Clegg Hammer—California-Bearing Ratio Correlations

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Abstract: This investigation was conducted to assess the efficacy of the Clegg impact hammer (CIH) for estimating the strength of compacted soils by conducting a comparative study between the California-bearing ratio (CBR) and CIH tests. The study was carried out in two phases. In phase 1, compacted marl samples were prepared in the laboratory under three different compactive efforts and different molding moisture contents and then subjected to CBR and CIH tests. Phase 2 focused on conducting in situ CBR and CIH tests on existing soils at some preselected locations as part of ongoing projects in Saudi Arabia. The test results of both phases were statistically analyzed and indicated that the Clegg impact value correlates relatively well with the CBR value for both the laboratory and field tests. These correlations were compared with those reported in the literature. A general, reliable, best-fit model has been proposed for the laboratory, field, and literature data. © *2002 The American Physical Society.*

CE Database keywords: California bearing ratio; Strength; Compaction; Predictions; Moisture content.

Introduction

Unbound earth materials (e.g., soils, gravels, etc.) have an important role in the design and construction of road and airfield pavements or foundations and other earth-fill structures. The assessment of the in situ properties of these materials (i.e., when they exist as bases, subbases or subgrades) in terms of density, strength, etc., is important, as well. However, the evaluation of compacted fills is an expensive and time consuming endeavor and, therefore, the testing of these materials is generally quite limited. In addition, the high variability encountered with most natural soil types and the number of soil types typically existing in a project necessitate the presence of a test method that is inexpensive and rapid. Consequently, the trade-off has been either a cursory survey with limited results or an in-depth assessment but on a limited number of sites.

The California-bearing ratio (CBR) test is frequently used in the assessment of granular materials in base, subbase and subgrade layers of road and airfield pavements. The CBR test was originally developed by the California State Highway Department and was thereafter incorporated by the Army Corps of Engineers for the design of flexible pavements. It has become so globally popular that it is incorporated in many international standards (ASTM 2000). In Saudi Arabia, the CBR test is considered as one of the most important tests used to assess earth backfills (Al-Abdul Wahhab and Abduljauwad 1989). The significance of the CBR test emerged from the following two facts: (1) for almost all pavement design charts, unbound materials are basically characterized in terms of their CBR values when they are compacted in pavement layers; and (2) the CBR value has been correlated with some fundamental properties of soils, such as plasticity indices, grain-size distribution, bearing capacity, modulus of subgrade reaction, modulus of resilience, shear strength, density, and molding moisture content (Doshi and Guirguis 1983). Because these correlations are currently readily available to the practicing engineers who have gained wide experience with them, the CBR test remains a popular one.

Despite its international popularity, it is known that this test is both tedious and time consuming. It requires a lot of preparation and needs different types of equipment, especially when used in the field (Habib-ur-Rehman 1995). These characteristics have encouraged field and research engineers to look for other simpler techniques that correlate well with the CBR test. An alternative method is the use of the nuclear gauge for the measurement of density and moisture content. This test is, however, both expensive and requires special precautions against its radioactive material (Wray 1986). Moreover, the density of the soil is not always proportional to the soil's strength (Al-Amoudi et al. 1995). Therefore, the density will not always reflect the actual strength of the soil. Other techniques, such as the vane shear apparatus, cone penetrometers, unconfined compression, and Texas triaxial tests (Ladner 1973; Clegg 1983a; Al-Joulani 1987) have been reported in the literature to correlate well with the CBR test but could not replace it due to either some inherent shortcomings of the tests or their limitations to laboratory applications. Therefore, there is an urgent need to develop a simplistic methodology that can be used as an alternative to, and is ultimately capable of replacing, the CBR test. A new device, known as the Clegg impact hammer (CIH), has been developed in Australia (Clegg 1983a,b; Mathur and Coghlans 1987; Habib-ur-Rehman 1995) under the commer-

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Note. Associate Editor: Dallas N. Little. Discussion open until May 1, 2003. Separate discussions must be submitted for individual papers. To extend the closing date by one month, a written request must be filed with the ASCE Managing Editor. The manuscript for this paper was submitted for review and possible publication on December 13, 2000; approved on January 2, 2002. This paper is part of the ***Journal of Materials in Civil Engineering***, Vol. 14, No. 6, December 1, 2002. ©ASCE, ISSN 0899-1561/2002/6-512–523/$8.00+$.50 per page.

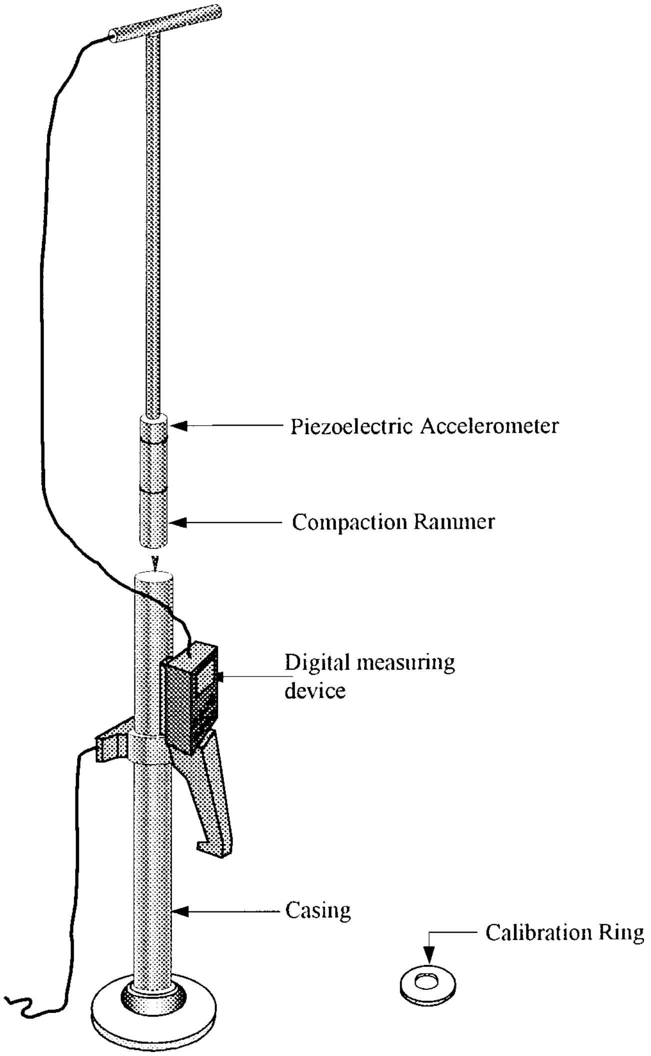

Fig. 1. Schematic representation of the Clegg impact hammer

cial name of "Clegg impact soil tester." This method is claimed as a possible alternative to the CBR test, because it may practically be performed in both the field and laboratory. Further, the hammer tester provides an easy to operate, quick, and portable device as well as a cost-effective means of process control by monitoring the effect of roller passes and checking the variability of field compaction easily. This device measures the "Clegg impact value" (CIV), which is an overall measure of the stiffness of the soil layers. This is achieved by measuring the dynamic rebound of the soil owing to a standard weight falling from a constant height. The apparatus, schematically shown in Fig. 1, typically consists of a 4.55-kg (10-lb) compaction hammer with a shape and a size conforming to the modified Proctor hammer. It is equipped with a piezoelectric accelerometer and connected to a digital reading unit. It is based on the principle of allowing the hammer to drop on a soil surface from a fixed height (45 cm), and the rate at which the hammer rebounds (i.e., soil resistance) is related to the soil strength, density, or stiffness. The built-in electronic meter displays and records the peak rebound. The stiffer the surface, the higher will be the rebound and so will be the CIV. The reading of the device at the fourth or fifth blow is typically recommended as the standard Clegg impact value (Clegg 1983a,b; Mathur and Coghlans 1987), because the CIV values start to stabilize at that blow.

Since the upsurge of oil prices in the early 1970s, Saudi Arabia and the other Arabian Gulf States have witnessed an unprecedented construction boom in terms of industrialization and establishment of the infrastructure. In eastern Saudi Arabia, these projects have included the construction of industrial complexes, major roads, highways and airports, the expansion of the existing petrochemical facilities, and urbanization of almost every locality in the region. The lack of good quality earth materials has resulted in the extensive exploitation of calcareous sediments, locally known as marls, in foundation and in base-course construction (Aiban et al. 1997). The formation of these soils is thought to be the result of the physical and chemical weathering of the parent carbonate rocks with the presence of diagenetic impurities such as organic matter, silt or sand (Akili 1980; Fookes and Higginbottom 1980; Qahwash 1989; Aiban et al. 1998).

The characteristics of marl soils are often obscured by their burial with detrital sediments. Furthermore, the carbonate material in these soils tends to be soluble, chemically reactive, and easily recrystallizable. Moreover, the formation of these materials has been reported to defy any satisfactory geologic, chemical, or pedological definition (Fookes and Higginbottom 1975; Fookes and Higginbottom 1980; Aiban et al. 1998). Accordingly, the behavior of calcareous sediments is complex. Aiban et al. (1997) have recently reported great variations in terms of the classification of 20 eastern Saudi marl soils. Moreover, all 20 marls exhibited acute sensitivity to water (i.e., sharp reduction in strength was observed when these soils were exposed to water or molded at high moisture contents).

There are few reports worldwide on the assessment of aggregate-surfaced pavement layers using the CIH despite the fact that this equipment has been demonstrated to be easy to operate, quick and a portable device that can efficiently replace the CBR test (Clegg 1983a,b; Mathur and Coghlans 1987; Habib-ur-Rehman 1995). Therefore, there is an exigent need to develop as much database as possible on the relationship between CBR and CIH test results using different types of soil. Accordingly, this investigation was initiated with the primary objective to assess the reliability of CIH for estimating the strength of compacted soils by conducting a comparative study between the CBR and CIH test results. To meet this objective, a typical eastern Saudi calcareous soil, known as marl, was first selected for the laboratory tests over a wide range of density, moisture content, and compactive effort. The second phase focused on the performance of in situ CBR and CIH tests on various types of soils at some preselected locations after they had been prepared for ongoing construction projects in some of the major cities in Saudi Arabia (Fig. 2). The data developed from both the laboratory and field tests were thereafter combined to arrive at the best statistical, reliable model which could predict the CBR values from CIH results. Further, this model was then compared with those reported in the literature for other soils to establish the validity and reliability of generalization, if possible, of the CIV-CBR relationship or otherwise.

Experimental Program

A schematic representation of the experimental program conducted in this investigation is shown in Fig. 3. The marl selected for the laboratory work was obtained from the Dhahran vicinity of eastern Saudi Arabia (Fig. 2). It represents one of the predominantly occurring carbonate soils of the landscape (Akili 1980). The soil was retrieved from a depth of approximately 1 m below the ground surface and was initially identified by its color and

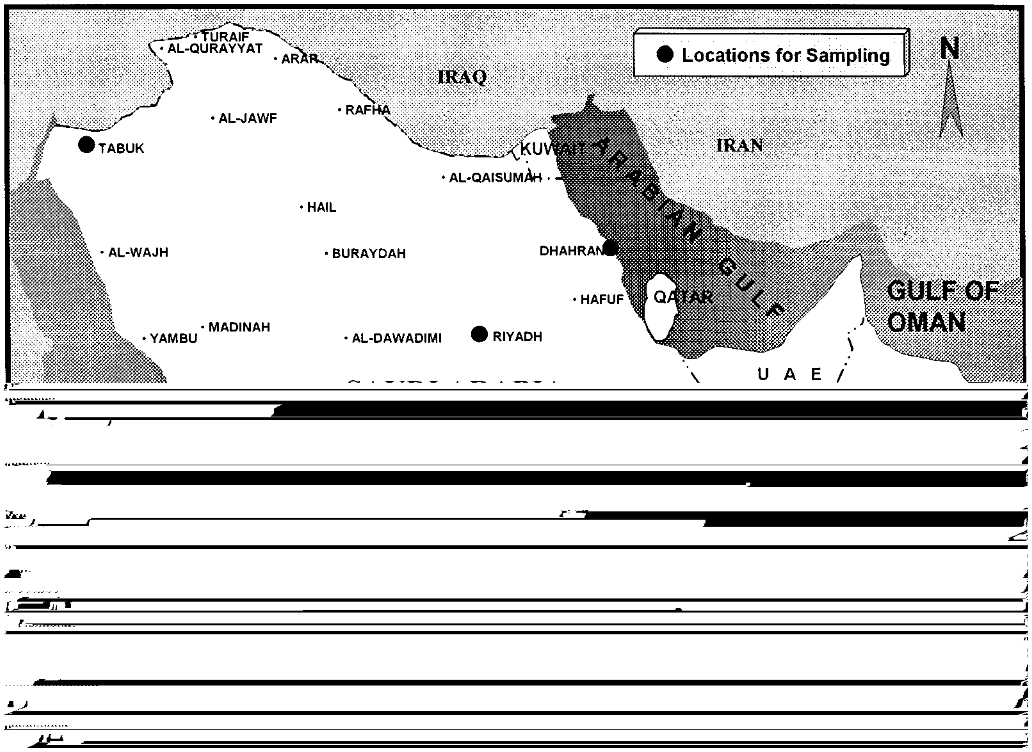

Fig. 2. Vicinity map showing the selected testing locations

texture with a maximum grain size of 12.5 mm (0.5 in.). Grain-size analysis was conducted according to ASTM D 422, using washed sieving with distilled water to get a better gradation analysis. The passing material was collected and dried and a hydrometer test was thereafter conducted.

The compaction test was performed using the standard CBR 15.2×11.7 cm (6.0×4.6 in.) mold at the following three different compactive efforts—the standard AASHTO (ASTM D 698; AASHTO T 99), the modified AASHTO (ASTM D 1557; AASHTO T 180), and one in between, as outlined in Table 1. The CBR test was also conducted at the same three compactive levels using the same molds as the compaction test, in accordance with

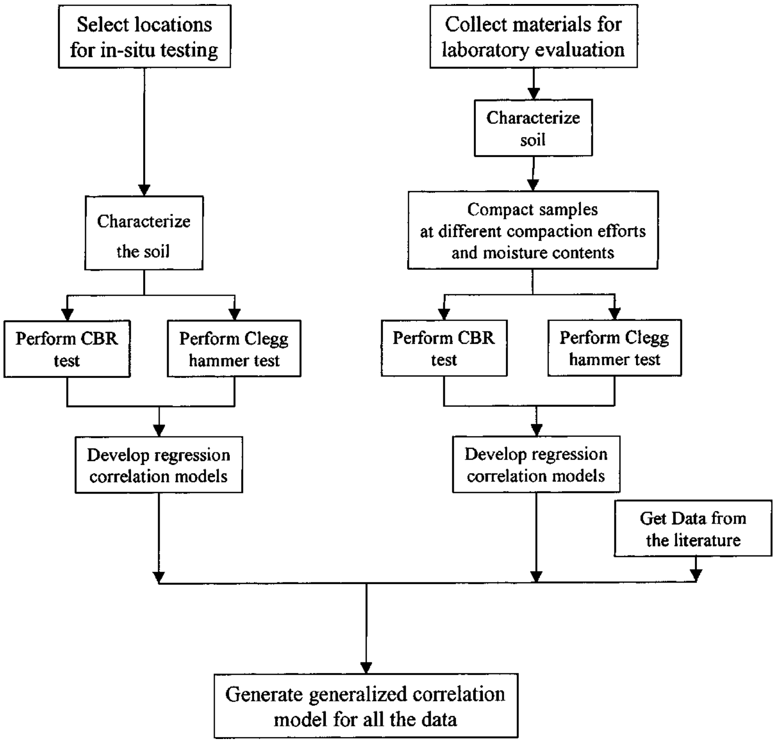

Fig. 3. Schematic representation delineating the experimental program

Table 1. Technical Specifications for the Compactive Efforts

Specification	Number of layers	Number of blows per layer	Weight of hammer (kg)	Drop height (cm)	Compactive effort (kJ/m^3)
Standard AASHTO	3	56	2.50	30.5	591
—	5	26	4.54	45.7	1,245
Modifed AASHTO	5	56	4.54	45.7	2,682

ASTM D 1883. The marl specimens were prepared at different moisture contents that ranged from well below the optimum to two or three points on the wet side of the optimum in an attempt to investigate the role of molding water content on the CBR values and, consequently, on the shear strength capability of the soil. This is because the maximum strength of a soil does not always occur at the maximum dry density (Al-Amoudi et al. 1995).

For each selected moisture content and compactive effort, three CBR specimens were prepared by compacting the wetted soil in five layers to achieve a dry density equivalent to that of the compaction test at the selected compactive effort. At each moisture content, three specimens were immediately loaded under a surcharge of 4.5 kg (9.9 lb) and subjected directly to the CBR penetration test. In the case of soaked condition, three additional CBR specimens were deferred until they had been soaked in water for 4 days under the same surcharge of 4.5 kg (9.9 lb). The CBR test was conducted at a loading rate of 1.27 mm/min (0.05 in./min), and the load/penetration data were recorded using a portable data logger. To determine the CBR value from the load-penetration curves, the loads at penetrations of 2.50 mm (0.10 in.) and 5.00 mm (0.20 in.) were determined. Because the CBR is defined as the ratio of the force required to penetrate a circular piston of 1,935 mm^2 (3 $in.^2$) cross section into soil in the CBR mold at a rate of about 1 mm/min (0.04 in./min), to that required for similar penetration into a standard sample of compacted crushed rock—13.24 and 19.96 kN (3,000 and 4,500 lb) at penetrations of 2.50 and 5.00 mm (0.10 and 0.20 in.), respectively—this ratio was determined at these two penetrations as follows:

$$\mathrm{CBR}=\frac{\text{measured force}}{\text{standard force}}*100$$

The higher of these two values is reported as the CBR value for that specimen.

After performing the CBR test, each specimen was turned upside down so that the undamaged surface of the specimen could be tested by the CIH as recommended by Clegg (1983a,b). A spacer was then placed beneath the sample between the base and the surface of the specimen to provide support to the specimen when performing the CIH test. The CIH was then placed at the top surface of the specimen and the hammer was raised to the 45 cm (17.7 in.) height required for testing and released to freely fall on the sample. The rebound (of the hammer) called the CIH was recorded. The test was repeated several times to arrive at the CIV stabilized reading.

For the in situ program, the field tests were conducted on a pavement network system located in some of the major cities in Saudi Arabia (Fig. 2) with subbase and base layers being of various types of soils. The in situ CBR tests were performed after digging 1 (1 m) test pit and then drilling a 152-mm (6-in.) diameter hole by an auger for a depth of 305 mm (12 in.) at the preselected locations. When conducting the field CBR tests, the

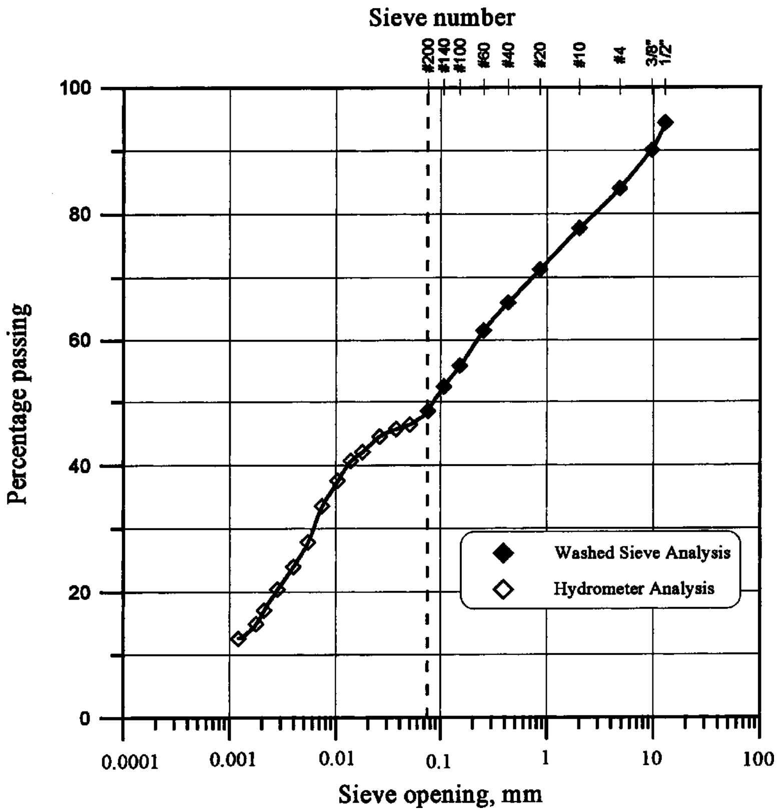

Fig. 4. Grain-size distribution of the marl soil

following steps were followed: the loading device was fixed in a channel beneath the truck, which was equipped with a hydraulic jack. The truck was brought above the test pit and jacked up so that there was no load on the rear axle. The required surcharge weight was thereafter placed in the center of the test area, and the test was conducted in accordance with ASTM D 1883. In an adjacent location at the same test pit, the CIH test was also conducted and the readings were recorded at different blows till the CIV values were stabilized.

Results and Discussion

The grain-size distribution curve shown in Fig. 4 indicates that the marl used in this investigation is a well-graded soil. It has about 66% passing sieve No. 40 [0.42 mm (0.017 in.)] and 49% passing ASTM sieve # 200 [75 μm (3 mils)]. The liquid limit,

Table 2. Index Properties of the Marl Soil Used in the Laboratory Investigation

Properties	Value
Liquid limit (%)	31.3
Plastic limit (%)	22.4
Plasticity index (%)	8.9
Percent Passing sieve No. 40	66
Percent Passing sieve No. 200	49
Uniformity coefficient	192
Curvature coefficient	0.3

Table 3. Summary of the Laboratory CBR-CIV Results

Compactive effort (kJ/m^3)	Test condition	Moisture content (%)	CBR[a]	CIV[a,b]
2,682	Soaked	8.9	4.0	9.0
		11.2	47	18
		13.2	37	28
		15.8	8.5	11
2,682	Dry	6.1	64	35
		7.8	92	49
		10.1	103	46
		12.0	95	42
		17.0	25	34
1,245	Dry	8.5	25	29
		10.7	46	40
		12.0	53	43
		15.3	37	20
		16.0	9.5	13
591	Dry	8.5	19	28
		10.4	23	29
		12.0	25	27
		13.0	21	24
		15.1	8.5	12

[a] Average of three values.

[b] Based on the fifth CIH drop reading.

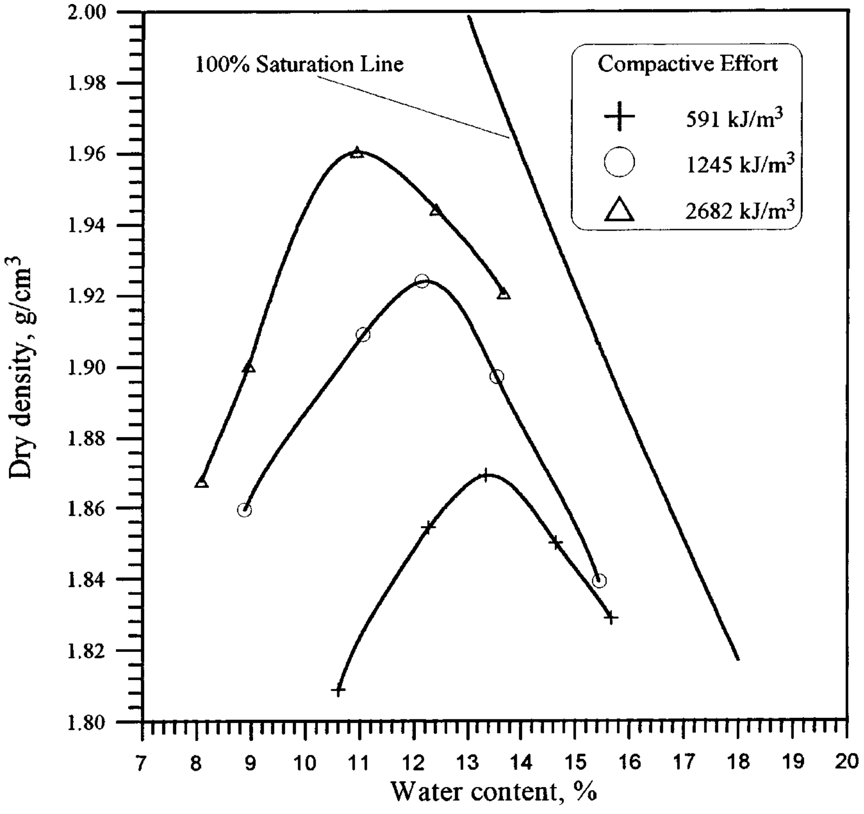

Fig. 5. Compaction test results

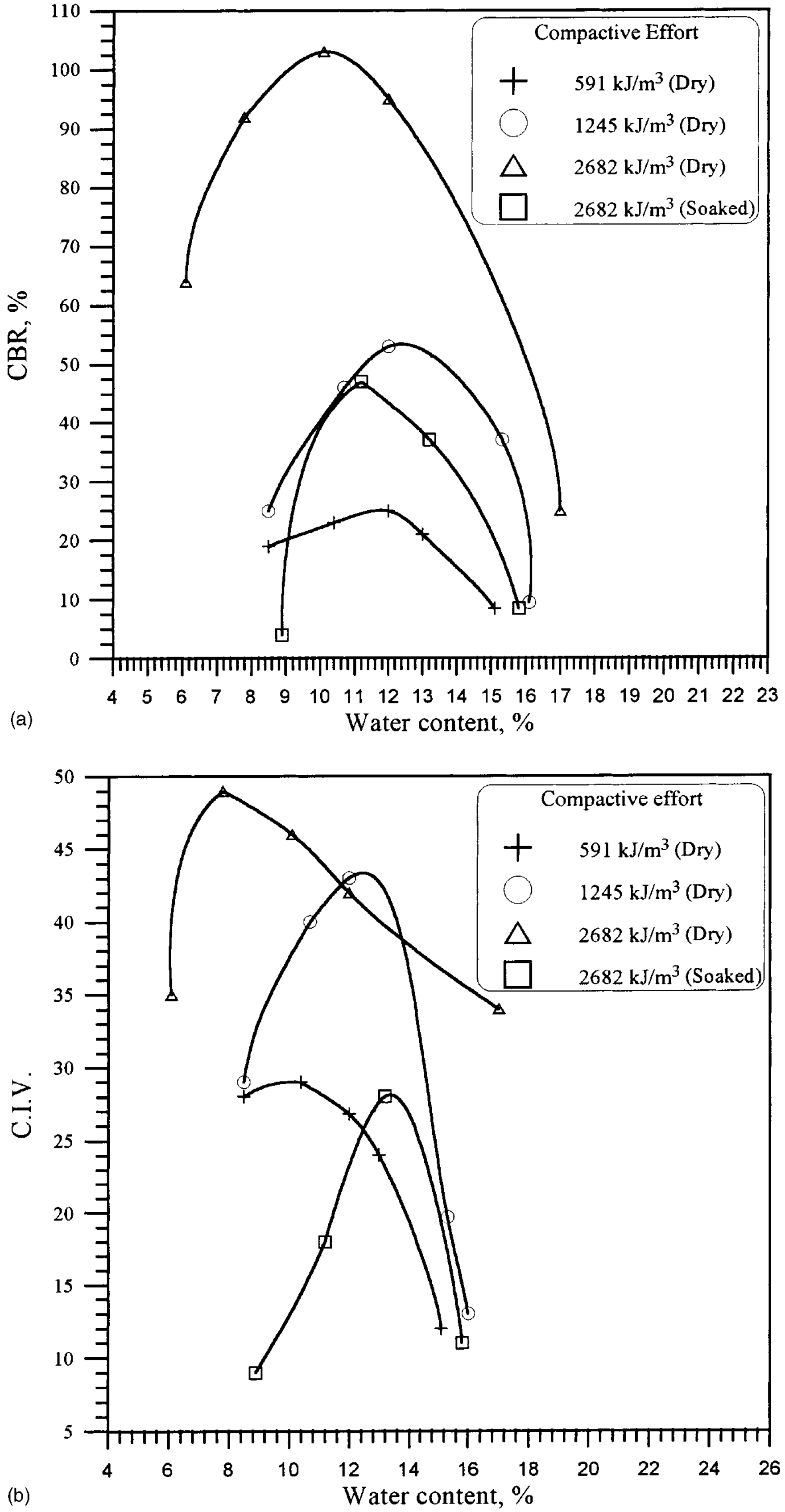

Fig. 6. CBR and CIV test results for the various compactive efforts: (a) CBR test results; (b) CIV test results

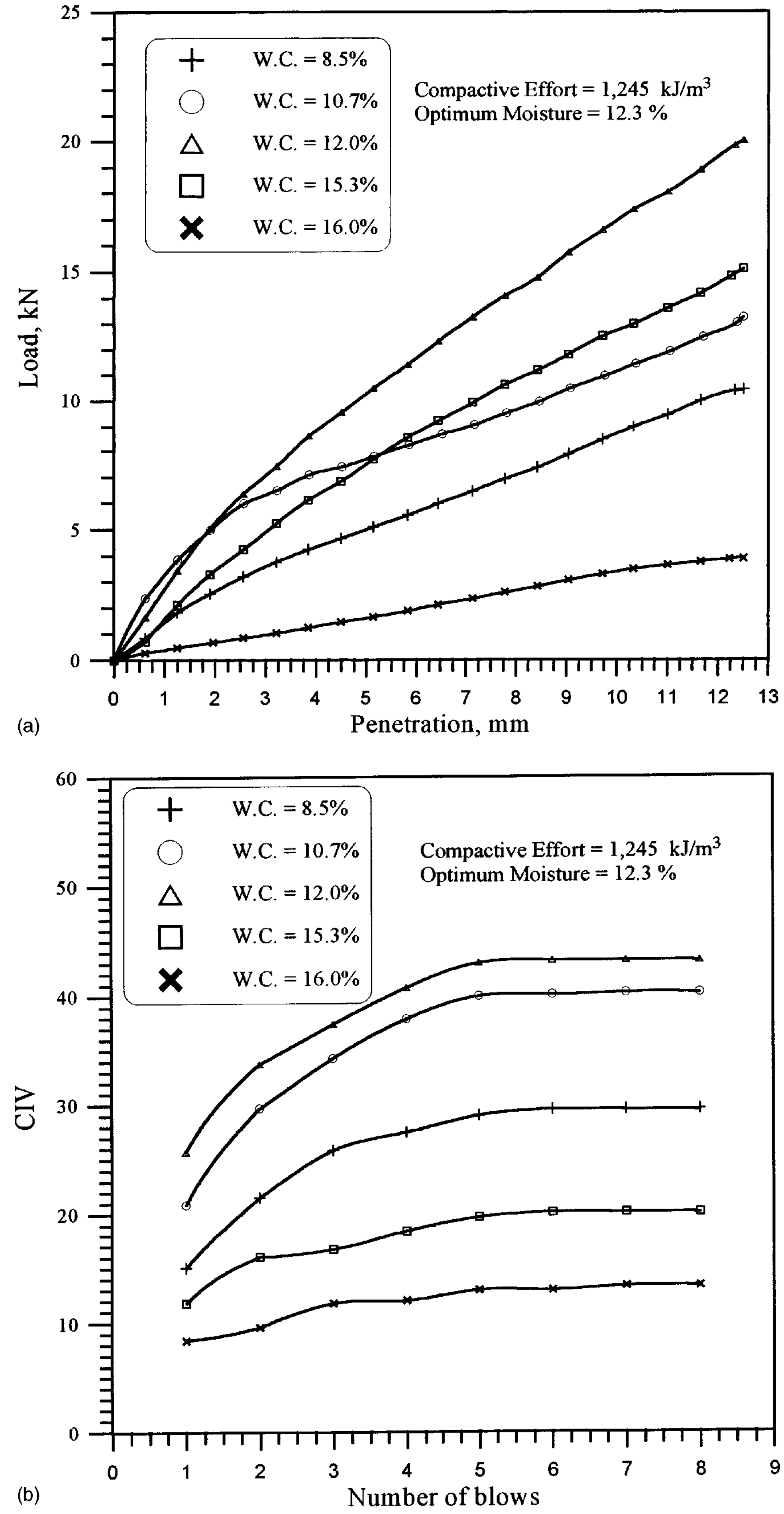

Fig. 7. Typical raw data of the laboratory test results: (a) load-penetration (CBR) curves; (b) CIV-number of blows (CIH) tests

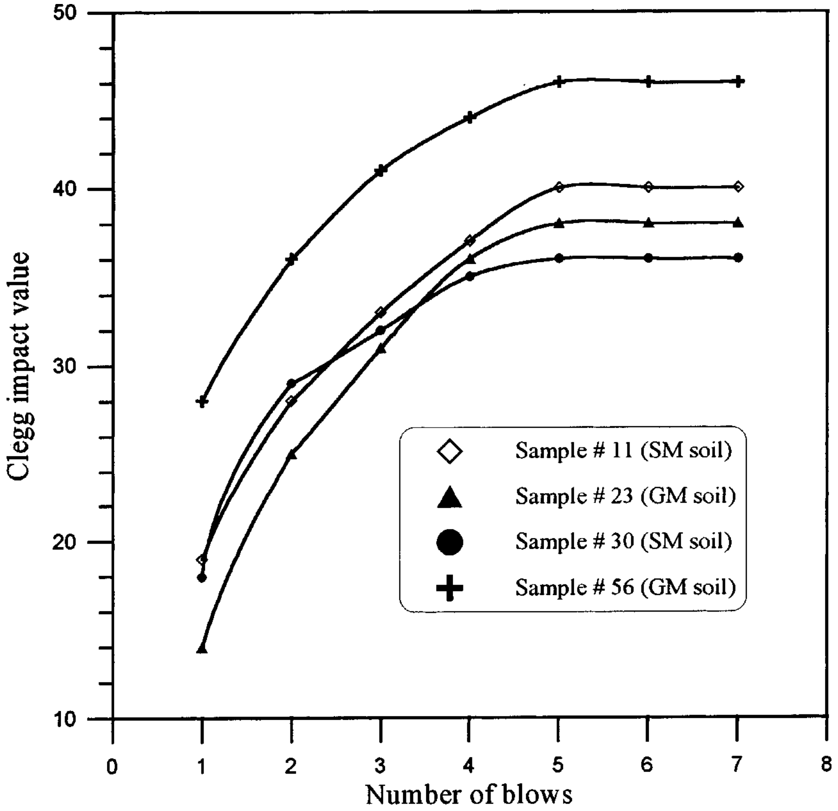

Fig. 8. Typical field CIV data

plastic limit, plasticity index, as well as the other basic information are presented in Table 2. Based on these data, the present marl can be classified as A-4 according to the AASHTO system and as SC according to the USCS system (Holtz and Kovacs 1981).

The dry density-moisture content relationships for the different compaction efforts are presented in Fig. 5. These curves indicate that the increase in the compaction effort resulted in an increase in the maximum dry density ($\gamma_{d\ \max}$) and a decrease in the optimum moisture content (OMC). The $\gamma_{d\ \max}$ was 1.960, 1.923, and 1.870 g/cm^3 (122.3, 120.0 and 116.7 lb/ft^3) for the 2,682, 1,245, and 591 kJ/m^3 compactive efforts, respectively, while the corresponding OMC values were 11.1, 12.3, and 13.3%. Similar performance has typically been reported for normal soils (Holtz and Kovacs 1981). The curves shown in Fig. 5 were used to prepare the CBR-CIH specimens at different moisture content-dry density combinations for each of the three compactive efforts reported herein. Only for the maximum compactive effort (2,682 kJ/m^3), two sets of specimens were prepared; one set for the dry CBR-CIH test and the other one for the soaked test.

A summary of the laboratory CBR and CIV test results for the various compactive efforts, test conditions, and moisture contents is numerically presented in Table 3 and schematically shown in Fig. 6. Typical variations of the raw data of CBR and CIH tests are depicted in Fig. 7. The data in Fig. 7 are for the medium compactive effort (1,245 kJ/m^3). However, all the other data follow, more or less, the same trend. The data in part (a) of Fig. 7 show the variation of the load with penetration for the CBR test and the five curves in this figure are for the specimens prepared with different moisture contents. Similarly, the data for CIH test are reported in Fig. 7(b), whereby the number of blows was plotted against the CIV, as previously explained. It can be easily recognized that the CIV increased initially with the number of blows and stabilized (did not change) at the fourth or fifth blow. In the case of field tests, the typical data shown in Fig. 8 indicate clearly that the CIV values did stabilize at the fifth blow for all the selected four samples shown in the figure. Accordingly, in this investigation, the CIV values at the fifth blow were selected to represent the soil strength for both the laboratory and field tests.

The CBR and CIH test results in Fig. 6 indicate that the maximum CBR and CIV values were attained by the specimens having 12.0% moisture content, while the specimens having the highest moisture content had the minimum values. It is to be noted that the maximum strength of eastern Saudi soils always occurs at a moisture content slightly lower than or at the optimum moisture content obtained from the compaction test results (Aiban et al. 1999). Comparison of the data in Figs. 5 and 6 indicates a similar trend for the calcareous soil used in this investigation. Such a trend could be observed for the three compactive efforts used herein.

Regarding the in situ investigation, Table 4 summarizes the results of the field tests. These results were presented in terms of the soil classification, CBR and CIV. A total of 56 field samples from various parts in Saudi Arabia were tested with a range of CBR and CIV values of 18 to 79 and 14 to 66, respectively. It is worth mentioning that although there is a general proportionality between the CBR and CIV results, the maximum CBR value in Table 4 (sample No. 10) does not coincide with the maximum CIV value (sample No. 35). The same observation was noted for the laboratory results in Table 3. This behavior could be attributed to each of the testing mechanisms of CBR and CIH tests.

Table 4. Summary of the in situ CBR-CIV Results

Sample number	Soil class	CBR (%)	CIV[a]
1	SM	30	25
2	SM	26	20
3	SM	28	15
4	SM	59	36
5	SM	41	28
6	SM	37	22
7	SM	38	24
8	SM	35	29
9	SM	41	35
10	SM	79	49
11	SM	57	40
12	SW-SM	36	26
13	SM	38	30
14	GM	36	31
15	SM	20	14
16	GM	42	33
17	SM	18	16
18	SM	20	15
19	SM	25	19
20	SM	39	36
21	SM	56	45
22	GM	42	30
23	GM	53	38
24	GM	29	21
25	SM	38	28
26	SM	45	34
27	SM	43	32
28	GM	25	18
29	SM	48	36
30	SM	45	36
31	SM	57	34
32	GM	29	14
33	SM	61	38
34	GM	39	25
35	SM	41	66
36	SM	43	34
37	SM	51	36
38	SM	47	37
39	GM	42	27
40	SM	57	38
41	SM	39	24
42	GM	49	35
43	SM	55	39
44	SM	53	38
45	SM	20	16
46	GM	51	28
47	SM	53	34
48	SM	26	19
49	SM	33	26
50	SM	54	39
51	SM	44	33
52	GM	61	40
53	SM	68	45
54	SM	54	35
55	SM	63	41
56	GM	71	46

[a] Based on the fifth CIH drop reading

Correlation of Test Results

To correlate the CBR values with the CIV results, different models were initially studied to arrive at the best fit among these two parameters. The models investigated were linear, exponential, and binomial as follows:

$$\mathrm{CBR} = a + b\ (\mathrm{CIV}) \tag{1}$$

$$\mathrm{CBR} = a * (\mathrm{CIV})^{b} \tag{2}$$

$$\mathrm{CBR} = a + b\ (\mathrm{CIV}) + c\ (\mathrm{CIV})^{2} \tag{3}$$

where a, b, and c are constants. These models cover a variety of statistical relationships that vary from the simple linear model to the exponential one. Each model was evaluated based on its coefficient of determination (R^2), standard error of estimate (SEE), and the statistical F-test. The R^2 value represents the proportion of variability in the data explained or accounted for by the regression model. The SEE measures the dispersion of the observed values about the regression line. While the F-test evaluates if there is a relation between the dependent variable (CBR) and the independent variable (CIV) and if the suggested type of the relation is the correct one. A well-known statistical package, ANOVA, was used in this analysis (SAS Introductory Guide 1985).

The linear model was included because Al-Amoudi et al. (1999) has recently reported a linear relationship between the CBR and CIV for cement-stabilized sabkha soil. Al-Ayedi has also reported a linear correlation between the CBR and CIV for lime-stabilized sabkha soil (Al-Ayedi 1996). However, the findings of our initial test results indicated that the best correlation model between the CBR values and the Clegg hammer CIV parameter should be of the exponential form (Asi et al. 1992).

The correlation of the laboratory test results is depicted in Fig. 9. The data therein were presented in terms of the compactive efforts and test condition as shown in Table 3. The best fitting model for the laboratory data is represented by the following exponential relationship: $\mathrm{CBR} = 0.1977\ (\mathrm{CIV})^{1.535}$ with a coefficient of determination (R^2) of 0.81 and SEE of 0.4790. It is to be noted that one data point was excluded as an outliner from the whole laboratory data set.

The relationship between the CBR and CIV parameters for the field test results is shown in Fig. 10. The field data were presented in terms of the soil type; whether GM or SM. The best fitting relationship for the in situ results can be modeled by the following exponential equation: $\mathrm{CBR} = 1.349\ (\mathrm{CIV})^{1.012}$ with an R^2 of 0.85. However, when the correlation is presented in terms of each of the two soil types alone, the following relationships represent the best fitting models:

$$\text{For GM Soil:}\quad \mathrm{CBR} = 0.861\ (\mathrm{CIV})^{1.136}$$

$$\text{For SM Soil:}\quad \mathrm{CBR} = 1.3577\ (\mathrm{CIV})^{1.011}$$

The R^2 values for these two models are 0.76 and 0.85, respectively.

Comparison of the laboratory and field tests results reveal that the laboratory data are less reliable due to the lower R^2 value(0.81 for the laboratory as compared with 0.85 for the in situ data) despite the fact that the laboratory specimens were prepared and tested under much better quality control. The reason for the lower R^2 value is probably ascribable to the smaller number of samples that were tested in the laboratory and, hence, decreasing the accuracy of correlation (Montgomery and Peck 1982; Montgomery 1984; Al-Amoudi et al. 1999). In addition, the variation in the properties of the laboratory samples (i.e., in terms of their dry

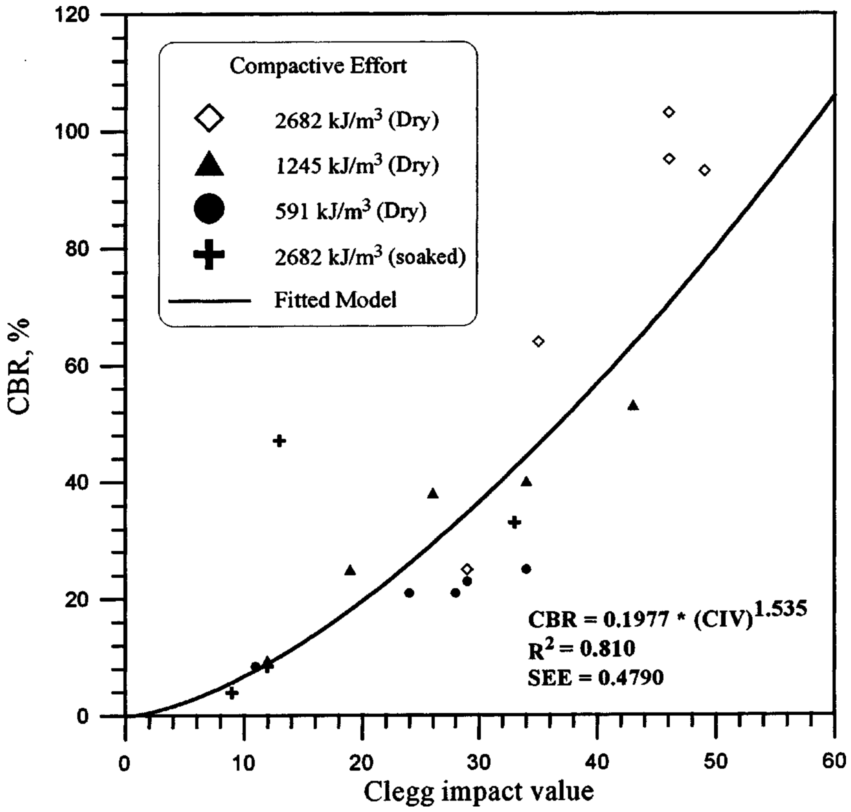

Fig. 9. Best-fit model for the CBR and CIV laboratory test results

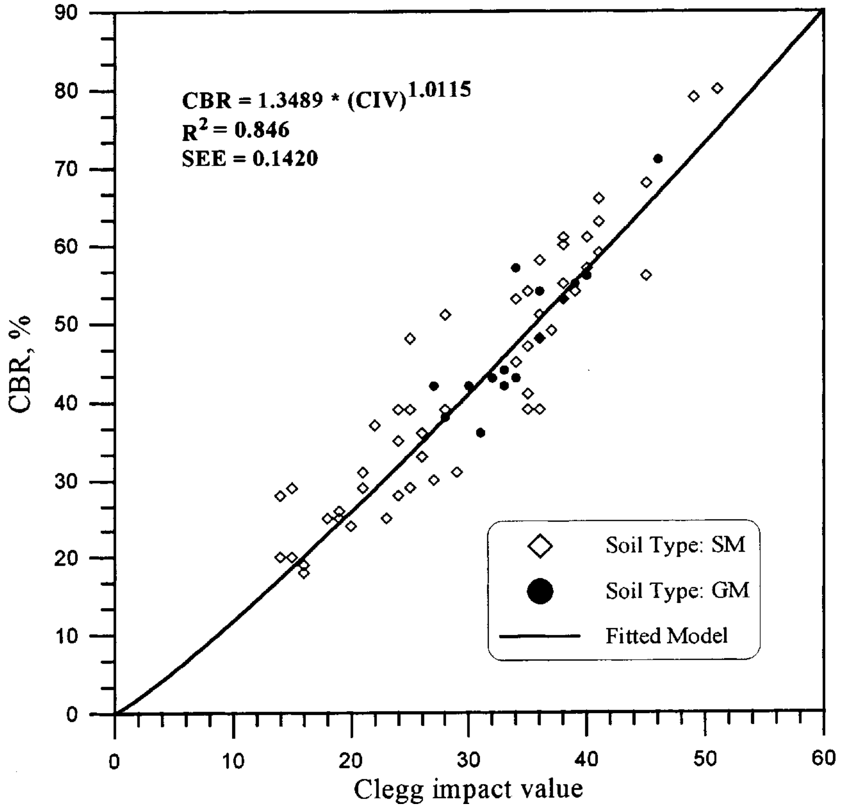

Fig. 10. Best-fit model for the CBR and CIV field test results

Table 5. Summary of the Correlations for the Field and Laboratory[a] CBR-CIV Relationships

Type of test	Correlation equation	R^2	SEE
Laboratory[a]	$CBR = 0.1977\ (CIV)^{1.535}$	0.810	0.4790
In situ			
GM Soil	$CBR = 0.8610\ (CIV)^{1.1360}$	0.757	0.0936
Sm Soil	$CBR = 1.3577\ (CIV)^{1.0105}$	0.845	0.1545
GM & SM Soils (combined)	$CBR = 1.3489\ (CIV)^{1.0115}$	0.846	0.1420
Literature			
Clegg (1980)	$CBR = 0.07\ (CIV)^{2.0}$	0.788	b
Mathur and Coghlans (1987)	$CBR = 0.1085\ (CIV)^{1.863}$	0.787	b
General Model[a]	$CBR = 0.1691\ (CIV)^{1.695}$	0.850	0.1719

[a] Based on laboratory in situ and literature data.
[b] Not reported.

density and moisture content) is more than that of the field samples and, therefore, might have contributed to the scatter. The same first reasoning can explain the lower R^2 value for the correlation of the GM soil type tested in the field as compared with the results of SM soil (i.e., the number of SM soil samples was 43 compared to 13 for the GM soil samples). When the samples of both the SM and GM soils were combined, the accuracy was marginally improved. This marginal improvement can be evidenced by the minimal improvement in the R^2 and SEE values. The R^2 value increased from 0.757 and 0.845 for GM and SM soil samples, respectively, to 0.846. Similarly, the SEE changed from 0.936 and 0.1545 for GM and SM soil samples, respectively, to 0.1420.

To elaborate further on the CBR-CIV correlations, the data reported by Clegg (1980) and Mathur and Coghlans (1987) were statistically analyzed in a similar way to the data reported in this paper. The results of this analysis indicate the following correlations:

$$\text{Clegg (1980):}\quad CBR = 0.07\ (CIV)^{2.0}$$

$$\text{Mathur and Coghlans (1987):}\quad CBR = 0.11\ (CIV)^{1.86}$$

Both groups of data have the same R^2 value of 0.79. It is interesting to note that the mode of correlation and the R^2 value reported by Mathur and Coghlans (1987) and Clegg (1980) are similar to those developed in this investigation, as summarized in Table 5.

It seems that the variation between the various models in Table 5 is primarily in the two constants (a and b) in Eq. (2) though the value of R^2 for all the models is around 0.8. This may suggest that one model may present a reliable tool to predict the CBR value from CIH test results. To develop such a generalized model, all the data developed in the laboratory and the field as well as the data reported by Clegg (1980) and Mathur and Coghlans (1987) were simultaneously statistically analyzed using the SAS package (1985) to produce the following general best-fit model:

$$CBR = 0.1691\ (CIV)^{1.695}$$

The coefficient of determination (R^2) for this model is 0.850; and SEE is 0.1719. It is known that if R^2 is more than 0.8, then the model can practically be considered as reliable (Montgomery 1984; Montgomery and Peck 1982). Therefore, this model is presented to the civil engineering community for use to estimate the CBR values of compacted soils using the Clegg impact hammer, particularly within the range of CBR and CIV values reported in Tables 3 and 4. Though this model was developed for many different types of soils, it is recommended to conduct few trial CBR-CIH tests to verify the reliability of this model for any proposed soil to be used in construction.

Concluding Remarks

This investigation was undertaken with the primary objective to correlate the CBR and CIH test results in both the laboratory and the field. The CBR and CIV data were statistically analyzed to develop predictive models that are reliable and capable of estimating the CBR values from CIV results. To fulfill this objective, laboratory and field CBR and CIH tests were conducted. Based on the findings of experimental and statistical analyses, the following main conclusions can be drawn:

- The maximum CBR and CIV values for the laboratory tested marl soils occurred at or lower than the optimum moisture content.
- The stabilization of CIV readings occurred at the fifth blow for both the laboratory and field tests.
- The CIV data correlated exponentially well with the CBR results.
- The best regression models developed in this investigation were in close similarity with those reported in the literature.
- Based on the results developed in this investigation and those reported in the literature, a general best-fit model [$CBR = 0.1691\ (CIV)^{1.695}$] was developed that can reliably predict the CBR values from CIV data. This model can virtually be used in any earthwork site, provided that the CBR and CIV values are within those reported in Tables 3 and 4.

Acknowledgments

The writers would like to acknowledge the support of the Department of Civil Engineering at King Fahd Univ. of Petroleum and Minerals. Thanks are extended to Hasan Zakariya Saleh and Efren C. Superales for their assistance during the experimental work and typing of the manuscript, respectively.

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