Some Properites of Remoulded Carbonate Soils

Quelques Propriétés de Sols Carbonés et Remaniés

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SYNOPSIS Tests were carried out on two selected carbonate soils (a siliceous carbonate sand and a marl) from the vicinity of Dammam, Saudi Arabia. The experiments included CBR tests at different water contents, undrained trixial, and repeated load tests on compacted samples. The data permitted the use of linear relationship (on the log-log plot) to represent the increase of axial plastic strain with repeated load cycles. A critical stress, about 40% of the undrained shear strength, separated samples' plastic and resilient deformation behaviour. The resilient modulus decreased with increased stress; and approached a constant value after 10^3 cycles at stresses lower than the critical.

INTRODUCTION

The extensive use of carbonate soils in Eastern Saudi Arabia coupled with lack of published information on their engineering characteristics have led to speculations and wide discrepancies in their description, classification and behavior when in use as subgrade material. Much of the difficulty arises because these deposits (referred to locally as Marl) exhibit a great deal of variations in their origin, texture, mineral composition and engineering properties.

The geological aspects of sedimentary rocks in the Arabian Gulf area have been discussed by Kassler (1973), Sarnthein and Walger (1973), Purser and Evans (1973), Chapman (1973) and Al-Sayyari and Zotl (1978). Reference should be made to the work of Fookes (1978) on the nature of soils in the Middle East and associated foundation problems.

The work presented in this paper is part of a research aims at providing typical laboratory test results on remoulded carbonate soils from the vicinity of Dammam, Eastern Saudi Arabia to help delineate their relevant engineering properties and establish a data base for their eventual classification. The specific data provided here are limited to two soils and comprise the results of (i) California Bearing Ratio tests as a function of water content, (ii) undrained triaxial tests on laboratory compacted samples and (iii) permanent and resilient deformation behavior resulting from the application of constant level of repeated load.

SOIL DESCRIPTION

General

The carbonate soils may form as a result of physical and chemical weathering of parent carbonate rocks (e.g. limestones, dolomites, carbonate sandstones, etc.). Often their original characteristics are obscured due to their burial with detrital sediments. The carbonate minerals tend to be soluble, chemically reactive and easily recrystalizable. According to Fookes and Higginbottom (1975), the main variables which occur in such deposits and seem to affect their engineering properties are: (1) Mineral composition, in terms of percent carbonate and to what extent is the carbonate admixed with clay minerals or quartz, (ii). If mainly carbonate, whether the carbonate is calcite ($CaCO_3$), aragonite ($CaCO_3$), dolomite ($CaMg(CO_3)_2$) or siderite ($FeCo_2$), (iii). The origin and processes of forming, (iv) Grain size, and (v) Degree of cementation normally reflected by in situ strength. Cementation in carbonate soils, normally caused by changes in temperature and/or carbonate dioxide concentration, may take place concurrently with deposition (Fookes and Higginbottom, 1975). This buildup in cementation is a positive attribute of carbonate soils when in use as a compacted fill. However, loss of porosity may follow cementation due to creep and recrystallization of particles particularly under relatively high overburden pressures.

Preliminary reconnaissance work in the vicity of Dammam, Eastern Saudi Arabia, has revealed the existance of different types of these carbonate deposits in terms of colour, composition, and gradation. The majority of these surface soils exist in cemented layers that overlie highly weathered carbonate rocks. Some may be burried by up to two meters of recent windblown sediments.

Economy in present day road building practice usually dictates that these carbonate soils be used as a capping layer on top of embankments built of compacted dune sand.

Soils selected

The two soils selected for this work were obtained from two separate locations south of the city of Dammam. They represent two of the predominantly occuring carbonate soils of the landscape. Both samples were obtained from a depth approximately 1.5 meters below ground surface and were initially identified by their colour and texture. Table I lists the various properties determined. The two soils tested, referred to here as Soil A

TABLE 1

Properties of Carbonate Soils Tested

VARIABLE	DATA Soil A	Soil B
Grain size analysis		
Percentage passing No. 200	23.5	20.5
Percentage silt size (0.05mm-0.002mm)	11.1	7.5
Percentage clay size (<0.002mm)	4.6	7
Liquid limit	Non- plastic	44
Plasticity index	Non- plastic	9
Standard Proctor max. dry density kg/m^3	1730	1674
Standard Proctor optimum moisture content, %	17.3	19.4
Specific gravity	2.71	2.80
pH	9.4	8.8
Organic carbone, %	3.70	.46
Description	Siliceous carbonate sand	Marl

and Soil B, contained approximately 70% calcium carbonate. Soil A is a non-plastic sand with 23.5% passing No.200 sieve (.074mm). Soil B,on the other hand, exhibited some plasticity due to the presence of approximately 10% clay. Based on the classification system proposed by Fookes and Higginbottom (1975) Soil A may be labeled as siliceous carbonate sand since it contains a small proportion of silica grains; while Soil B can be termed marl which is a simple binary mixture of calcium carbonate and clay. The indurated equivalents of Soils A and B are respectively limestone and argillaceous (or clayey) limestone.

CALIFORNIA BEARING RATIO (CBR) TESTS

CBR test is widely used in Saudi Arabia for evaluation of subgrades. In attempting to determine the effect of moulding water content on CBR value, it was necessary to perform CBR tests on samples with different water contents. For each selected moulding water content four CBR samples were prepared by compacting the wetted soil in standard CBR moulds, in three layers, applying 56 blows per layer to reach a dry density equivalent to that of the standard Proctor test at the selected moisture content. The moulding moisture contents ranged from well below optimum to two or three water contents on the wet side of optimum. At each moisture content selected, two specimens were subjected to CBR penetration test (ASTM D 1883, 1974) directly after moulding, and two were deffered until they had soaked in water for 96 hours under a surcharge of 2.5 KPa. During soak, sample's height was monitored at different time intervals to determine swell. Figs. 1 and 2 show the variation in dry density (standard and modified Proctor), soaked and unsoaked CBR, final water content after 96 hours of soaking, and percent swell (in case of soil B) for different moulding water contents. Each point plotted represents the average of two tests.

The following observations can be made with respect to the results plotted in Figs 1 and 2: (i) For both soils tests, maximum CBR occured on the dry side of optimum. For soil A the maximum

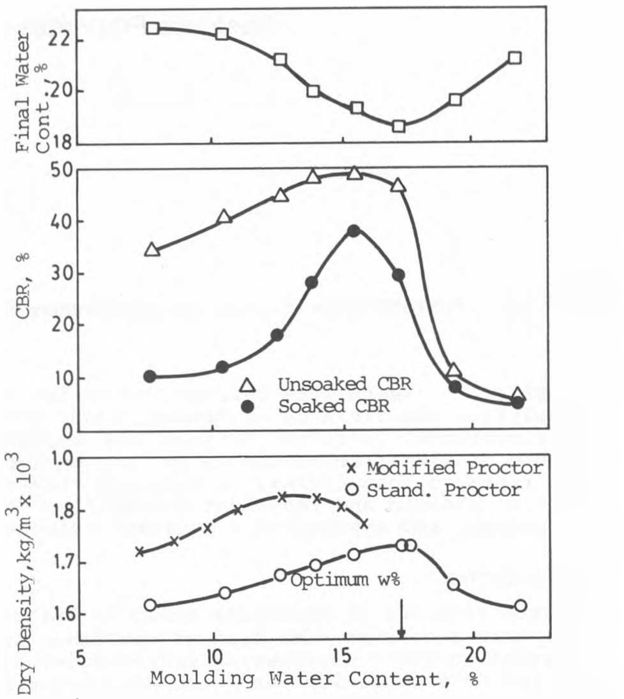

Fig.1 Variation of Dry Density, CBR, and Final Water Content with Moulding Water Content(Soil A)

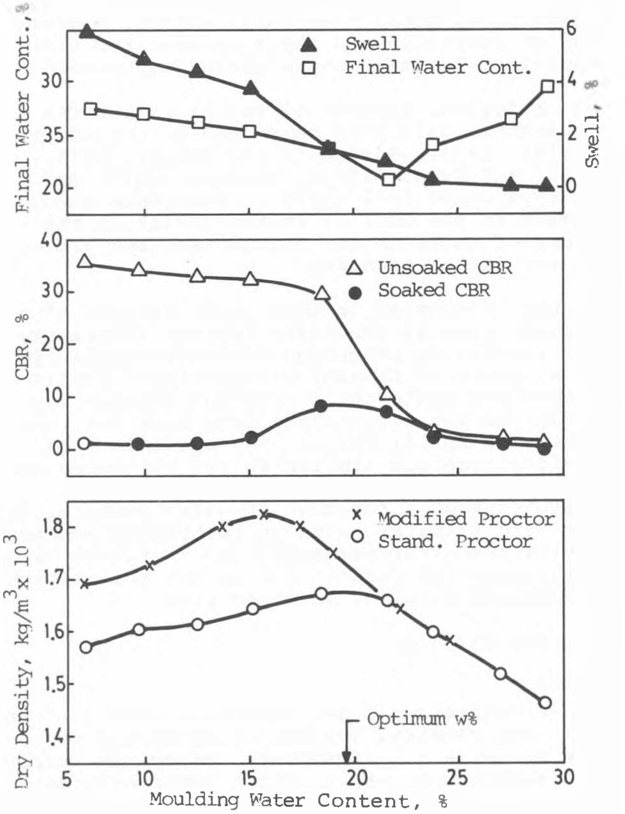

Fig.2 Variation of Dry Density, CBR, Final Water Content, and Percent Swell with Moulding Water Content (Soil B)

soaked and unsoaked CBR are well defined and correspond to a water content 1.5% drier of optimum as determined by standard Proctor. (ii) Increasing moulding water content dry of optimum, increased soaked and unsoaked CBR values of soil A whereas it lowered the unsoaked CBR value of soil B. (iii) As expected, and for both soils tested the lower the initial moulding water content the higher its water absorption after soaking. This change is reflected in the final water contents. (iv) Soaked samples of Soil B, dry of optimum, exhibited considerable swelling, i.e. measured percent swell dropped from 6% at approximately 6% water content to 1% at optimum water content. No appreciable swell was detected in the case of soil A.

UNDRAINED TRIAXIAL AND REPEATED LOADING TESTS

Specimen preparation, equipment and procedures

Conventional undrained triaxial tests and repeated load tests were carried out on laboratory compacted samples moulded by dynamically compacting the wet soil into cylindrical moulds (71mm in diameter and 150mm high) in three equal layers. The compaction energy was imparted by means of 2.5 kg. hammer falling 305mm. The number of blows used per layer was 30 for Soil A and 15 for Soil B. This level of energy produced samples of densities and water contents equivalent to maximum dry density and optimum water content of standard Proctor test. Test samples' average dry densities and moulding water contents were respectively: 1730 kg/m and 17.3% for Soil A, and 1674 kg/m^3 and 19.4% for Soil B. After moulding, samples were extruded, measured, weighed, sealed, and stored in a humidifier at a temperature of 12^{o}C for a duration of one week prior to testing. All samples were tested at room temperature (23 ± 2^{o}C) after equilibrating for a period of 8 hours.

Undrained triaxial tests were performed on test samples at a deformation rate of 1.3mm/min. under confining pressures of 35, 70, and 140 KPa. The data, in terms of axial stress difference versus axial strain are shown in Fig. 3.

The repeated load test equipment was similar to that described by Akili and Monismith (1978). The axial load was applied by means of a Bellofram rolling diaphragm air cylinder. The load duration selected was 0.1 sec. at a frequency of 10 cycles per minute. This arrangement permitted the axial stress to increase from the static confining pressure of σ_3 to σ_1 and then decrease back to σ_3 in each loading cycle. All repeated load tests were carried out at a confining pressure of 35 KPa to simulate a typical subgrade condition. Each repeated load test was performed at a selected constant axial stress difference until failure occured, sample deformed excessively, or needed information was obtained.

Plastic strain

The plastic strains measured during repeated load testing were plotted versus number of stress cycles, N, and are shown in Figs. 4 and 5. The repeated stress level is expressed as an axial stress difference factor, X, defined as a ratio of the repeated axial stress difference divided by the axial stress difference that would cause failure in the static undrained test at the same

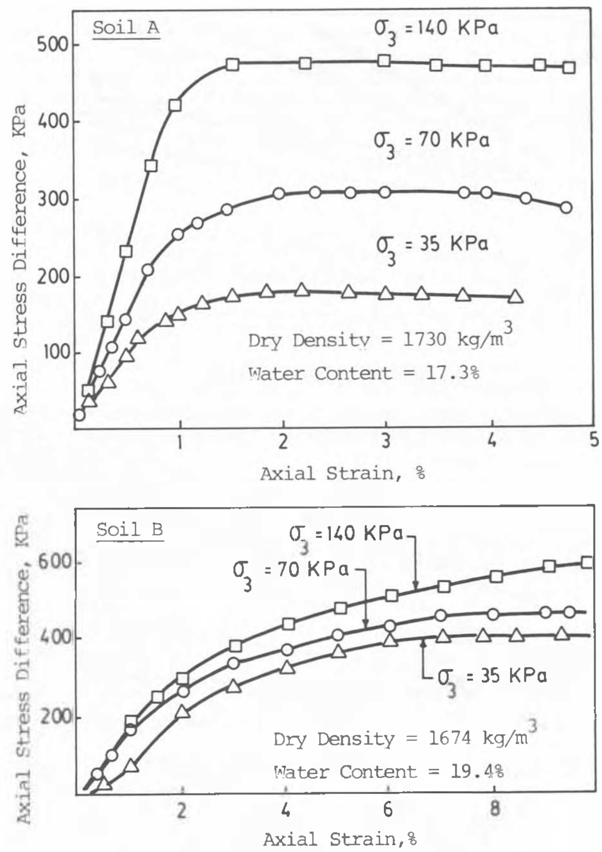

Fig.3 Undrained Triaxial Shear Test Data at Different Confining Pressures.

confining pressure (Raymond et al, 1979).

The log-log plots which appear to fit the data reasonably well (Figs. 4 and 5) will permit its presentation, for both soils tested, in the following form:

$$\varepsilon_a^p = KN^b \qquad (1)$$

Where ε_a^p is plastic axial strain and the terms K and b^a are experimentally determined coefficients. The above equation has the same form as that determined for other soils and materials in other investigations (Monismith et al, 1975; Akili and Monismith, 1978). The coefficients K and b for the test series shown in Figs. 4 and 5 are summarized in Table II. Both coefficients are dependent on axial stress difference factor (X), b to a larger extent than K. As shown in Figs. 4 and 5, the number of repeated stress cycles required to reach a certain level of plastic strain decreased as the stress difference factor increased. It was observed from the limited experimental data, that values of the stress difference factor greater than approximately 0.40 caused the rate of plastic strain of Soil A samples to progressively increase until a tertiary stage was reached and terminated with sample's failure. Samples in this category had a well defined failure plane. This type of failure has been reported by others in undrained repeated loading studies

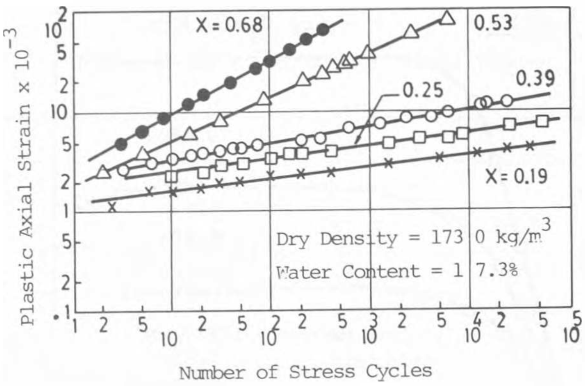

Fig.4 Plastic Axial Strain Versus Number of Stress Cycles at Different Stress Levels (Soil A)

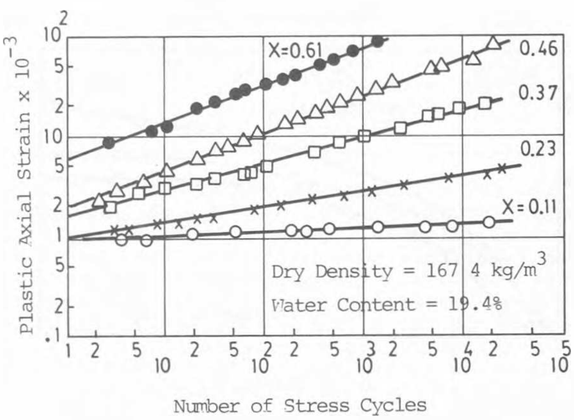

Fig.5 Plastic Axial Strain Versus Number of Stress Cycles at Different Stress Levels (Soil B)

(Larew, 1960; Mitchell and King, 1977; and Raymond et al, 1979).

In the case of Soil B, samples deformed excessively when the stress difference factor exceeded 0.40, but no shear failure was exhibited in any of the samples tested. However, this does not preclude the possibility of shear failure since the number of tests were limited and tests had to be terminated on or before 2 x 10^4 cycles. In any case, large plastic strains are usually not tolerated in subgrades and it is often necessary to limit plastic deformation to a certain magnitude, somewhere in the range of 2 to 5%. Table III shows the estimated number of cycles to cause 2% plastic axial strain for both soils tested. Some of the numbers in table III were arrived at by linear extrapolation of the data in Figs. 4 and 5.

Resilient response

Soils under repeated loading undergo recoverable deformation in addition to plastic deformation. This resilient behavior is characterized by the resilient modulus, M_R, which is calculated by dividing the repeated axial stress by the resilient (recoverable) axial strain. Fig. 6 illustrates the influence of the stress difference factor on resilient modulus after 200 load cycles. Figs. 7 and 8 show the change in resilient modulus with increasing load cycles at different stress difference factors.

TABLE II

Plastic Strain Determinants Versus Axial Stress Difference Factor (X)

	Axial Stress Diff. Factor X	Coefficients* $K \times 10^{-3}$	b
Soil A	.19	1.25	.10
(Siliceous carbonate sand)	.25	1.90	.15
	.39	2.25	.23
	.53	1.92	.53
	.68	2.75	.92
Soil B	.11	.95	.08
(Marl)	.23	.98	.13
	.37	1.60	.32
	.46	2.00	.45
	.61	5.8	.62

* $\varepsilon_a^p = KN^b$

TABLE III

Estimated Number of Repeated Load Cycles to Induce 2% Plastic Axial Strain

	Axial Stress Difference Factor X	Number of Load Cycles
Soil A	.19	2.5 x 10^9
(Siliceous carbonate sand)	.25	8 x 10^7
	.39	4 x 10^5
	.53	210
	.68	25
Soil B	.11	> 10^{20}
(Marl)	.23	6 x 10^7
	.37	10^4
	.46	460
	.61	25

The resilient modulus, for both soils tested, decreased with increasing magnitude of repeated axial stress. Similar behaviour was reported by Thompson and Robnett (1979). It is interesting to note that the resilient modulus of Soil A samples increased with increasing number of stress cycles; while the modulus of Soil B samples decreased with increased number of cycles. This varied behaviour is likely to be attributed to strain hardening effect in the case of Soil A,

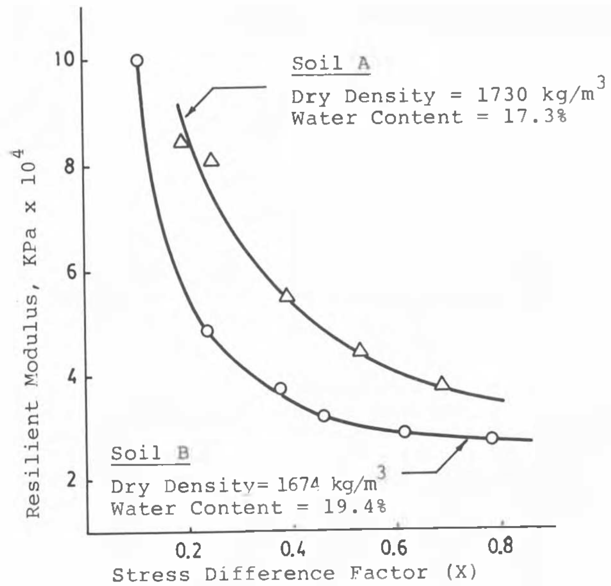

Fig.6 Resilient Modulus Versus the Stress Difference Factor at 200 Stress Cycles

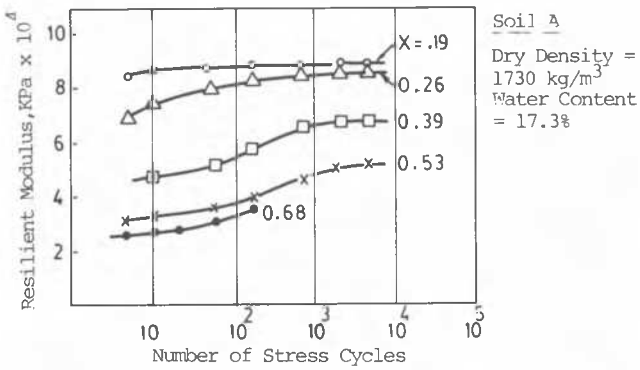

Fig.7 Variation of Resilient Modulus With Number of Stress Cycles (Siliceous Carbonate Sand)

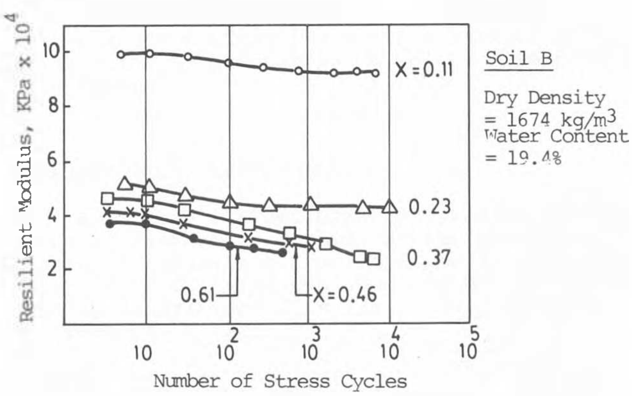

Fig.8 Variation of Resilient Modulus With Number of Stress Cycles (Marl)

while strain softening appears to prevail in the latter. The resilient modulus for both soils approached an almost constant value between 10^3 10^4 cycles under stress difference factors lower than 0.40. Samples subjected to higher values failed before reaching 10^3 cycles.

SUMMARY AND CONCLUSIONS

California Bearing Ratio (CBR), undrained triaxial and repeated loading tests were carried out on two remoulded carbonate soils, a siliceous carbonate sand (Soil A) and a marl (Soil B) from the vicinity of Dammam, Eastern Saudi Arabia.

Appreciable difference in behaviour between the two soils was exhibited in the CBR test at different moulding water contents. The siliceous carbonate sand followed the pattern of a cohesionless soil while the marl's performance was much affected by its plasticity.

Repeated loading tests on dynamically moulded samples of equivalent densities and water contents to standard Proctor, have revealed the following:

1. Under repeated loading, the plastic deformation(ε_a^p) of both soils can be represented by the following relationship:

$$\varepsilon_a^p = KN^b \qquad (1)$$

   where K and b are dependent on repeated axial stress.

2. For stress levels above a certain critical stress, plastic strains increased with repeated stress application until failure of sample occured (Soil A), or sample deformed excessively (Soil B). At stress levels below the critical stress, plastic deformation continued to increase with increased number of stress cycles even at very low stresses. For both soils, the critical stress was approximately 40% of the undrained shear strength at the same static confining pressure.

3. For both soils tested, the resilient modulus decreased with increasing stress level. Increasing the number of stress cycles, led to an increase in the resilient modulus of Soil A and a decrease in the modulus for Soil B. Both soils approached a constant modulus value after 10^3 cycles under stresses lower than the critical stress.

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