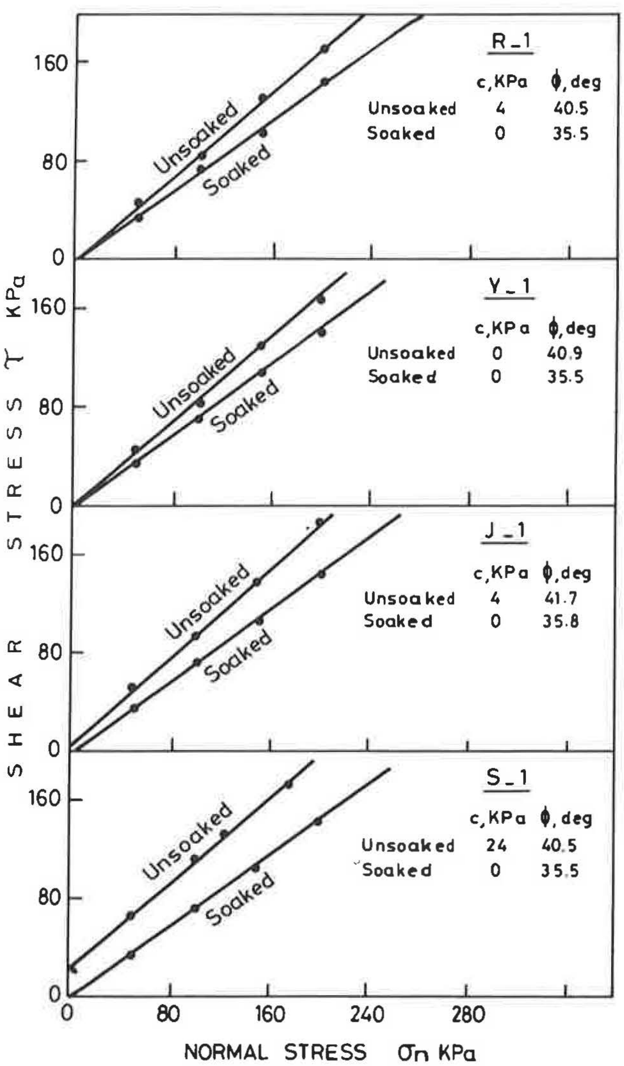

FIGURE 4 Mohr Coulomb envelopes for unsoaked and soaked specimens.

To investigate the effect of soaking on compressibility characteristics and settlement under load, double consolidation tests were performed on the four soils examined earlier *(12)* by using the method employed by Clemence and Finbarr *(17)*. Two identical undisturbed samples are placed in consolidometers under a 1-kPa (0.15-psi) load for 24 hr. At the end of this period, one sample is flooded with water, and the other sample is kept at its natural water content. Both samples are left for a further 24 hr. The test is then completed in the ordinary manner (ASTM D2435). The results of the current tests for J-1 and S-1 soils are shown in Figure 6. As is evident, the two curves do not start from the same point; the curve for the soaked samples is usually the lower one.

For settlement calculations, the total overburden pressure (P_0) at the depth of the sample is calculated and plotted on the e log p curves obtained from the two tests. The preconsolidation pressure *(P_c)* is found from the soaked curve and compared with P_0. For the case of normally consolidated soil in which P_c/P_0 is equal to 0.8 to 1.5, compression is considered to occur along the virgin curve and the natural moisture content curve is adjusted to the (e_0, P_0) point by drawing a curve parallel to the natural moisture consolidation curve, as shown in Figure 6 *(bottom)* for the S-1 soil. In the case of an overconsolidated soil, the adjustment to the curve follows ordinary settlement computation practice after the determination of the (e_0, P_0) point as shown in Figure 6 *(top)*.

If the load is increased by Δp, the unit settlement will consist of two components as follows:

$$S/H_0 = [\Delta e_s/(1 + e_0) + \Delta e_c/(1 + e_0)] \quad (2)$$

With reference to Figure 6, the first term is the unit settlement due to an increase in pressure (Δp) without change in the moisture content, and the second term is the unit additional settlement due to soaking. A comparison of the second term of Equation 2 for J-1 and S-1 soils under the same pressure increment of Δp = 200 kPa (29 psi) is evident from Figure 6. It indicates that settlement has nearly doubled for the S-1 soil, whereas it has increased by nearly 40 percent for the J-1 soil due to soaking. This appears to be in general agreement with what is expected in view of the results shown in Figure 5, which indicate the largest collapse potential for the S-1 soil. Each point of the curves shown in Figures 5 and 6 represents the average of two tests performed to obtain accurate results. These results indicate that the compression index is less than 0.10 and the term $C_c/(1 + e_0)$ ranges up to 0.05 for all test soils.

PENETRATION AND PLATE LOAD TESTS

To examine the effect of saturation in more detail, the site located in Salmiya (S-1), which displayed the greatest sensitivity to saturation, was selected for field testing. The site is flat, measures 100 by 200 m (330 by 660 ft), and has uniform

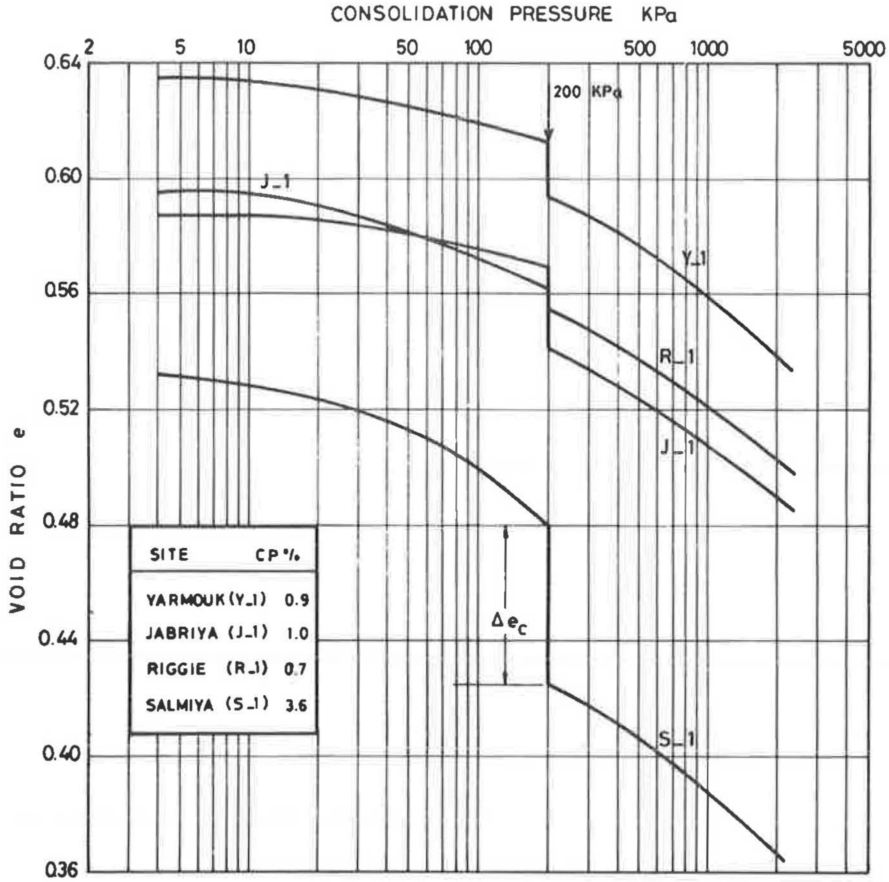

FIGURE 5 Collapse potential test results.

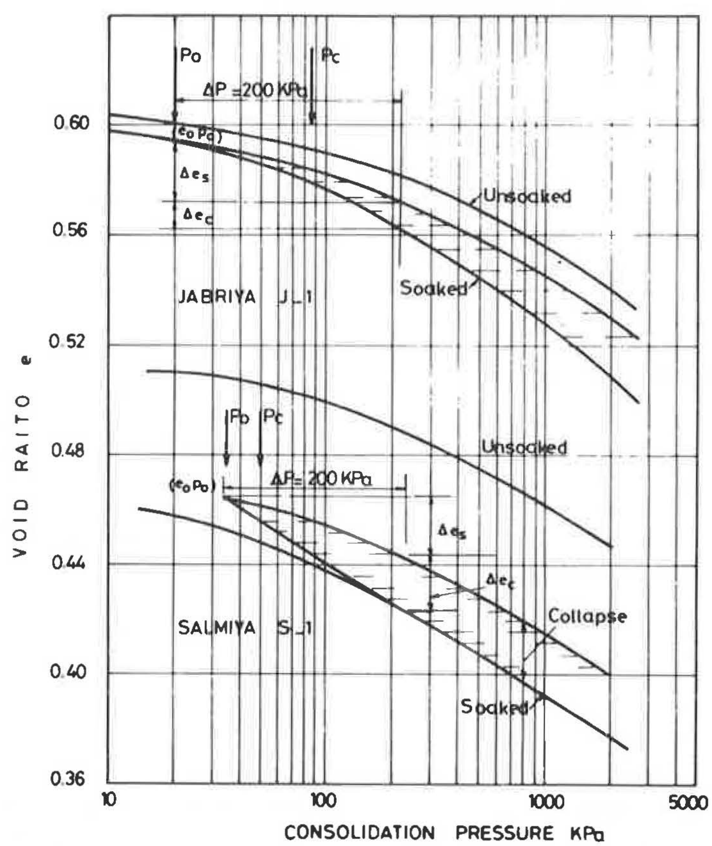

FIGURE 6 Double consolidation test results and adjustments for settlement calculations.

soil conditions. Three areas 40 m (131 ft) apart were chosen for penetration and plate bearing tests.

Penetration Tests

Standard penetration tests and dynamic and static cone penetration tests were performed in each area under in situ moisture conditions and after soaking. Wetting was achieved through four 4-m (13-ft) drill holes at the boundary of a 5 × 5-m (16 × 16-ft) area. These holes were filled with water and the surface area was saturated for 24 hr. Water was again poured before testing. Test results are summarized in Figure 7, which shows the soil profile, moisture content, and unit weight and plots of the various penetration test results under in situ moisture (unsoaked) conditions and after soaking. These results are the average of the data obtained at the three test locations.

The data in Figure 7 show that the windblown sand layer extends to a depth of 2.85 m (9.3 ft) only at this site. Within this layer the soaked penetration values are substantially reduced in comparison with the unsoaked values. By calculating the average values along the depth of the upper layer, it was found that the soaked SPT values and static cone penetration test values decreased to 62 percent of the corresponding unsoaked values. The dynamic cone penetration test values decreased to 67 percent of the unsoaked values. This shows marked consistency and implies a reduction in bearing capacity of the same order of magnitude.

To determine whether a relationship exists between the static cone penetration test values and SPT values as reported by several workers (*19; 20*,pp.473–499), the average values obtained from the three test areas under unsoaked and soaked conditions were compared. The results, summarized in Table 4, indicate a ratio of the cone penetration test values to the SPT values (q_c/N), in kilograms per square centimeter (tons per square foot), of 4.4 for the unsoaked conditions and 4.06 for the soaked conditions. This is similar to the values recommended by Sutherland *(20)* for fine sand and silty fine sand. Additional tests will be carried out at different locations along the test corridor to confirm the validity of this relationship at other sites with similar soils.

Plate Load Tests

At each of three areas selected, three plate load tests were conducted at a depth of 0.5 m (1.6 ft) in test pits 1.5 × 1.5 m (5 × 5 ft) according to ASTM D1194. The program at each area consisted of testing a 0.3-m (1-ft) plate under unsoaked conditions, soaked conditions at a pressure of 200 kPa (29 psi), and presoaked conditions in which saturation (S_r = 100 percent) was imposed before the test. The loads were applied by jacking against the rear wheel axle of a CMR 750-XL drill anchored down to 1.5 m (5 ft). Before each test was conducted, the test area was leveled properly to ensure that the plate rested on leveled, undisturbed soil. Soaking was achieved by a slow but

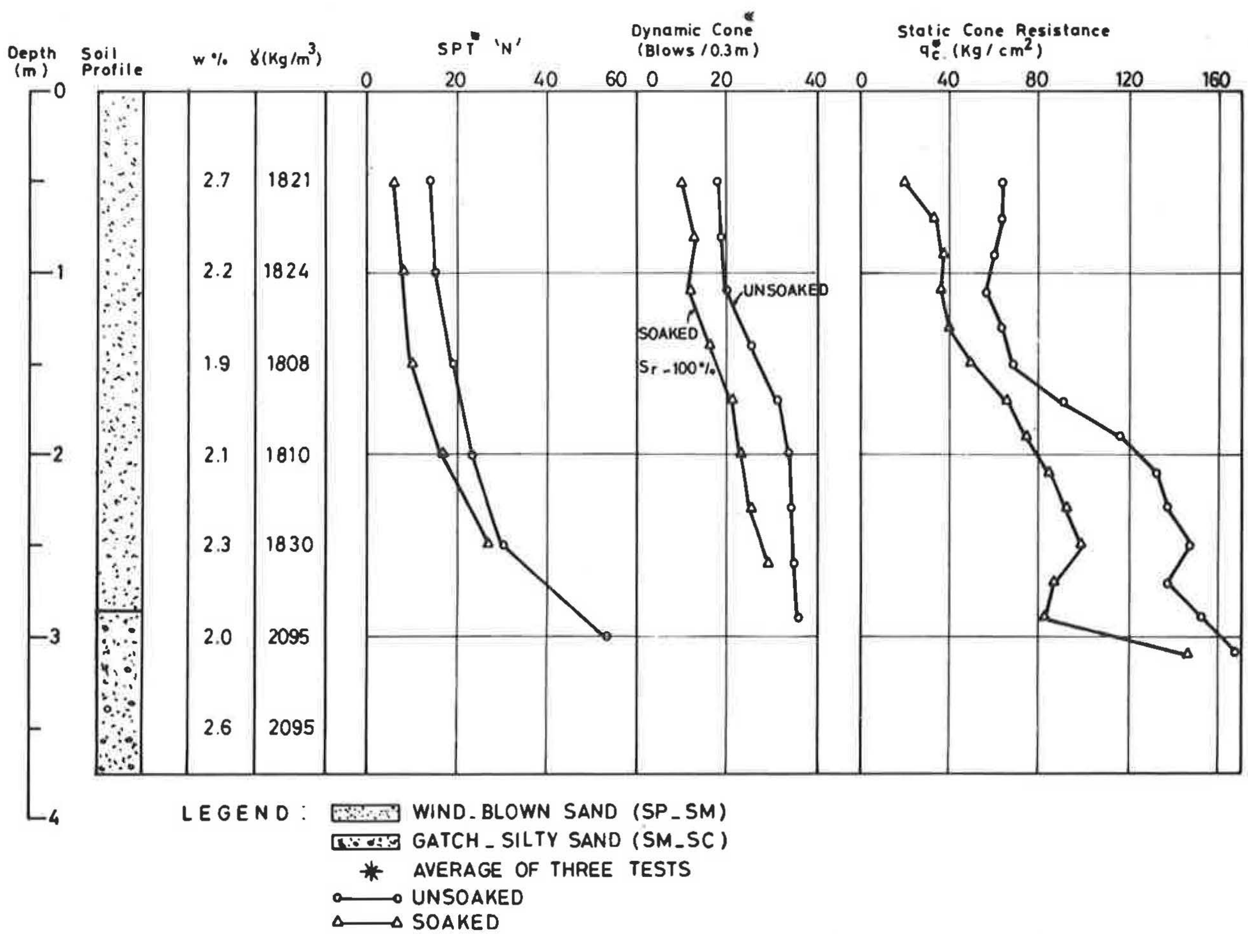

FIGURE 7 Soil conditions and penetration test data at Salmiya, Site S-1.

TABLE 4 SPT AND CONE PENETRATION TEST RESULTS AT SALMIYA (SITE S-1)

Depth (m)	Unsoaked SPT	Unsoaked CPT		Unsoaked Ratio	Soaked SPT	Soaked CPT		Soaked Ratio
0	–	–		–	–	–		–
0.5	14	64		4.572	6	20		3.334
0.7		64				34		
0.9		60				38		
1.0	15		58[a]	3.866	8		37[a]	4.626
1.1		56				36		
1.3		64				40		
1.5	19	68		3.580	10	50		5.00
1.7		90				66		
1.9		116				74		
2.0	23		124[a]	5.392	17		79[a]	4.648
2.1		132				84		
2.3		136				92		
2.5	30	148		4.934	27	98		3.630
2.7		136				84		
2.9		152				82		
3.0	39		159[a]	4.078	36		114[a]	3.166
3.1		166				196		

Note: Average ratio: unsoaked, 4.4; soaked, 4.06; therefore $q_c/N = 4.4$ (unsoaked), $q_c/N = 4.06$ (soaked). CPT = cone penetration test.

[a]Average of values enclosed in braces.

continuous pouring of water to wet the ground beneath the plate to a depth not less than twice its diameter. To do this, a 1.9-m^3 (500-gal) truck-mounted tank was employed, and the soaking period lasted for almost 2 hr after a pressure of 200 kPa (29 psi) was reached. For the tests carried out under presoaked conditions, soaking was maintained overnight before testing. The measured degree of saturation of the underlying soils (S_r) was 100 percent after soaking.

The results obtained from the three tests areas are plotted in Figure 8 in the form of pressure settlement curves for unsoaked and soaked conditions. Each curve is obtained from the average results of the three similar tests. For the tests in which partial saturation was imposed at 200 kPa, the data concided with those for the unsoaked curve before this load and were nearly identical to those for the saturated curve for loads larger than 200 kPa. Examination of Figure 8 reveals that the failure was progressive, a characteristic of local and punching shear failure. The failure load was taken at the point of maximum curvature on the pressure settlement curves as 580 kPa (84 psi) and 430 kPa (62 psi) for the unsoaked and soaked conditions, respectively. This signifies a reduction of 25 percent in the bearing capacity.

The allowable soil pressure for foundation design in sands is usually based on a permissible settlement of 25.4 mm (1 in.) (20). What then would the reduction be in the allowable soil pressure due to saturation for footings of different sizes?

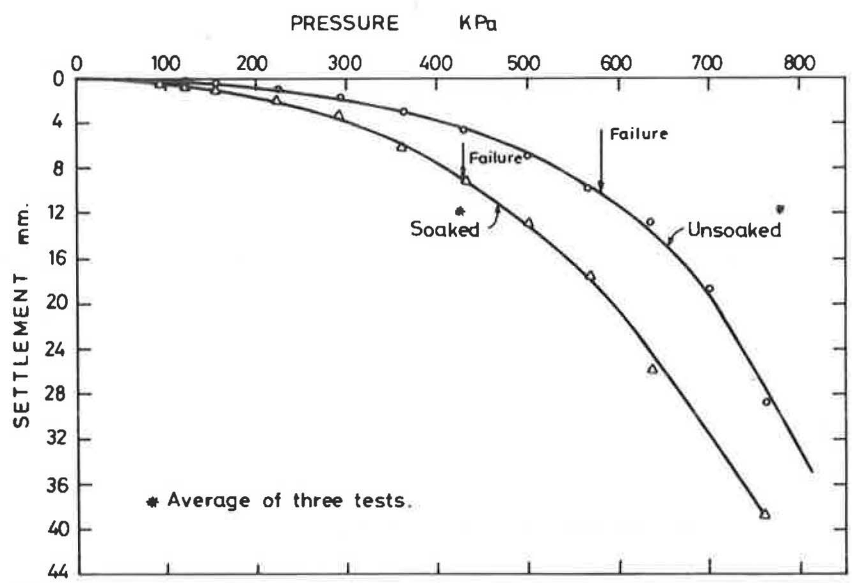

FIGURE 8 Plate load test results for unsoaked and soaked conditions.

To answer this question, one would first have to extrapolate from test-plate to full-size foundations. As explained by Parry *(21)*, this depends on how the soil stiffness (sand modulus) varies with depth. Recent field test results at seven sites in Kuwait on the same soil deposit as that examined here were in excellent agreement *(22)* with the following relationship proposed by Terzaghi and Peck (*23*, pp.494–496) between the settlement (S_B) of a footing of width B meters and the observed settlement (S_1) of a standard plate 0.3 m (1 ft) wide loaded to the same load intensity.

$$S_B = S_1\,[2B/(B + 0.3)]^2 \quad (3)$$

The implication of this relationship is that the settlement of a footing of any size would never exceed four times the settlement of a 0.3-m plate. The close agreement between plate tests *(22)* and Equation 3 is not surprising in view of the analysis made by Parry *(21)* for soil conditions with a progressive increase in the relative density with depth. For such soils, which have conditions similar to those for the present soils, the analysis indicated excellent agreement with Equation 3.

Employing Equation 3, the plate settlement corresponding to 12.7-mm and 25.4-mm permissible foundation settlement was determined for different sizes of footings ranging in width from 1 to 3 m (3.3 to 10 ft). Subsequently the allowable soil pressures were determined from Figure 8 for the unsoaked and soaked conditions. The results are summarized in Table 5 in which the ratio of the soaked to unsoaked soil pressure is given. As shown, the data indicate marked uniformity with an average ratio of 0.75. On this basis the decrease in the allowable soil pressure due to saturation is 25 percent, which is the same as the reduction in the ultimate bearing capacity.

For the same applied pressure of, say, 200 or 300 kPa (29 or 43 psi) the settlement is nearly doubled because of saturation (see Figure 8). However, if the pressure is reduced by 25 percent, the settlement will be reduced by half, thus offsetting the effect of saturation. It should be emphasized, however, that a flat reduction of this magnitude applied indiscriminately at all locations where similar soils exist will be conservative. This is in view of the high sensitivity to saturation prevailing at this test site compared with the others as demonstrated by the relatively large collapse potential value in Figure 5.

COMPARISON OF FIELD AND LABORATORY TEST RESULTS

On the basis of field and laboratory tests, it is evident that saturation of calcareous windblown sands leads to a reduction of shear strength and bearing capacity and an increase in compressibility and settlement under load. Moreover, it is possible to observe some similarity between the field and laboratory test results. For example, under an applied load increment of, say, 200 kPa the settlement due to saturation for S-1 soil doubles, as shown in Figure 6. The same trend is observed if the same pressure is applied to the test plate (see Figure 8). With regard to bearing capacity, the plate tests indicate a reduction of 25 percent. However, the penetration tests indicate a decrease in penetration resistance of nearly 40 percent. The laboratory direct shear tests indicating a decrease in ϕ from 40 to 35.5 degrees correspond to a reduction of the bearing capacity factors N_q and N_γ of 40 to 50 percent.

As previously stated, the S-1 site is considered to be the most sensitive to saturation. In absence of field tests at the other sites, a simple approximate approach may be adopted to account for saturation effects. Because saturation led to a unit settlement increase of ~40 to 50 percent for the J-1, Y-1, and R-1 sites compared with 100 percent for the S-1 soil, it is possible that the same relative effect holds true for bearing capacity and allowable soil pressure. If this is true, the allowable soil pressure would be reduced by 12.5 percent for the other three sites. This needs to be confirmed, however, by additional field tests at the other sites.

CONCLUSIONS

On the basis of the laboratory and field test results presented in this paper, the following conclusions are drawn:

1. Calcareous windblown sands or silty sands cover major areas of Kuwait and other desert areas. These surface sands, consisting mainly of fine sand, are usually not cemented; however, they possess slight interparticle cementations at some locations.
2. Sensitivity to soaking was examined by laboratory direct

TABLE 5 ALLOWABLE SOIL PRESSURES FROM PLATE LOAD TEST RESULTS

Width of Footing (m)	Permissible Settlement (mm)	Equivalent Plate Settlement (mm)	Allowable Soil Pressure (kN/m²) Unsoaked Conditions	Soaked Conditions	Ratio: Soaked/Unsoaked
3	12.7	3.84	405	290	0.716
2	12.7	4.2	420	305	0.726
1.5	12.7	4.57	440	320	0.727
1	12.7	5.36	465	345	0.742
3	25.4	7.68	530	400	0.755
2	25.4	8.4	545	415	0.768
1.5	25.4	9.14	560	430	0.768
1	25.4	10.73	590	460	0.780

Note: Average ratio = 0.747-0.75.

shear tests and double consolidation tests on undisturbed samples. A decrease in the angle of shearing resistance of ~5 degrees and an increase of the unit settlement under load were recorded. The additional settlement due to collapse varies from site to site; however, at the majority of the test sites it poses no major problem.

3. Plate load test results indicate that saturation leads to a reduction of 25 percent of both the ultimate bearing capacity and the allowable pressure on the basis of a settlement criterion of 25.4 or 12.7 mm (1 or 0.5 in.). This was found for the site most sensitive to saturation. At other locations smaller reductions may be applicable.

4. Standard penetration and static cone tests show a decrease of 38 percent in the average resistance due to 100 percent saturation. This is somewhat larger than the reductions deduced from the plate tests.

5. Design of earth structures in this soil should be based on the soaked strength parameters to account for the effect of increase in the degree of saturation.

6. The ratio of the cone penetration to the standard penetration resistance in kilograms per square centimeter (tons per square foot) was found to range from 4 to 4.4 under soaked and unsoaked conditions. This is based on testing at one site only and should be confirmed by additional field testing before its use in design calculations.

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