Deformation Characteristics of Subgrade Soils in Kuwait

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Comprehensive laboratory triaxial dynamic testing of subgrade soils in Kuwait was conducted to determine the engineering parameters for pavement design and construction. A literature survey and a review of ongoing road construction projects indicated that the subgrade soils were predominantly granular (A-1-b and A-2-4) according to the AASHTO classification system. A chemical and mineralogical analysis indicated that quartz is the principal component. Gypsum, calcium magnesium, and sodium sulphate are also present in varying proportions. The results of California bearing ratio (CBR) and direct shear tests indicated that these soils exhibited the characteristics of a high-performance subgrade. The angle of internal friction ranged from 20 to 40°, with an average value of about 30. The soaked CBR ranged from 10 to 47. Varying the moisture content around the optimum by ±2% resulted in significant changes in soil strength and deformation in certain cases and had an insignificant effect in some other conditions. This depends on the soil compaction curve. Soils with nearly flat curves around the optimum were less sensitive to moisture variation. A model for rut depth prediction was developed, and its material constants were evaluated. The model was implemented in a rut depth prediction system. The developed model and its guidelines were demonstrated to be of practical significance for the range of the investigated soils.

During the past decade, road construction in Kuwait has advanced, and the road network has increased considerably. The expressway network has been under construction since the mid-1970s; most of the major arterial highways have been improved to urban freeways. Because the roads department became concerned about the maintenance management of the newly developed network, it began to establish an engineering data base of inventories and construction records, which are needed for a successful pavement management system. The research for the data base, which was performed before the Gulf War, established engineering data about the subgrade soils in Kuwait, with an emphasis on locations of the freeways for pavement design and evaluation purposes. The data base was an essential element in the rebuilding of the road network after the Gulf War in 1991.

Four main objectives were set for this project:

1. Identify different types of subgrade soils existing under the major expressways;
2. Establish design values for the subgrade moduli at different conditions of moisture and stress;
3. Establish procedures to estimate the pavement rutting contributed by the subgrade soils;
4. Develop an implementation plan and guidelines for using the research results in the design system currently adopted in Kuwait.

The scope of this paper is limited to the subgrade evaluation with respect to its deformation characteristics as related to soil types and their modulus of resilience.

SURVEY OF RECORDS ON SUBGRADE SOILS IN KUWAIT

There are few documented records on subgrade soils in Kuwait. Some published reports addressed classification and general types. However, no research was found about the deformation characteristics, especially under dynamic traffic loads. One report summarized in a map form the types and distribution of surface soils (*1*). Another report described the general classes of soils according to AASHTO classification system (*2*). Unpublished reports at the Kuwait Institute of Scientific Research (KISR) (*3*) and Research Station of the Ministry of Public Works (*4,5*) have information on the geological and geographical distribution of the surface soils in Kuwait. Data documented in these reports indicate that the surface and near-surface soils are generally granular, ranging from gravelly sand to silty sand. They are mostly calcareous sandy soils, which are known locally as "Gatch."

Gatch soils have sufficient fines content to give a measure of cohesion when watered and rolled. Fines in the Gatch have low plasticity indices; therefore, the control of moisture content during compaction and rolling is quite critical. In the best situation, moisture varied from 7 to 12 percent, depending on the degree of compaction. However, a small variation in moisture may cause a large variation in soil strength (*3,4*).

The literature indicates that there are a lot of wind-blown uniform sands (*1*) that are difficult to compact. These sands are readily broken by traffic, and once in a loose state, they cannot be recompacted.

Another characteristic feature of Kuwait's arid region is the presence of salts in the ground close to the surface, especially in locations where the groundwater level is high, as it is near the Gulf shore. The effect of these salts in the crystalline state may render strong subgrade materials for road construction. However, under high-moisture conditions, the salts would dissolve, causing loss of cohesion and thus a considerable weakening of the subgrade materials.

Soil reports from road and building construction projects indicate that the groundwater table may be as close to the surface as 508 mm (20 in.), and it may be as deep as a few

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meters under the surface for inland locations far from the sea shore. Construction reports of Riyadh and Fahaheel Expressways (Figure 1) indicated that, although the subgrade compaction was acceptable in almost all cases on the basis of measured field density and moisture content at the time of testing, it was observed that the moisture content (MC) changed rapidly, deviating from the optimum by as far as 50 percent of the optimum moisture content (OMC). For example, reported data on an OMC of 8.4 percent at the surface have shown an MC as low as 4.3 percent 2 days after the subgrade has been compacted. However, at deeper depths, MC did not vary much. A deviation of about ±2 percent from the optimum was found in most of the surveyed records. The effect of temperature on moisture variation was not evaluated because there were no temperature records available for MC filed measurements. A moisture variation of ±2 percent was considered in the laboratory program.

EXPERIMENTAL PROGRAM

The methodology adopted was to collect soil samples from several locations to cover most of the construction projects of the expressway network in Kuwait. Physical and mechanical properties of soil samples were then evaluated to determine the engineering properties as well as the chemical composition of these soils. Laboratory-made samples were then tested under triaxial dynamic testing to determine the resilient

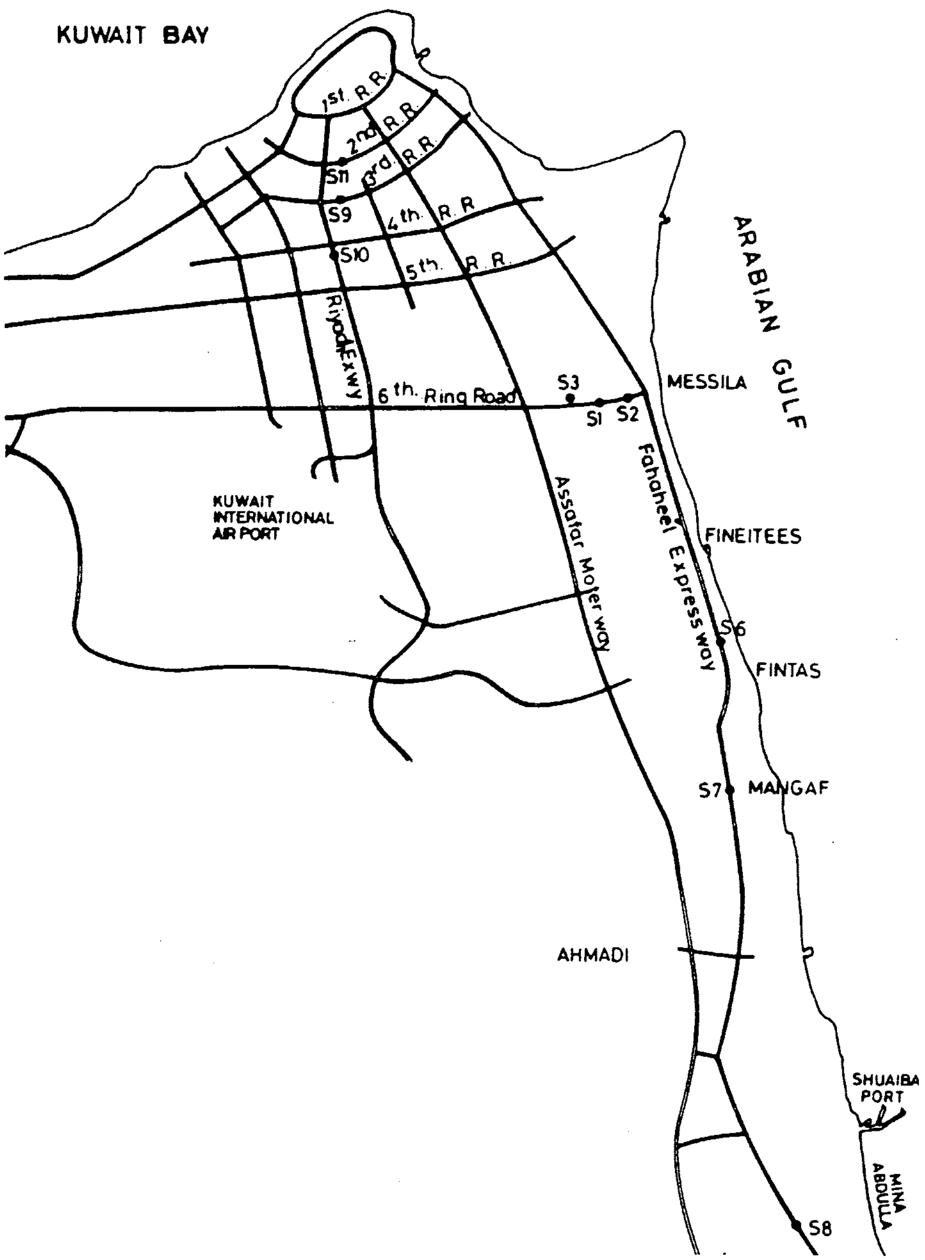

FIGURE 1 Kuwait major road network showing subgrade soil sites.

and permanent deformation (residual) characteristics of these soils at OMC as well as at an MC within ±2 percent of the optimum. The nine sites selected had varied soil conditions and covered almost all geographical locations. The site locations, shown in Figure 1, are denoted by S1, S2, and S3 on the 6th Ring Road; S6, S7, and S8 on the Fahaheel Expressway; and S9, S10, and S11 on the Riyadh Expressway. Representative soil samples were collected from each site for laboratory testing.

Basic tests required for classification (AASHTO, ASTM D 3282 and Unified, ASTM D 2487) and compaction (moisture-density relationship, ASTM D 1557) were performed on all soils. Shear strength was determined by the direct shear test (ASTM D 3080), and the bearing capacity was determined by the California bearing ratio (CBR) test (ASTM D 1883). Dynamic triaxial testing was performed on three of the nine soils to determine the resilient and residual deformation behavior under cyclic repeated loads at different moisture and stress conditions. The test adopted was similar to that of the AASHTO T 274 testing method. The test, however, was modified to develop the relationship between the permanent deformation and the number of loading cycles.

CHARACTERIZATION

Classification

Grain size distribution and Atterberg limits were performed to determine the fine content (percent passing a #200 sieve) and the soil classification. The results are provided in Table 1. The nine sites fit into four groups according to AASHTO (A-1-b, A-3, A-2-4, and A-2-6). According to the Unified system, all soils were in the range of poorly graded to well-graded sand.

Chemical and Mineral Composition

Using x-ray diffraction and chemical analysis, the chemical and mineral composition of the soil samples were determined as shown in Tables 2 and 3. Quartz constituted the main component (the SiO_2 ranged from 71 to 87 percent). Gypsum, calcium salts, magnesium, and sodium sulphate were found in varying proportions, indicating that the soils were contaminated with calcium carbonate and sulfates.

TABLE 1 Physical Characteristics and Classification of Soils at Selected Sites

Soil No.	% Pass. No. 200	Cu	Cc	Liquid Limit	Plastic Limit	Plasticity Index	Classification	
							Unified	AASHTO
S1	5.50	4.67	1.17	23.2	--	NP	SW	A-1-b
S2	4.08	4.62	1.23	17.8	--	NP	SW	A-1-b
S3	3.78	4.31	1.24	22.8	--	NP	SW	A-1-b
S6	1.08	2.75	1.11	20.6	--	NP	SP	A-1-b
S7	1.93	3.00	1.33	22.2	--	NP	SP	A-1-b
S8	1.81	2.89	0.96	23.6	22.5	1.1	SP	A-3
S9	4.29	6.83	1.47	28.1	23.9	4.2	SW	A-2-4
S10	3.50	5.92	1.32	24.0	11.1	12.9	SW	A-2-6
S11	7.50	5.75	0.69	21.3	--	NP	SP	A-3

TABLE 2 Chemical Composition (%) of Soils at Selected Sites

Soil No.	SiO2	Al2O3	Fe2O3	MgO	CaO	SO3	Cl	% Loss in Ignition
S1	81.50	3.65	0.56	1.59	2.17	0.46	0.085	10.05
S2	80.74	6.96	0.64	1.63	3.99	0.23	0.144	5.14
S3	82.60	5.06	0.56	1.62	1.75	0.46	0.089	7.93
S6	87.52	4.92	0.48	1.56	2.40	0.15	0.025	2.67
S7	79.44	4.71	0.72	2.23	5.52	0.42	0.032	6.91
S8	80.38	4.32	0.48	1.56	5.84	0.49	0.021	6.81
S9	74.86	4.71	0.81	2.58	6.14	0.66	0.032	9.13
S10	71.82	5.14	0.88	2.81	6.64	0.66	0.039	7.96
S11	71.02	6.78	1.28	2.97	8.80	1.24	0.067	8.88

TABLE 3 Presence of Minerals in Soils at Selected Sites

Soil No.	Quartz	Clay	Feldspar	Dolomite	Gypsum	Calcite	
S1	Y	Y					
S2	Y	Y	Y			Y	Note:
S3	Y	Y		Y			Y: YES it is present
S6	Y	Y	Y		Y	Y	
S7	Y					Y	
S8	Y					Y	
S9	Y	Y	Y		Y	Y	
S10	Y	Y		Y			
S11	Y	Y	Y		Y	Y	

Compaction

Moisture-density relationships were developed for all soils using the modified AASHTO procedures, AASHTO T 180 (ASTM D 1557). Fresh soil samples were used at each moisture content to eliminate degradation of weak minerals. OMC and the maximum dry density for all soils are listed in Table 4. The OMC ranged from 7.5 to 9.2 percent. The maximum dry density was in the order of about 2,083 kg/m^3 (130 pcf).

Bearing Ratio

The soaked CBR test (ASTM D 1883) conducted at 95 percent compaction indicated a CBR range of 10 to about 48 percent, whereas the maximum swell was about 0.22 mm (0.0087 in.) (Table 5). Comparing these values with the ones specified in the general specifications of road construction in Kuwait, the soils tested satisfied the minimum CBR value of 15 and the maximum swell of 2 mm (0.08 in.), except Soil S11, which had a lower CBR value.

TABLE 4 Compaction Test Results

Soil No.	Optimum Moist. Cont. (OMC), %	Max. Dry Density, kg/meters cubed
S1	8.8	2070
S2	8.0	2150
S3	8.5	2080
S6	7.5	2050
S7	9.2	2050
S8	8.9	2060
S9	7.5	2100
S10	7.5	2140
S11	7.5	2140

1 kg/meters cubed = 0.0642 pcf

TABLE 5 CBR and Swell Test Results

Soil No.	Swell, mm at 65 Blows	CBR at 95 % Compaction
S1	--	47.5
S2	--	22.5
S3	--	21.0
S6	0.005	19.5
S7	0.220	10.0
S8	0.060	27.5
S9	0.150	16.0
S10	0.090	16.5
S11	0.060	10.0

1 mm = 0.0394 in.

Shear Strength

Direct shear testing was conducted on the nine soils (12 samples each). The results show that, although the soils were classified as sands, they exhibited cohesive intercepts. Shear strength parameters, cohesion (c) and angle of internal friction (ϕ), were determined using linear regression. Results of c and ϕ values are presented in Table 6. In general, c values confirm the trend of decreasing strength with increased moisture content. However, the angle of internal friction ϕ did not change significantly. A study of the dilation of all tested samples revealed that the samples were in a very dense state.

DEFORMATION CHARACTERISTICS

Two types of deformation are generally generated under the repetition of traffic loads, elastic or resilient and nonelastic (residual) or permanent deformation. The latter is often called rutting.

To evaluate the resilient and residual characteristics, three of the nine soils were selected on the basis that they differ in their plasticity and classification. The basic tests performed revealed that Soils S1, S2, S3, S6, and S11 have similar characteristics. Soils S9 and S10 are similar, and Soil S8 was distinct among the others. Therefore, Soils S2, S8, and S10 were selected for permanent deformation evaluation. Soil S2 was a nonplastic well-graded sand, S8 was a nonplastic poorly graded sand, and S10 was a well-graded sand with a plasticity index of about 13 (Table 1).

Cylindrical samples that were 101.6 mm (4 in.) in diameter and about 177.8 mm (7 in.) were tested for rutting using a cyclic haversine stress function in a dynamic triaxial test. The stress function has a frequency of 2 pulses per second with a loading period of ⅛ sec and a rest period of ⅜ sec. Four levels of deviatoric stress were designated in the range of 10 to 40 percent of the ultimate uniaxial compressive strength. For each soil, three groups of samples (at OMC, OMC − 2 percent, and OMC + 2 percent) were tested under the aforementioned conditions.

A rutting model, as given by the following equation, was used to study the behavior of the investigated soils (*6,7*).

$$\varepsilon_p = A(N)^b \tag{1}$$

where

ε_p = permanent strain (mm/mm or in./in.),
N = number of load cycles, and
A and b = rutting parameters.

A literature review on the rutting behavior of subgrade soils indicated that three main parameters control the rutting behavior of a certain soil: the soil type and its particle structure, the applied stress, and the density-moisture condition of the compacted soil (*8–10*). It has been well documented that soil type and condition could be well represented by the modulus of resilience (M_r) of the soil at a specified applied stress (*7,10–12*). Accordingly, this study attempted to relate the variation of the rutting parameters A and b to M_r.

The parameters A and b were determined for each tested sample by fitting the experimental data to Equation 1. The

TABLE 6 Shear Strength Parameters (Direct Shear Test)

Soil No.	Cohesion "C", kPa			Angle of Internal Friction, Degrees		
	OMC-2%	OMC	OMC+2%	OMC-2%	OMC	OMC+2%
S1	346.2	270.2	116.4	33.6	31.1	36.5
S2	79.8	168.3	31.7	35.5	26.6	32.5
S3	227.0	222.1	120.2	30.9	28.6	31.8
S6	85.6	54.8	9.6	32.8	31.7	32.8
S7	250.0	192.3	98.1	29.3	28.4	32.0
S8	278.9	221.2	89.4	25.6	25.3	30.5
S9	557.8	438.5	327.9	21.0	13.4	16.2
S10	413.5	273.1	76.9	37.6	38.7	41.0
S11	253.9	269.3	4.8	40.0	36.3	41.2

1 kPa = 0.145 psi

modulus of resilience for each sample was determined by the following equation

$$M_r = \sigma_d/\varepsilon_e \qquad (2)$$

where

M_r = modulus of resilience (kPa or psi),
σ_d = applied deviatoric stress ($\sigma_1 - \sigma_3$) (kPa or psi), and
ε_e = elastic (resilient) strain (mm/mm or in./in.).

Rutting parameters A and b as well as the modulus of resilience for each tested groups are presented in Table 7.

Variation of Modulus of Resilience

The results of M_r presented in Figure 2 show that variation of the M_r with applied stress is not significant in the range of stresses considered. This means that the soil resiliency has

TABLE 7 Results of Modulus of Resilience (M_r) and Rutting Parameters A and b

Soil No. and Moist. Condition	Statistical Parameter*	Modulus of resilience, MPa				Parameter "A"				Parameter "b"			
		Stress level, kPa				Stress level, kPa				Stress level, kPa			
		44.13	87.56	131.69	175.82	44.13	87.56	131.69	175.82	44.13	87.56	131.69	175.82
S2 OMC	AVG	316.4	351.0	318.7	328.0	5.57E-05	2.26E-03	3.65E-03	7.70E-03	3.01E-01	1.02E-01	5.20E-02	3.53E-02
	Std. Dev.	29.8	10.0	39.2	69.3	3.60E-05	1.30E-03	7.79E-04	1.78E-03	8.47E-02	8.32E-02	1.58E-02	8.08E-03
S2 OMC + 2%	AVG	239.5	206.8	265.9	232.6	7.63E-05	2.05E-03	6.52E-03	1.43E-02	2.29E-01	2.13E-01	1.32E-01	1.16E-01
	Std. Dev.	98.0	67.5	80.5	45.9	0.00E+00	1.17E-11	4.43E-11	9.84E-11	0.00E+00	1.33E-09	5.88E-10	0.00E+00
S2 OMC - 2%	AVG	607.5	358.3	358.6	314.8	1.28E-05	4.68E-04	1.58E-03	2.86E-03	3.33E-01	1.89E-01	9.25E-02	6.93E-02
	Std. Dev.	270.0	7.3	66.0	36.2	2.92E-06	2.18E-04	5.85E-04	1.27E-03	1.07E-01	4.00E-03	1.16E-02	7.65E-03
S8 OMC	AVG	172.7	189.9	227.0	263.7	5.95E-04	2.02E-03	3.88E-03	6.09E-03	2.18E-01	1.08E-01	7.04E-02	5.21E-02
	Std. Dev.	61.4	38.0	25.6	21.0	4.39E-04	9.42E-04	1.58E-03	1.99E-03	1.56E-01	1.86E-02	1.40E-02	9.95E-03
S8 OMC + 2%	AVG	138.9	199.3	192.0	212.5	3.08E-03	5.21E-03	9.87E-03	1.72E-02	7.43E-02	6.78E-02	7.79E-02	4.40E-02
	Std. Dev.	31.0	61.1	28.8	15.2	9.03E-04	8.30E-04	3.18E-03	3.63E-03	1.96E-02	1.75E-02	2.27E-02	1.06E-02
S8 OMC - 2%	AVG	280.9	227.3	244.5	265.2	1.51E-04	8.16E-04	1.85E-03	2.93E-03	2.87E-01	1.38E-01	7.11E-02	6.44E-02
	Std. Dev.	43.8	39.4	13.3	22.1	1.82E-04	4.74E-04	8.57E-04	1.29E-03	1.83E-01	3.69E-02	2.26E-02	2.11E-02
S10 OMC	AVG	194.0	215.1	215.9	256.6	4.08E-04	1.32E-03	3.06E-03	4.86E-03	1.49E-01	1.34E-01	7.06E-02	3.16E-02
	Std. Dev.	26.1	25.3	36.2	69.7	1.27E-04	6.49E-04	1.71E-03	2.65E-03	2.91E-02	2.40E-02	2.49E-02	1.89E-02
S10 OMC + 2%	AVG	174.0	193.0	225.4	207.8	2.27E-03	5.19E-03	9.95E-03	1.93E-02	1.20E-01	1.19E-01	7.71E-02	3.26E-02
	Std. Dev.	47.1	53.7	53.6	37.1	1.52E-03	3.09E-03	3.95E-03	5.45E-03	4.68E-02	5.77E-02	1.87E-02	1.72E-03
S10 OMC - 2%	AVG	256.0	302.8	312.9	294.6	1.21E-04	3.37E-04	9.12E-04	1.30E-03	1.36E-01	1.26E-01	8.10E-02	6.20E-02
	Std. Dev.	59.1	34.3	35.8	41.1	4.14E-05	8.41E-05	1.98E-04	4.40E-04	5.10E-02	1.39E-02	1.82E-02	1.90E-02

* Based on three samples (1 kPa = 0.145 psi, 1 MPa = 145 psi)

slight stress dependency. For granular soils, the modulus of resilience would be stress-dependent (11,12). A suggested formula for M_r less the stress relationship is

$$M_r = k1(\theta)^{k2} \quad (3)$$

where

M_r = modulus of resilience (kPa or psi),
θ = bulk stress = $(\sigma_1 + \sigma_2 + \sigma_3)$ (kPa or psi), and
$k1$ and $k2$ = stress sensitivity factors.

However, data obtained for the investigated soils did not verify this stress dependency formula within the considered range of stresses. The moisture content, on the other hand, has shown a more pronounced effect of the modulus of resilience for the tested soils. The effect was different from one soil to another. For instance, for Soil S2, a well-graded sand for which OMC was 8 percent, the M_r values at the optimum and at OMC − 2 percent were close, while M_r values at OMC + 2 percent dropped significantly. This is unlike Soil S10, for which M_r values at the optimum and at +2 percent from the optimum were close. This difference may be attributed to (a) the slight OMC variance within samples and (b) whether the determined OMC was closer to the dry side or the wet side of soil compaction curve. The results suggest that moisture variation of ±2 percent from the optimum may not have a significant effect on the resilient moduli of sandy subgrade soils. However, more variation may lead to considerable changes in the soil modulus. For the tested soils, S8 and S10 showed close M_r values, whereas S2 showed slightly higher values at the different stress levels.

Variation of Rutting Parameters

Variation of rutting in subgrade may be detected through the variation of the rutting parameters A and b as previously discussed. For cohesive fine grained soils, it was found that the slope of the rutting curve b is quite independent of the testing conditions and may be a characteristic of the soil type itself. Parameter "A" is dependent on the testing conditions and the soil type as well (7–10). Data developed in this research were analyzed to determine the variation of these parameters for the sandy subgrades in Kuwait.

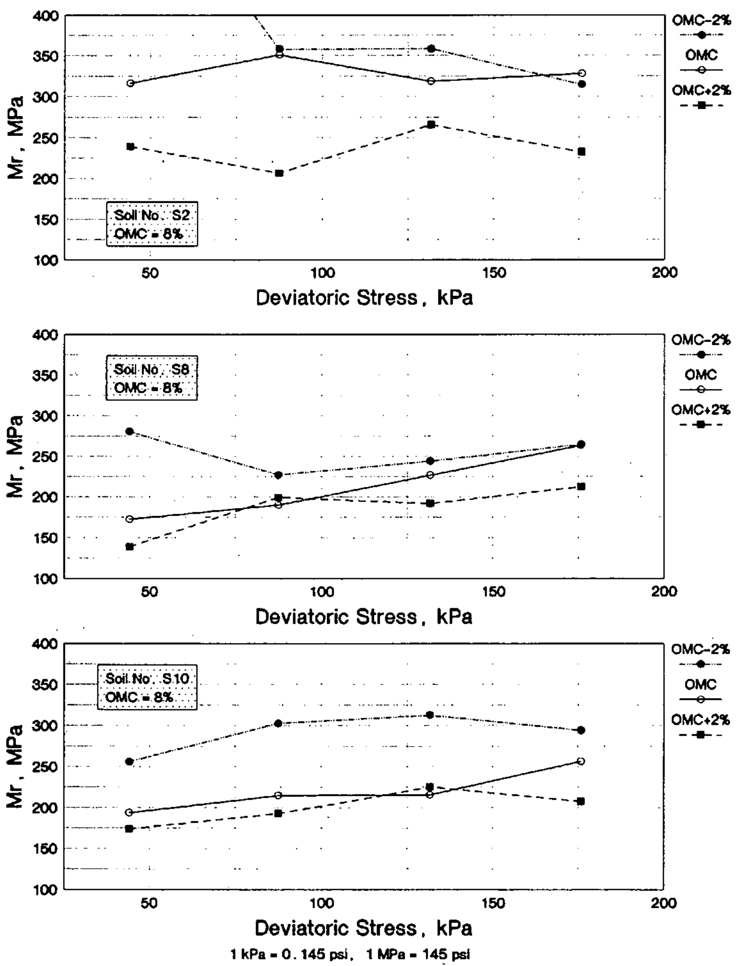

FIGURE 2 Relationship between M_r and deviatoric stress.

Parameter A

Parameter A is the intercept of the regression line $\log_e \varepsilon_p$ versus $\log_e N$ as given in Equation 1. The calculated values of A at different applied stresses are presented in Figure 3. The results show that A increases with the increase of stress and depends on the soil type and moisture. By comparing M_r results in Figure 2 and A results in Figure 3, it can be concluded that effects of moisture are similar in both cases. This suggests that parameter A could be related to the modulus of resilience at a given stress.

Based on previous research on cohesive soils (*7–10*), a relationship was suggested to relate Parameter A to the soil condition as presented by M_r and the deviatoric stress. The relationship can be presented in the form

$$A = R(M_r)^s \; x \; \exp(c\sigma_d) \qquad (4)$$

where R, s, and c are material constants to be determined experimentally. These constants are independent of the stress level and the compaction conditions because the M_r value represents these conditions for a given soil.

Using regression analysis to fit the model in Equation 4, the parameters R, s, and c were determined using the Statistical Analysis System (SAS) (*13*). The results are given in Table 8. The parameter A model given in Equation 4 fits very well. The r^2 (square of the coefficient of correlation) and the F statistical test (*14*) indicate that the suggested model is statistically significant to correlate Parameter A with the modulus of resilience (M_r) and the applied deviatoric stress (σ_d). Figure 4 shows the observed A values, which are experimentally determined, versus the predicted A values using Equation 4. Figure 4 shows that A could be significantly predicted by the model in Equation 4.

Parameter b

Parameter b is the slope of the regression line $\log_e \varepsilon_p$ versus $\log_e N$ as given in Equation 1. Values of Parameter b are given in Table 7 for the tested soil groups. Unlike Parameter A, Parameter b was found to be independent of the stress applied or the moisture condition. It is a material property that would be unique for a soil type. Results of b values plotted versus M_r for each soil in Figure 5 indicate that b is unrelated to M_r. The best expected value, statistically, would be the grand mean b_{mi} of all tested samples for each soil. For instance, b_{m2} is the mean b value for Soil S2 for all various stress and

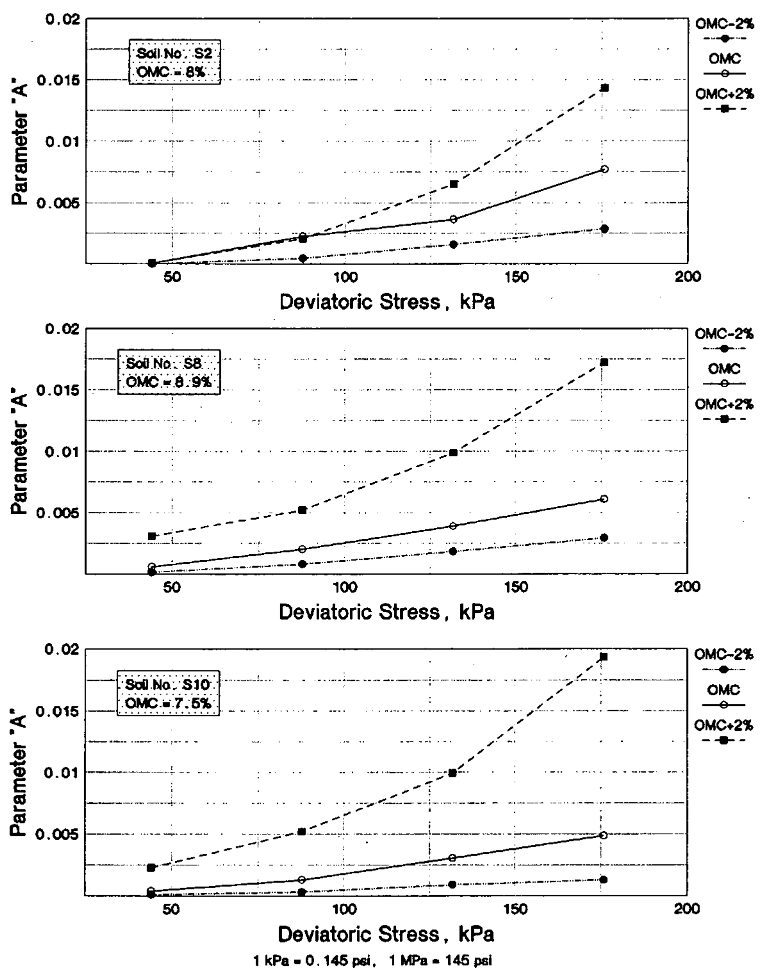

FIGURE 3 Relationship between Parameter A and deviatoric stress.

TABLE 8 Results of Regression Analysis of Parameter A Model

Soil No.	Regression Constants				R-Square, r^2	F Statistic	Prob. > F
	Log_e R	R	s	c			
S2	17.858	5.7×10^7	-2.659	0.222	0.931615	54.49	0.0001
S8	2.725	15.256	-1.021	0.201	0.950057	86.60	0.0001
S10	51.198	1.72×10^{22}	-5.797	0.183	0.863242	28.40	0.0001

Note: R, s and c constants are defined in equation {4}, Values given are for M_r and σ_d in psi units.

moisture conditions. The grand mean values and their standard deviations are listed in Table 9.

Inspecting the distribution of Parameter b in Figure 5, it was suggested that there probably was no significant difference among the grand means. To test the hypothesis that $b_{m2} = b_{m8} = b_{m10}$, an analysis of variance was performed to determine the F^* statistic for the b distribution. Results of the F-test are presented in Table 9. Because the F^* value is less than the critical F at the level of significance $\alpha = 0.05$, the null hypothesis that the b means are equal cannot be rejected. Therefore, it was concluded that the sandy soils investigated would not exhibit variation in their Parameter b. The data show an average b value of about 0.113 may be appropriate for a rutting prediction using the rutting model given in Equation 1.

ESTIMATION OF RUT DEPTH IN SUBGRADE LAYER

Methodology

The rut depth R_d contributed by a subgrade layer in a pavement system can be determined by means of Equations 1 and

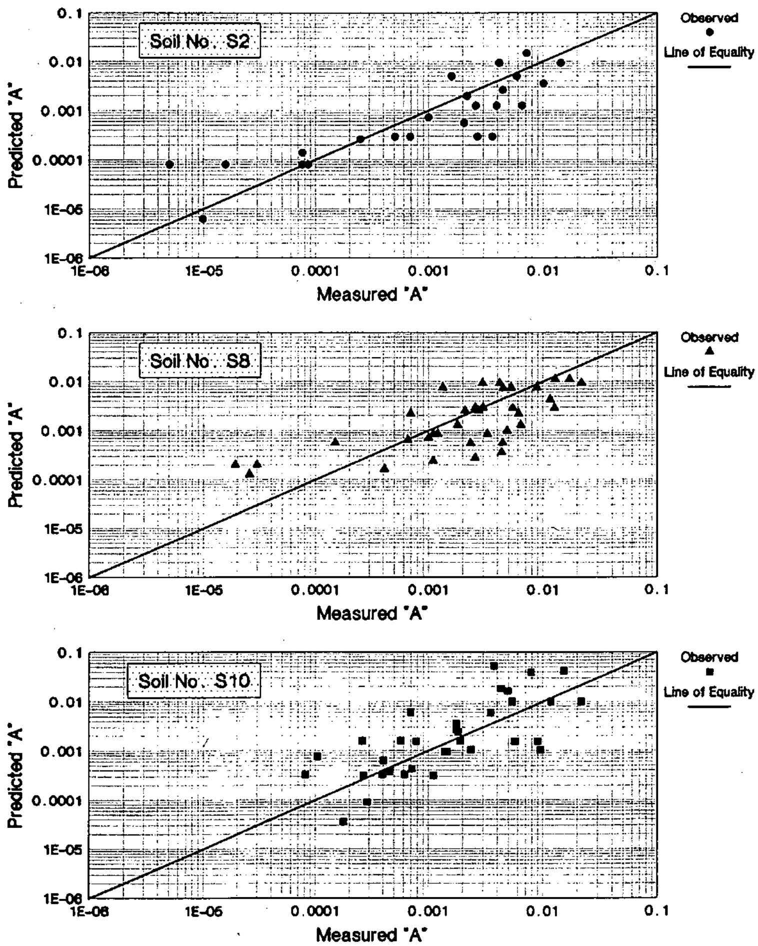

FIGURE 4 Predicted versus observed (measured) Parameter A.

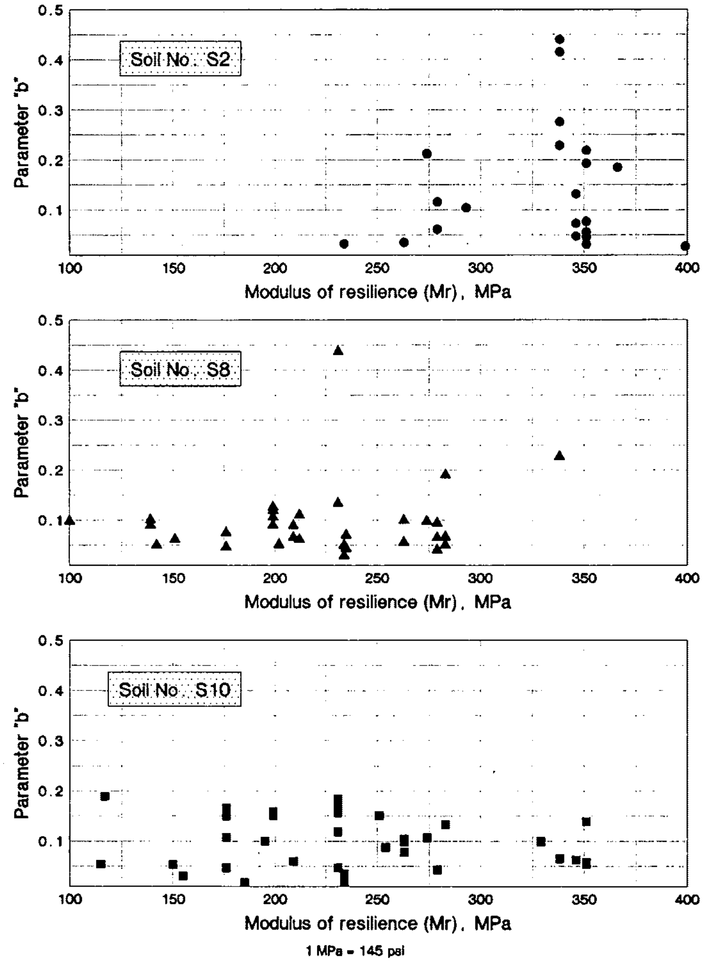

FIGURE 5 Variation of Parameter b with modulus of resilience (M_r).

TABLE 9 Mean Values of Parameter b

Statistical Parameter	Soil No.		
	S2	S8	S10
No. of samples	24	36	35
Mean Value of Parameter "b_{mi}"	0.147041	0.106094	0.095345
Standard Deviation	0.113881	0.101049	0.050721
Minimum Value	0.027100	0.029000	0.0175
Maximum Value	0.440000	0.535000	0.189000
Standard Error of Mean	0.022836	0.017075	0.008527

F^* - test for the null hypothesis,

Ho: Mean values of b_{mi} are equal

Results: Calculated $F^* = 0.212640$,
Critical $F(0.05, 2,92) = 3.11$

Decision: Do not reject the null hypothesis that b_{mi} are equal.

4, where the permanent strain in a thin subgrade Layer i with a thickness Δh_i can be determined by

$$\varepsilon_{pi} = R(M_{ri})^s \cdot \exp(c\sigma_{di}) \cdot N^b \tag{5}$$

Then the total rut depth of all subgrade layers would be equal to

$$R_d = \sum_{i=1}^{n} \varepsilon_{pi} \cdot \Delta h_i \tag{6}$$

where

R, s, c, and b = constants of the subgrade soil;
M_{ri} = modulus of resilience of Layer i;
σ_{di} = deviatoric stress at the mid-depth of Layer i;
Δh_i = thickness of Layer i; and
n = number of subgrade layers.

Using a multilayer elastic analysis computer program, such as CHEVRON or ELSYM5, the stress state in the pavement system can be determined. Hence, the rut depth (R_d) can be determined as follows:

1. Determine the soil constants—R, s, c, and b—by performing a triaxial repeated load test as performed in this research. The given values in this study may be suggested for sandy soils similar to those investigated.
2. Determine an effective value of the modulus of resilience to represent the soil conditions around the year. The AASHTO 1986 design guide (*15*) or the field estimated modulus of resilience using backcalculation techniques from falling weight deflection tests may be used to determine this value.
3. Use multilayer elastic analysis to determine the stress state of the pavement system at the midpoint of subgrade layers. Then determine the deviatoric stress using $\sigma_d = \sigma_1 - \sigma_3$.
4. Estimate the design life of the pavement in N cycles. For instance, N may be considered the design value of $ESAL_{18}$ (equivalent 18-kips single axle load).
5. The rut depth (R_d) may be determined by using Equations 5 and 6 if the deviatoric stress and thickness at each layer are known.
6. Check the value of rut depth (R_d) determined in Step 5 against a preselected critical value of rut depth that may be allowed in the subgrade.

It is important to note that the critical value of rut depth has to be field calibrated and correlated to values obtained in the lab. This investigation does not suggest certain values because no field data were available. However, on the basis of measurements made in the AASHTO road test, rut depth in the subgrade could be assumed to be between 0 to 20 percent of the total rut depth in the pavement system. If a total rut depth of 25.4 mm (1 in.) is allowed in the pavement, then a maximum value of 5.1 mm (0.2 in.) may be allowed in the subgrade. The determination of R_d as presented here is entirely laboratory based and may not be assumed to equal the field value. However, due to the confinement of the pavement system in the field and the protection of the subgrade by the pavement layers, it is expected that the values obtained by this laboratory-based method may be higher than the actual field value.

Example for Rut Depth Calculation

For a flexible pavement composed of 140 mm (5.5 in.) asphalt concrete surface course with M_r = 2,068,400 kPa (300,000 psi), constructed on a sandy subgrade soil with M_r = 310,260 kPa (45,000 psi), the rut depth contributed by the top 381 mm (15 in.) of the subgrade after 1 million 18-kips ESAL load applications was estimated. The constants R, s, c, and b were assumed to equal those of Soil S2 in this paper.

The CHEVRON program for multilayer elastic analysis was used to determine the stress state at the mid-depth of three 127-mm (5-in.) layers in the subgrade. Using Equations 5 and 6, the total rut depth in the top 381 mm (15 in.) of the subgrade was estimated to be 2.1 mm (0.083 in.). When the analysis was repeated using 103,420 kPa (15,000 psi) subgrade modulus of resilience (high moisture content) instead of 310,260 kPa (45,000 psi), the rut depth increased to 8.8 mm (0.347 in.).

CONCLUSIONS

On the basis of the results of this study, the following conclusions are drawn:

1. Subgrade soils in Kuwait are mostly granular materials composed mostly of sand with little silt and clay fines. Soils tested from various locations were classified as A-1-b, A-2-4, A-2-6, and A-3 group soils, according to the AASHTO Classification System, with A-1-b being the most predominant group type.
2. Chemical and mineralogical analysis indicated that quartz is the principal component. Gypsum, calcium, magnesium, and sodium sulfates are found in varying proportions.
3. The strength and bearing ratio indicated good quality support conditions of these soils at the optimum moisture content. Varying the moisture content around the optimum by ±2 percent resulted in significant changes in soil strength and deformation in certain cases and insignificant effects in other conditions. This depended on the soil optimum moisture content, whether it was close to the dry side or the wet side of the compaction curve.
4. The rutting of subgrade soils could be evaluated using the models given by Equations 5 and 6. Parameters included in these equations have been evaluated. Analysis of these parameters revealed that the Rutting Parameter A depended on the soil modulus of resilience and the existing stress, whereas Parameter b was constant for the material and independent of the stress and the moisture conditions.
5. The developed rutting model was based on the laboratory findings. The predicted rut depth value may differ significantly from the actual field value. However, it is anticipated that the model predicted value may be more conservative (higher) than the actual value expected in the field.

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