UNCERTAINTY ASSOCIATED WITH PRE-DEFINED CORRELATIVE EXPRESSIONS OF VARIOUS IN-SITU TEST OUTPUTS

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ABSTRACT

The paper deals with the following full-scale and small-scale strength and stiffness measuring devices: Dynamic Cone Penetrometer (DCP), Vane-Shear Strength (VSS), Falling Weight Deflectometer (FWD), and the Light Drop Weight (LDW) tests. Various established correlative expressions between CBR and each of the following testing outputs are given in the technical literature: (a) DCP index, (b) VSS, (c) M_R (backcalculated Resilient Modulus from FWD testing or Resilient Modulus from direct laboratory testing), (d) M_{FWD} (Resilient Surface Modulus, also known as Stiffness, from FWD testing), and (e) M_{LDW} (Resilient Surface Modulus, also known as Stiffness, from LDW testing). The paper presents a comparison of local correlative expressions with some of those described. It indicates that the variation in the correlative expression output results for each type of test makes their use entirely uncertain, at least for the studies carried out in Israel. Although some good correlations have been obtained in various cases, the results have been found to be material dependent, and so the equations should be used with care and only with a full understanding of the material properties of the soils on which the correlative expressions were developed and of the soil being tested.

INTRODUCTION

In recent years, there has been an increasing number of full-scale and small-scale strength and stiffness measuring devices available in many countries around the world for characterizing subgrades and granular layers. These tests include, inter alia, the Dynamic Cone Penetrometer (DCP), the Vane-Shear Strength (VSS), the Falling Weight Deflectometer (FWD), and the Light Drop Weight (LDW) tests.

Various correlative expressions have been established, and published in the technical literature, between CBR and each of the following testing outputs: (a) DCP index, (b) VSS, (c) M_R (Resilient Modulus from FWD testing on the pavement surface or from cyclic triaxial testing on prepared representative subgrade samples), (d) M_{FWD} (Resilient Surface Modulus, also known as Stiffness, from FWD testing of the subgrade soil surface), and (e) M_{LDW} (Resilient Surface Modulus, also known as Stiffness, from LDW testing on the subgrade soil surface). For example, the Mechanistic-Empirical Pavement Design Guide (developed under NCHRP Projects 1-37A and 1-40D), also known as the suggested 2002 AASHTO Guide for Design of Pavement Structures, makes use of the subgrade CBR from DCP and, through this value, the resilient modulus of the subgrade. These two measures are calculated with the aid of pre-defined correlations given in the Design Guide. In the same manner, the FAA's LEDFAA 1.3 program utilizes a pre-determined relationship between subgrade modulus and subgrade CBR although it allows the use of resilient modulus and non-destructive (NDT) data where the reliability of the measurements is felt to be high.

For the same two variables, the above-mentioned correlative expressions lead, in many cases, to different outputs. Given this background, therefore, the objectives of the present paper were formulated as follows:

- To present the various correlative expressions of CBR with the other described measures: DCP index, VSS, M_R, M_{FWD}, and M_{LDW}.
- To compare local correlative expressions with some of those described abroad and to analyze the uncertainty involved.

At this point, it would be worthwhile to offer the following note, based on similar lines of thought imparted by Freeman et al. [1]. Pavement engineering continues to advance reliability-based design procedures. These approaches commonly consider the variability associated with construction materials, pavement thickness, and traffic. Uncertainty associated with use of predefined correlative equations in frequently used in-situ testing (such as DCP, LDW, etc.), however, is considered less often. The contribution of this aspect of design to reliability can be important and warrants consideration.

Finally, the sections to follow will detail the process of reaching this paper's two objectives and their associated conclusions.

DCP INDEX VERSUS CBR

The DCP test for determining the in-situ California Bearing Ratio (CBR) and, more recently, the resilient modulus (M_R) has been used increasingly in many parts of the world in the past two decades. This is due to the fact that the test is economical, simple, and able to provide a rapid measurement of the in-situ strength of pavement layers and subgrades without the need for excavating the existing pavement as in the CBR test.

The DCP testing procedure is detailed in ASTM D6951-03. In this procedure, the number of blows is recorded with the depth of penetration. Then, the DCP index is calculated. This index is defined by the slope of the curve relating the number of blows to the depth of penetration (in mm per blow) at a given linear-depth segment. Now, in order to be able to relate the DCP index value to structural pavement parameters under local pavement-design technologies, extensive controlled laboratory and field tests have been carried out in many parts of the world. According to Livneh et al. [12], the quantitative relationship between CBR and DCP index values for any given material is given as

$$\log CBR = 2.20 - 0.71\times(\log DCP)^{1.5} \tag{1}$$

where DCP is the penetration index in mm per blow.

This correlation was found to be valid for a wide range of granular and cohesive materials. Furthermore, it should be mentioned that other researchers [2, 3, 4, 5, 6, 7, and 8] have also found the following relationships:

$$\log CBR = A - B\times\log DCP \tag{2a}$$

$$CBR = \frac{C}{DCP^{B}} \tag{2b}$$

and

$$CBR = \frac{2559.44}{-7.35 + DCP^{1.84}} + 1.04 \tag{3}$$

where A, B, and C are the regression coefficients, as given in Table 1, for which A=logC; also, Equation 3 is according to Nazzal [10].

Figure 1 displays Equation 1 (Curve 12; R^2=0.85, N=152), Equation 2 (Curves 1 to 9), and

Equation 3 (Curve 10) graphically for the respective A, B, and C coefficients given in Table 1. Curve 3 (R^2=0.81) of this figure, it should be noted, is recommended by the U.S. Army Corps of Engineers (3). This curve is used for all soils except for CL soils below CBR 10% and CH soils. For those soils, it recommends the following curves: Curve 5 (R^2=0.12) and Curve 6 (R^2=0.72), respectively.

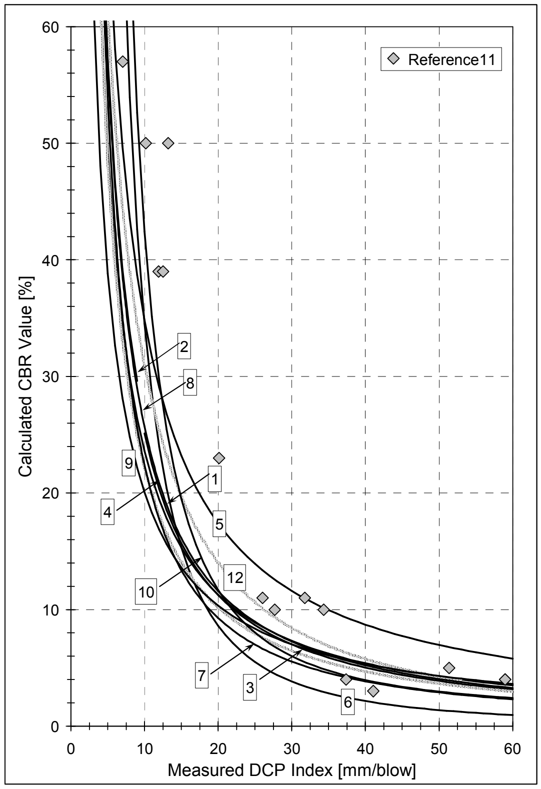

Figure 1. Calculated CBR Versus DCP Index According to Various Sources.

Table 1.
Values of A, B, and C Coefficients in Equations 2a and 2b According to Various Sources.

Curve No.	Reference	Type of Material	A	B	C
1	2	All types with DCP≥10	2.560	1.160	363
2	2	All types with DCP<10	2.540	1.120	347
3	3	All types (except CL & CH)	2.465	1.120	292
4	4	All types	2.438	1.065	274
5	5	CH only	2.542	1.000	358
6	5	CL with CBR<10% only	3.538	2.000	3,452
7	6	All types	2.620	1.270	417
8	7	All types	2.560	1.150	363
9	8	All types	2.256	0.954	180

Figure 1 includes recent experimental data of measured in-situ CBR values versus recorded DCP index values. These data-points are taken from Philips [11]. It seems that they are compatible with Curve 12 (Equation 1).

Figure 1 demonstrates that the differences among the various expressions are relatively moderate in the region of DCP≥20 mm per blow, except for the values obtained by Curve 5 for CH materials and Curve 6 for those CL materials that possess CBR values equal to or smaller than 10%, for which the differences are rather high. These differences are shown numerically in Table 2. For DCP=40 mm per blow, the maximum CBR value obtained from all curves is 8.7% (Curve 5) and the minimum CBR value is 2.2% (Curve 6). This is a considerable difference. As for the values obtained from all curves except Curves 5 and 6, the comparative CBR values are 5.8% (Curve 12) and 3.8% (Curve 7). Even this difference has a considerable effect on the final thickness design. For example, Garg et al. [9] show in their sensitivity analysis for the FAA's airport pavement-thickness design that the sensitivity to changes in the value of subgrade CBR may amount to a high value of 12. Thus, for a 20% change in the value of subgrade CBR, there is a change of 240% in pavement life.

In this context, it is important to mention that Burnham [13] of the Minnesota Department of Transportation makes use only of the Corps' overall correlative relationship (see Curve 3). Furthermore, the suggested 2002 AASHTO Guide for Design of Pavement Structures suggests the use of Curve 3 for all types of soils. These facts are compatible with the work done in [14] to validate the existence of a single relationship for all types of clays. To this end, results from light to heavy clayey test pits enabled a comparison of the CBR values obtained from DCP tests using Equation 1 with those from undisturbed samples extracted from the pits. Statistical analyses performed on the results indicate that the null hypothesis of the equality of means --i.e., that the mean of the CBR values derived from the indirect DCP tests is equal to the mean of the CBR values derived from the direct CBR tests--cannot be rejected.

Table 2.
Values of Calculated CBR for Defined DCP Index Values According to Various Sources.

DCP	Curve Number of Figure 1									
mm/blow	1	3	5	6	4	7	8	9	12	10
20	11.2	10.2	17.4	8.6	11.3	9.3	11.6	10.3	14.0	11.7
30	7.0	6.5	11.6	3.8	7.3	5.5	7.3	7.0	8.4	6.0
40	5.0	4.7	8.7	2.2	5.4	3.8	5.2	5.3	5.8	4.0
50	3.9	3.6	7.0	1.4	4.3	2.9	4.0	4.3	4.2	3.0
60	3.1	3.0	5.8	1.0	3.5	2.3	3.3	3.6	3.3	2.4

The aforementioned facts demonstrate the uncertainty associated with the use of pre-defined correlative expressions for translating in-situ DCP test outputs into CBR values. In this regard, ASTM D6951-03 states that the selection of the appropriate correlation is a matter of professional judgment. It is not certain, however, that the appropriate correlation can be really pointed out for all cases. To this uncertainty, one should add the repeatability standard deviation effect of 20% or less according to ASTM D6951-03 or to the findings reported by Freeman et al. [15]. Thus, for a DCP index of 50 mm per blow, the range of calculated CBR values for ± one standard deviation is given in Table 2 for DCP indices of 40 and 60 mm per blow.

Similar conclusions concerning the DCP index and R-value relationships are reported by Jones and Harvey [35]. These authors state the following: "Any relationships already developed between DCP penetration and R-value should be used with extreme caution, if those relationships were developed outside of the area and/or on difficult soils in which the DCP penetrations have been carried out."

VSS VERSUS CBR

For cohesive soils, the Vane Shear Test is used to measure their in-situ shear strength. Several investigations reported in the technical literature describe correlation analyses of VSS (the vane shear strength) and CBR. The output of these analyses is given in the following expression:

$$CBR = q \times 10^{-2} \times VSS^{r} - t \quad (4)$$

where q, r, and t are the regression coefficients, as given in Table 3; VSS is given in kPa; and CBR is given in percentages.

Table 3.
Values of q, r and t Coefficients in Equation 4 According to Various Sources.

Curve No.	Reference	q	r	t	R^2	N
1	22	4.30	1.000	0.2	0.62	86
2	23	3.70	1.000	1.5	0.66	38
3	24	9.00	1.000	0.0	---	---
4	14	21.10	0.653	0.0	0.52	70
5	25	9.07	0.810	0.0	0.56	29
6	37	6.57	1.014	0.0	---	---
7a	26	8.26	1.000	3.5	0.85	130
7b	26	0.36	1.554	0.0	0.89	130

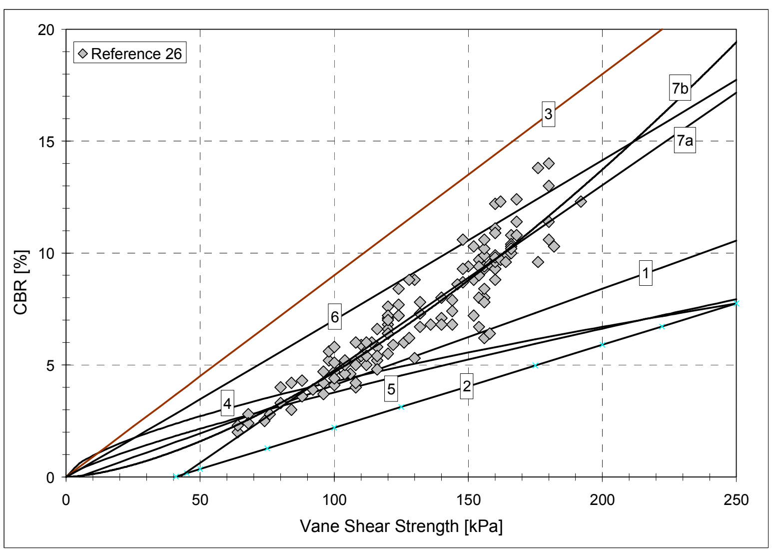

Figure 2. Calculated or Measured CBR Versus Vane Shear Strength According to Various Sources.

Figure 2 displays Equation 4 graphically for the q, r, and t coefficients given in Table 3. The figure shows again the uncertainty associated with the use of pre-defined correlative expressions for translating in-situ VSS outputs into CBR values. In the figure, Curves 4 and 5 represent the current Israeli practice. It seems that Curves 2 and 3 represent two extreme cases, while the other curves represent almost similar results for the VSS=100 kPa region. For these curves and other values of VSS, the lack of knowledge of the “true” expression may lead to considerable errors, even of up to 100%.

Figure 2 also includes experimental data points reported by Garg [26]. For these data points, in-situ CBR and in-situ Vane Shear tests were performed in four excavated trenches, at their subgrade surface and 0.3 meter below the surface. These trenches were excavated in the Federal Aviation Administration’s (FAA’s) National Airport Pavement Test Facility (NAPTF), located in Atlantic City, NJ. It may be recalled that the full-scale testing performed at this test facility was generally aimed at establishing thickness-design criteria for current trends in New Large Aircraft (NLA) gears. The scatter of the measured points, shown in Figure 2, leads to a standard error in the CBR estimation of 1.0%. Thus this error increases the intensity of the general uncertainty issue.

CBR AND DCP INDEX VERSUS RESILIENT MODULUS

The elastic moduli for soil subgrades can be characterized by the resilient modulus and obtained from cyclic triaxial tests on prepared representative samples. The recommended standard tests methods for modulus testing are NCHRP 1-28 or AASHTO T 307. Because of the time and skill required to conduct these tests, approximate correlations between resilient modulus and some of the more easily measured parameters are utilized. The FAA allows the use of resilient modulus from non-destructive (NDT) data where the reliability of the measurements is felt to be high. Thus, over a period of several years, data have been accumulated from comparisons of the subgrade resilient modulus (M_R in MPa), some of which were determined from deflection bowl measurements of the pavement surface, with the in-situ CBR (in %) of the subgrade as measured in test pits. Following is the general expression obtained from these comparisons:

$$M_R = k \times CBR^{1/m} \tag{5}$$

where k and m are empirical parameters varying from source to source; M_R is the direct-cyclic triaxial test modulus or the FWD backcalculated modulus.

Equation 5 is displayed graphically in Figure 3 for frequently used values of k and m. The m=1.00 and k=10.3 or 14.0 values were taken from Heukelom and Klomp [16] and Uzan [17], respectively. The m=1.41 and k=15.0 or 20.0 values were taken from Livneh [14]. The TRL or the suggested 2002 AASHTO Guide for Design of Pavement Structures expression was taken from Powell et al. [18], for which k=17.6 and m=1.56. A similar expression was taken from the study performed by the South African Council on Scientific and Industrial Research, for which k=20.7 and m=1.56. Further, the U.S. Army Corps of Engineers expression was taken from Green and Hall [19], for which k=37.3 and m=1.41 and that of the Georgia Department of Transportation for medium clay sand was taken from Webb and Campbell [33], for which k=21.5 and m=2.09. Finally, the expression of FAA’s NAPTF for low to high strength clays was taken from McQueen et al. [47], for which k=23.2 and m=1.46.

Figure 3 indicates that the range of subgrade modulus estimations for a given CBR value is significant. For any given specific case, the lack of knowledge of its “true” expression may lead to a considerable error of up to 300% in the modulus estimation. Part of the discrepancy in this CBR estimation comes from the range of M_R values that can be obtained from applying various backcalculation-program codes to the same deflection bowl measurements, and from postulating various depths to a hard bottom. In Israel, the default expression for the M_R-CBR relationship is defined by k=20.0 and m=1.41 or, alternatively, by k=14.0 and m=1.00. For low CBR values, the difference in the subgrade modulus between these two expressions is insignificant. In addition, it should be mentioned that the TRL or the suggested 2002 AASHTO Guide for Design of Pavement Structures expression is almost the same as that defined by k=15.0 and m=1.41. On the other side, the FAA's default expression is defined by k=10.3 and m=1.00.

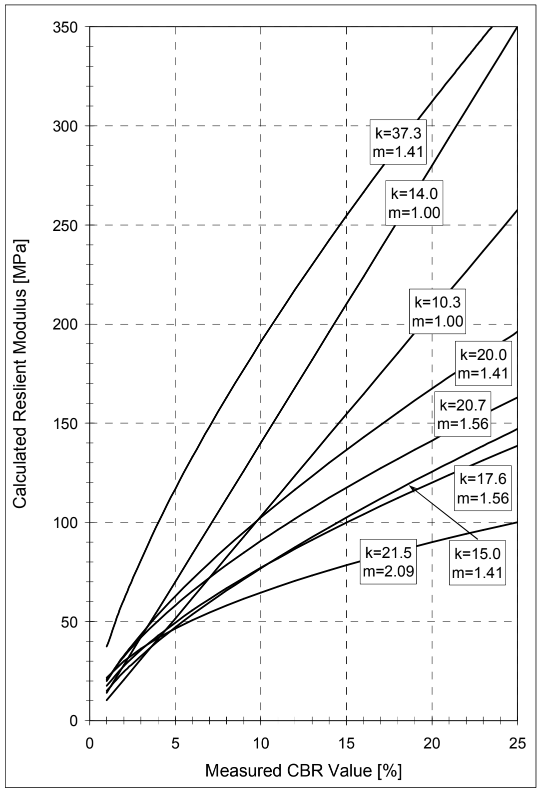

Figure 3. Calculated Subgrade Resilient Modulus versus CBR Values According to Various Sources.

Obviously, the relationship between the measured DCP values and the estimated M_R values can be derived by combining Equation 1 and Equation 3. The output of this combination for k=20.0 and m=1.41 is given in Equation 6 and Table 4 (see second row in that table); thus:

$$\log M_R = F - G \times \log DCP \tag{6}$$

where F and G are the regression coefficients, as given in Table 4; DCP is given in mm per blow; and M_R is given in MPa.

For reasons of comparison, Table 4 contains additional values for F and G as derived from other sources. Equation 6 is also displayed graphically in Figure 4 for the values of F and G listed in Table 4.

Figure 4 indicates that the Israeli default relationship (Curve 1) is very much comparable with all other curves, expect for Curves 5, 6, and 7. The discrepancy associated with those curves is of much concern. In any case, great care should be taken in the selection of a specific relationship for practical utilization, or as George [34] states: “Simple strength correlations, for example, the CBR test to estimate resilient modulus, should be used with caution.”

Table 4.
Values of F and G Coefficients in Equation 5 According to Various Sources.

Curve No.	Reference	Details	F	G
1	---	k=20.0 and m=1.41	3.026	0.759
2	20	Subgrade - Phoenix Program	3.250	0.890
3	20	Subgrade - Peach Program	3.662	1.170
4	14	All types of material	3.330	1.000
5	21	All types of material	3.048	1.062
6	29	Granular material	2.618	0.250
7	29	Cohesive material	2.012	0.168

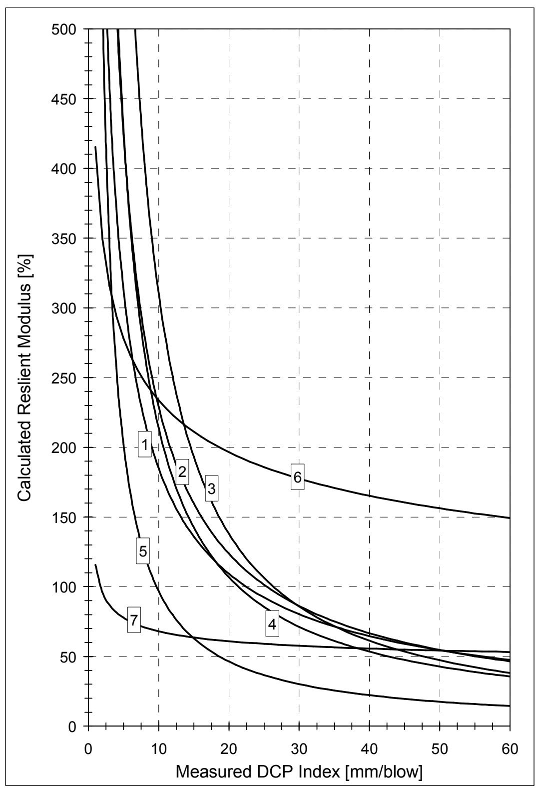

Figure 4. Calculated Resilient Modulus Versus DCP Index According to Various Sources.

It should be noted here that Curve 7 of Figure 4 has been calculated from the following expression, derived by Gudishla [29]:

$$M_R = 1100 \times \frac{DCP^{-0.44}}{w} + 2.3 \times \gamma_d \quad (7)$$

Median values of moisture content (w=14.7% for the 8.5%-20.9% range) and of dry density (γ_d=17.1 kN/m^3 for the 15.3 -18.9 kN/m^3 range) were substituted in Equation 7 for calculating Curve 7 of Figure 4. M_R of Equation 7 is also given in MPa.

Finally, the remarkable range of M_R values given in Figures 3 and 4 can also be supported by various findings. The 1993 AASHTO guide [38], for example, allows the use of both laboratory and in-situ backcalculated moduli; however, it recognizes that moduli determined from both procedures are not equal. The guide, therefore, suggests that the subgrade modulus determined from deflection measurements of the pavement surface be adjusted by a factor of 0.33. Other ratios, ranging from about 0.2 to about 1.2, have been documented in the technical literature. A detailed discussion of the differences between laboratory measured M_R (lab) and backcalculated can be found in [39, 40, and 41]. It also includes the deviator-stress effects.

To sum up, no unique relationship can be found between laboratory measured M_R and backcalculated M_R, just as no unique relationship can be found between laboratory measured M_R and backcalculated M_R and CBR. This is a very important finding in light of the statement made in the 1993 AASHTO guide [38]: "Users are cautioned that the resilient modulus value selected has a very significant effect on the resulting structural number determined. Therefore, users should be very cautious about using high resilient modulus values, or their overlay thickness values will be very thin."

FWD AND LDW RESILIENT MODULUS VERSUS CBR

Combined end-product and mechanistic-empirical-based specifications call for including formation and foundation stiffness tests together with in-situ density tests. Toward this end, the conventional Falling-Weight Deflectometer (FWD) is classified as a suitable device for obtaining stiffness measurements with a non-standard 450mm diameter plate and a 200kPa contact stress.

Target values for deformation-stiffness to characterize the construction of formations and foundations have started to appear in the technical literature [27]. *Formation* here means the part of the pavement structure that contains the natural and compacted subgrade and the capping layers on top of this subgrade. *Foundation* means the part of the structure containing the formation and the granular subbase layers lying on the above formation.

Experimental data indicate that the resilient modulus derived from FWD testing (M_{FWD} in MPa, also known as the FWD surface modulus or stiffness) is correlated with the CBR values in the following manner:

$$M_{FWD} = \alpha \times CBR^{1/1.41} \quad (8)$$

According to Figure 5, α is equal to 12.7 for the experimental data measured by Philips [11] when CBR is measured by direct testing, or α is equal to 12.9 when CBR is measured by DCP testing. The standard errors obtained for these two correlations are 56.7MPa and 48.2MPa, respectively, which are rather high values. Moreover, a similar value of α equal to 12.1 was obtained for the experimental data measured by Nazzal [10]. Comparing Equation 8 to the data of Figure 4 indicates that direct FWD testing on the subgrade leads to lower values of resilient modulus than those obtained from backcalculation of FWD deflections measured on the pavement surface.

In this context, it is interesting to compare the resilient moduli obtained from the above-mentioned direct FWD testing of the subgrade with the resilient moduli obtained from laboratory testing. This comparison is made in the paragraph to follow.

Several studies have targeted FWD tests conducted directly on the subgrade surface. In their study of the Minnesota Road Research Project (Mn/ROAD), Van Deusen et al. [42] reported a week correlation between laboratory and backcalculation moduli. Resilient modulus versus first sensor elastic modulus was explored in a study called the Virginia Smart Road Project [43]; the relationship, however, was less than satisfactory. Employing SPS-1 and SPS-2 data, Stubstad et al. [44] compared laboratory M_R and composite moduli calculated from FWD tests conducted directly on the subgrade and demonstrated a satisfactory correlation. Another investigation, by Rahim and George [45], found that when backcalculated moduli are obtained from testing directly on the subgrade, they are in satisfactory agreement with the laboratory values. Finally, contrary to the foregoing, a recent investigation by George et al. [46] indicated that the dispersion of resilient modulus versus first sensor elastic modulus was quite significant as shown in Figure 2 of this reference. Again, the uncertainty in determining a design value for a subgrade M_R is very problematic.

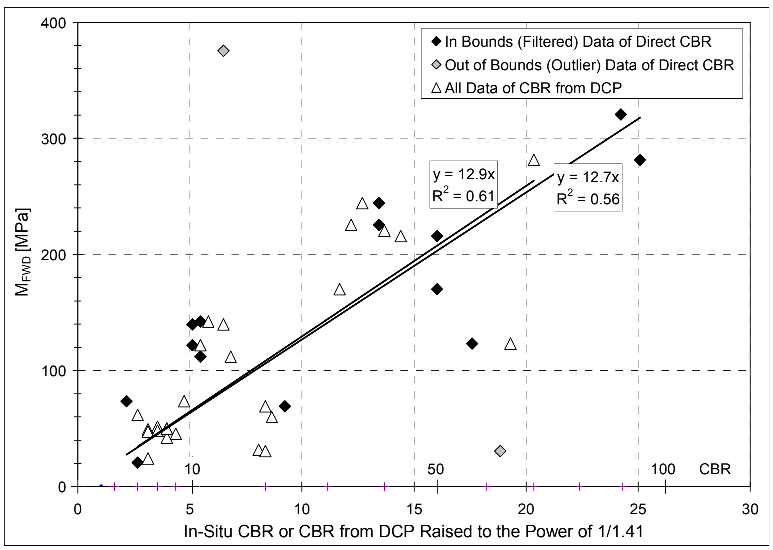

Figure 5. Resilient Modulus from FWD Testing (M_{FWD}) Versus In-Situ CBR or Calculated CBR from DCP [11].

The conventional FWD, classified as a suitable device for stiffness measurements, is sometimes considered unnecessarily sophisticated for formation and foundation testing. Furthermore, it is not without limitations on weaker substrates in regard to both transducer range limits and portability as discussed by Fleming [30]. Thus, the German Light Drop-Weight (LDW), also known as the German Dynamic Plate Test (GDP), which is lightweight, portable, and simple to apply for repeated testing, is used by various agencies around the world [28]. In the UK, it is also known as the Lightweight Drop Tester [32].

In order to evaluate M_{LDW} values (i.e., the resilient modulus values measured by the LDW device, also known as the deformation modulus, the LDW surface modulus, or LDW stiffness), test-pits were excavated at various locations containing clayey and sandy stratum. Comparative LDW and DCP tests were carried out on staggered surfaces, arranged at depths of about every half meter. The results of these tests have already been published elsewhere [27]. Restrained regression analysis of the 1.41 power function was conducted on the total combined test data as shown in Figure 6. It can be seen that the data of Figure 6 lead to the following expression:

$$M_{LDW} = \beta \times CBR^{1/1.41} \qquad (9)$$

where β equals to=4.33; M_{LDW} is given in MPa; and the standard error of the correlation derived equals 10.7 MPa.

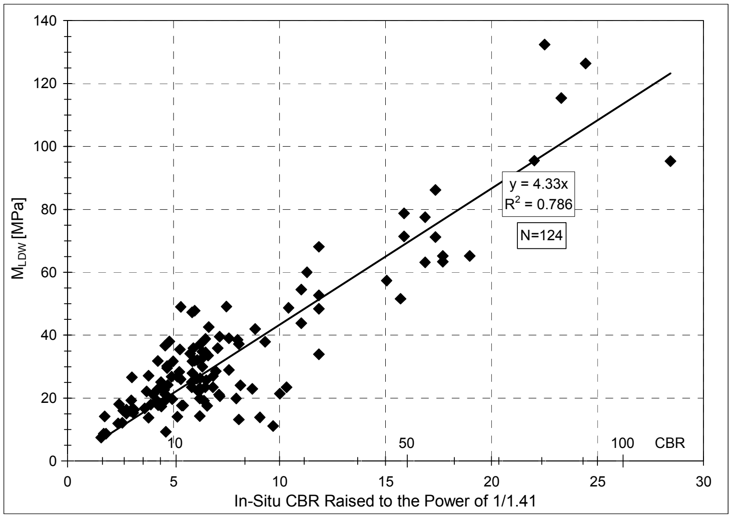

Figure 6. Resilient Modulus from LDW Testing (M_{LDW}) Versus Calculated CBR from DCP Testing [11].

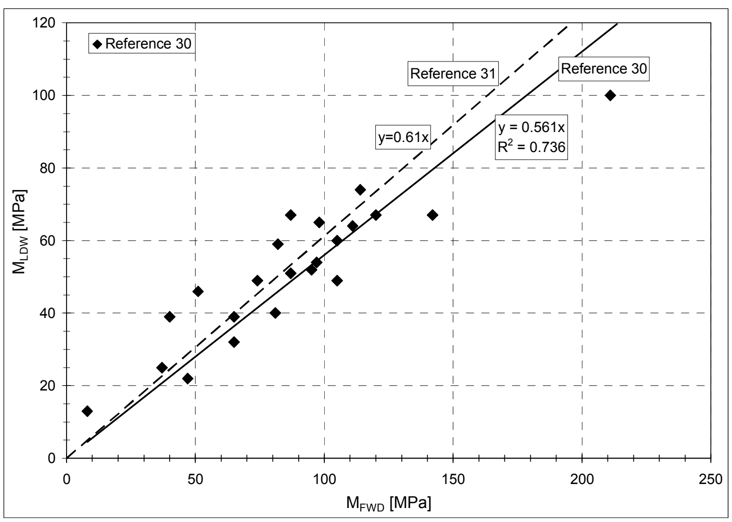

Figure 7. LDW Resilient Modulus Versus FWD Resilient Modulus According to Reported Data.

At this juncture, it should be noted that M_{LDW} values are much smaller than M_{FDW}, about 0.35 times M_{FDW}. The same phenomenon was found in Fleming et al.'s comparative studies [30, 31], which contained the use of several portable deflectometers, together with the conventional FWD device. The data from these studies are given in Figure 7. This figure indicates that M_{LDW} is about 0.56 times to 0.61 times M_{FDW}. The considerable range in the M_{LDW} to M_{FDW} ratio (from 0.35 to 0.61) again makes the use of the above-mentioned relationships of Equation 8 and Equation 9 uncertian.

SUMMARY AND CONCLUSIONS

Recent years have seen in many countries around the world an increasing number of available full-scale and small-scale strength and stiffness measuring devices for characterizing subgrades and granular layers. These testing devices include, inter alia, the Dynamic Cone Penetrometer (DCP), the Vane-Shear Strength (VSS), the Falling Weight Deflectometer (FWD), and the Light Drop Weight (LDW) tests.

Various established correlative expressions, given in the technical literature and presented again in this paper, exist between CBR and each of the following testing outputs: (a) DCP index, (b) VSS, (c) M_R (Resilient Modulus from FWD testing on the pavement surface or from cyclic triaxial testing on prepared representative subgrade samples), (d) M_{FWD} (Resilient Surface Modulus, also known as Stiffness, from FWD testing on the subgrade soil surface), and (e) M_{LDW} (Resilient Surface Modulus, also known as Stiffness, from LDW testing on the subgrade soil surface).

The paper also presents a comparison of local correlative expressions with some of the ones described abroad. It argues that the variation in the output results of correlative expressions for each type of test makes their use entirely uncertain, at least for the studies carried out in Israel. Although some good correlations have been obtained in various cases, all the studies have found that the results are material dependent, and that equations should be used with care and only with a full understanding of the material properties of the soils on which the correlative expressions were developed and of the soil being tested.

Sensitivity analyses for pavement-thickness design show that changes in the value of the CBR subgrade impart the highest sensitivity of all material property input parameters. For airfield-pavement design, the sensitivity value can amount to a high value of 12. Thus, for a 20% change in the value of the CBR subgrade, there is a change of 240% in pavement life. Therefore, only a proven correlative expression for determining the CBR value should be used.

To highlight the problem raised in the preceding paragraphs, we advance the argument that there is a certain risk in implementing the aforementioned correlations. This risk is due to the solid possibility of the creation of a totally wrong interpretation of the tests results obtained. Adding this misinterpretation to the inherit variability of the tests results may considerably enlarge the problem.

Thus, it is very much recommended that any correlative expression should be implemented only after its validity is checked against limited in-situ testing. For example, for the M_R-CBR relationship in any rehabilitation design, it has been highly recommended elsewhere (36) that the required correlative expression be assessed by conducting both FWD and in-situ CBR (DCP) tests on existing pavements.

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