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INVESTIGATIONS ON UNSOAKED BLENDED LATERITE USING PFWD, PBT, DCP AND CBR TESTS †

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ABSTRACT

The stiffness of sub-grades estimated using the elastic modulus of deflection is used as the basis in present pavement design approaches. Recently, the use of the Portable Falling Weight Deflectometer (PFWD) and the Dynamic Cone Penetrometer (DCP) has gained popularity due to their inherent advantages in providing rapid estimates of sub-grade strength. The work described herein is focused on conducting an investigation on the effect of percentage of fines, maximum dry-density and voids ratio on blended laterite soils using the PFWD, PBT, DCP and CBR tests for compacting moisture contents at dry of optimum, optimum and wet of optimum, for unsoaked soil conditions. Correlations were also developed connecting the observations made using the PFWD, PBT, DCP and CBR tests.

1 INTRODUCTION

The service-life and performance of roads depend to a large extent on the strength and stiffness characteristics of sub-grade. Sub-grade characteristics assist road engineers in the selection of materials for sub-base and base courses of pavements. Hence, the evaluation of sub-grade strength assumes great importance in pavement design. Also, mechanistic and empirical design and rehabilitation procedures, require information on the stiffness of unbound material to predict the structural capacity of a pavement system[1]. In determination of the strength of soil sub-grades, the California Bearing Ratio (CBR) test that estimates the bearing capacity of pavement sub-grades with respect to the strength of well-graded high quality crushed stone aggregate, was used for a long time. The CBR method of pavement design introduced by O.J. Potter in the 1930s gained popularity in the late 1980s[2]. Although, it is considered to be an empirical design approach, it is still the most sought-after method in the design of pavements for military and civil aviation[3]. Soil texture, moisture and density are the major factors affecting the CBR of soil sub-grades[4].

Similarly, the Plate Bearing Test (PBT) is one of the most versatile equipments used in the determination of stiffness of road sub-grades and in proof testing pavement structure layers in European countries[5]. The PBT that mainly measures the deformations under rigid plates for various loading conditions, is used to estimate the Young's modulus (or the modulus of deformation), the recoverable compressibility of the soil[6] and the determination of the modulus of sub-grade reaction (k).

However, the development of expand Portable Falling Weight Deflectometer (PFWD) has revolutionized the field of pavement-evaluation mainly due to its simplicity and ease of use, portability, reliability and ruggedness. PFWD provides information on the composite-stiffness of sub-grades for all layers, up to a particular depth[7]. These are designed based on the working principle of the FWD[8].

Similarly, the Dyanmic Cone Penetrometer (DCP) too is a low-cost portable device that has gained popularity in the recent years. It permits rapid testing and evaluation of sub-grades and pavement layers in order to make reliable estimates of the CBR values of underlying layers effectively[9].

In view of the present pavement design procedures that combine mechanistic and empirical design approaches[10, 11] and the need for performing direct monitoring of stiffness of sub-grades during and after road construction, it was felt that there exists a need to correlate the results obtained using the PFWD, the CBR, the PBT and the DCP.

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2 PROBLEM FORMULATION AND SCOPE OF WORK

Approximately forty percent of the soil sub-grades of highways constructed in the District of Dakshina Kannada, the coastal regions of Karnataka and Kerala are lateritic in nature. In a number of cases related to realignment and widening of existing roads in these areas, road-engineers often encounter situations where laterite sub-grades mixed with silty soils are required to be compacted to prepare sub-grades for new pavement surfaces.

The name laterite was first given by Buchanan[12] to describe the "ferruginous, vesicular, unstratified and porous soil with yellow ochres due to high iron contents, occurring in Malabar, India". Dakshina Karnataka district is located close to the Malabar region.

Also, it is often observed during various stages of construction, that although it is required to achieve hundred percent relative compaction at Optimum Moisture Content (OMC) as measured using the standard proctor test, only about ninety-five percent of compaction level is achieved in practice due to minor changes in compaction-water-content, that tends towards 'dry-of-optimum' or 'wet-of-optimum'[13]. This is observed, although sparingly, even in major road construction projects where vibratory rollers are employed. This leads to under-compaction of subgrades, causing pavement failure.

The present study focuses on conducting an investigation on the effect of intrusion of silty soils and the effect of minor changes in field moisture conditions on the stiffness characteristics of laterite sub-grades. The PFWD, PBT, CBR and DCP tests were performed on remolded laterite soils blended with various percentages of silty soils on soil samples compacted at Maximum Dry Density (MDD) with moulding water content at the drier side of optimum (designated as M_1 for samples prepared at OMC-3%), optimum (designated as M_2 for samples prepared at OMC) and wetter side of optimum (designated as M_3 for samples prepared at OMC+3%) on unsoaked soil conditions.

The correlations developed using the Statistical Package for Social Sciences (SPSS), among the results obtained using the PFWD, PBT, DCP and the CBR have high R-square values, with a confidence level of more than 95 percent as indicated by the F test values and the p-value. The correlations developed above are expected to be applicable to a large extent for laterite soils of similar characteristics elsewhere.

3 LITERATURE REVIEW

The American Association of State Highway and Transportation Official (AASHTO) formulated the guidelines for analysis of pavement structures in 1986 for the mechanistic analysis and design of pavement structures[14, 15], considering the importance of the modulus of resilience, a measure of the stiffness of the sub-grade, on pavement design. The AASHTO guidelines of 1993 too recommend the need to employ the modulus of resilience for characterizing the base and sub-grade soils and for the design of flexible pavements.

But, the AASHTO Supplement-1998 to the AASHTO Design Guidelines, suggests a hierarchical design concept where for routine pavement designs, either the California bearing ratio or the resilient modulus of the soil may be used to represent the characteristic property of the subgrade soil for asphalt pavements, whereas the modulus of subgrade reaction of the soil obtained from plate bearing tests may be used in the case of concrete pavements. This has resulted in new research initiatives focused on development of relationships between results obtained using alternative methods of sub-grade and pavement evaluation using the CBR, the PBT, the DCP and the FWD[5].

3.1 Important Studies on Influence of Soil Parameters, Water-content, Gradation and Fines on the Modulus of Resilience

The modulus of resilience (M_R), also called as the resilient modulus, is defined as the ratio of the maximum stress (σ) to the recoverable elastic strain (ϵr). This plays a major role in pavement strength-evaluation. The resilient modulus of granular materials depends upon various factors including the loading-stress, water content, dry-density and the gradation[15, 16, 17]. In a similar study, Barksdale and Itani[18] observed a significant decrease in resilient modulus for four categories of sub-grade materials tested upon soaking. Raad et al[19] observed that the effect of water on the resilient properties is indeed significant in well-graded materials with a high amount of fines.

3.2 Important Studies on Correlations between CBR, DCP, PFWD and PBT

3.2.1 *Correlations using the CBR*

Extensive studies undertaken by Smith and Pratt[20], Harison[21], Livneh and Ishai[22], as reported by Farshad-Amini[23] and Abu-Farsakh et al[5]. have focused on investigating the correlations between the results obtained using the DCP and the CBR.

Powell et al[24] proposed one of the most widely accepted relationship between the CBR value and the modulus of resilience of the sub-grade measured using the FWD. Chen et al[7] reported studies conducted by AASHTO[25] for the design of pavements where a correlation model was developed for the determination of the sub-grade modulus (Es) based on CBR observations. The authors also developed correlations connecting measurements made using the DCP to the modulus of resilience obtained using FWD. Mohammad et al[26] (2007) developed two regression models for prediction of the resilient modulus of cohesive sub-grade soils based on the water-content (expressed in percentage) and the dry unit weight (expressed in kN/cu.m). With the development of the Light Weight Falling Deflectometer (LFWD), a low cost alternative to the FWD, further investigations were performed by Nazzal[27] on correlating the CBR values to the modulus of resilience measured using the Prima 100 (LFWD) (E_{pfwd}). The relationship developed for Louisiana soils is expressed as given in Eq.1.

$$CBR = -14.0 + 0.66\,(E_{pfwd}) \qquad ...(1)$$

3.2.2 *Relationships using the DCP and the PBT*

A relationship connecting the observations made using the DCP to the modulus of deformation obtained using the PBT for unbound aggregates and gravelly and sandy was developed by Konard and Lachance[28]. Seyman[29] (2003) correlated the initial-tangent modulus and the reloading-elastic modulus obtained using the PBTs to the observations made using the DCP. Abu-Farsakh et al[5] conducted investigations on similar lines and developed a correlation for the field data.

3.2.3 *Relationships using the PLT*

Seyman[29] derived the correlation relating the modulus of resilience obtained using the LFWD (E_{LFWD}), to the initial tangent modulus determined using the PBT ($E_{PBT\,(i)}$), R-square value of 0.844 as shown below:

$$E_{PBT(i)} = 0.907*(E_{LFWD}) - 1.8 \qquad ...(2)$$

Nazzal et al[30] (2007) developed linear regression models relating the modulus of resilience obtained using the LFWD (E_{LFWD}), to the initial tangent modulus ($E_{PBT\,(i)}$), and the reloading tangent modulus obtained from the PBT ($E_{PBT\,(R2)}$).

$$E_{PBT\,(i)} = 1.041*E_{LFWD} \qquad ...(3)$$

Kim et al[31] (2007) developed a correlation for Korean soils between the static deformation modulus obtained in the PBT and the dynamic deflection modulus determined using the LFWD with an R-square value of 0.77 as given below:

$$E_{LFWD} = 1.41\,E_{PLT} - 7.48 \qquad ...(4)$$

4 EXPERIMENTAL SETUP

Laterite soil was first excavated from a site close to National Institute of Technology Karnataka, Surathkal and was labelled as B_1. The laterite soil excavated was then blended with silty-soil obtained from Kavur, a nearby locality in various proportions and labelled as B_2, B_3, and B_4. These blends were prepared with 25, 50 and 75 percent of silty-soil respectively. The original silty-soil with a high percentage of fines, obtained from Kavur, was labelled as B_5. It may be observed that the mixing of laterite soils with silty-soil was performed with the intention of altering the fines-content (referring to particles of size lesser than 75 microns).

The index properties of the laterite soils blended with various percentages of fines were first determined as provided in Table 1. Grain-size distributions for various blends are represented in Fig. 1.

Table 1 Index Properties of Blended Laterite Soils

Property	B_1	B_2	B_3	B_4	B_5
Laterite : Silty	100:0	75:25	50:50	25:75	0:100
OMC (%)	12.0	13.0	15.2	19.4	24.1
MDD (kg/cu.m)	1880	1800	1700	1540	1440
Specific Gravity	2.64	2.61	2.57	2.52	2.46
Gravel (%)	33.0	29.0	20.0	11.0	2.0
Sand (%)	57.0	44.0	32.0	19.0	6.0
Fines (%)	10.0	27.0	48.0	70.0	92.0
Voids Ratio	0.41	0.44	0.51	0.63	0.70
Liquid Limit (%)	56.0	58.0	59.0	60.0	60.0
Plastic Limit (%)	27.0	27.0	28.0	29.0	30.0

It was then planned to perform the PBT and tests using the PFWD (Inspector – II) of Englo make, CBR and DCP for un-soaked remolded soil samples for three different moisture-conditions, namely M_1, M_2 and M_3 in the laboratory at standard proctor density for various blends B_1, B_2, B_3, B_4 and B_5. These tests were aimed at generating simple correlations that could assist engineers in estimating the required design parameters for a wide range of soil types of the region varying from laterite to silty soils.

4.1 Impact of Variations in Percentages of Fines on OMC, MDD and Atterberg's Limits

The chart, as provided in Fig. 2, was obtained based on information compiled from Table 1. It was mentioned earlier that the blended laterite soil samples B_1, B_2, B_3, B_4 and B_5 have a resulting fines-content (referring to particles of size lesser than 75 microns) varying from 10 to 92 percent. The OMC of these remolded blended soil samples were found to increase from 12.08 percent to 24 percent. This is due to the reason that addition of silt to laterite soils increases the amount of smaller sized particles and consequently, the total surface area of the soil particles requiring more of moisture for attaining the MDD.

Also, it can be observed that the above increase in the percentage of fines from 10 to 92 percent has resulted in a corresponding decrease in the proportion of gravel and sand from 33 to 2 % and 57 to 6 %, respectively and has caused an increase in the voids ratio in the soil sample. This has consequently resulted in a decrease in the MDD from 1880 kg/m^3 to 1440 kg/m^3. Similar observations were made by Hicks and Monismith[17]. Omotosho[32] (2004) also provides similar observations based on studies conducted on laterite soils of Nigeria.

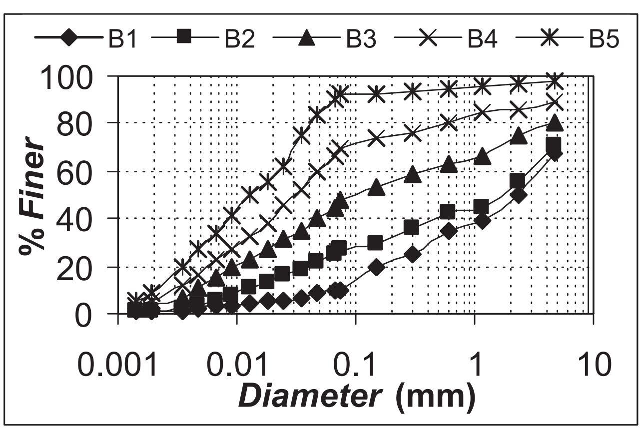

Fig.1 Grain-size Distribution for Various Blends

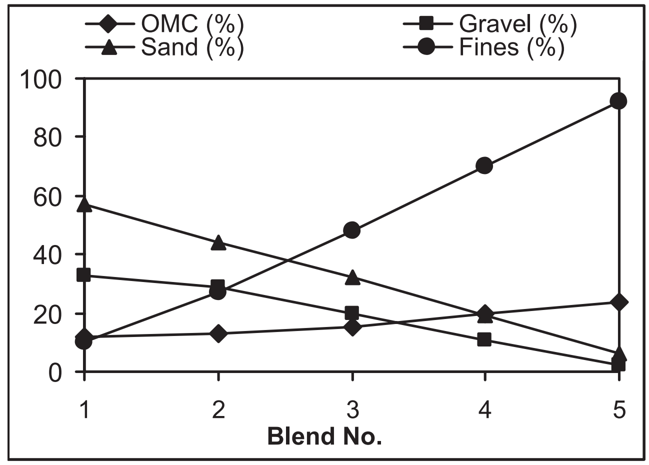

Fig. 2 Effect of Variations in Percentages of Fines, Gravel and Sand on OMC

5 LABORATORY SETUP FOR PFWD, PBT AND DCP

As part of these investigations, it was proposed to perform PFWD , Plate Bearing Test, DCP and CBR tests in the laboratory on un-soaked laterite soils blended with various percentages of fines. The tests were performed for standard proctor densities corresponding to three different moisture-conditions, namely M_1, M_2 and M_3 in the laboratory for various blends B_1, B_2, B_3, B_4 and B_5. Descriptions on the test setup for these devices are provided below.

5.1 Test Setup for PFWD Investigations

It may be noted that the influence depth of PFWD in the measurement of modulus values ranges between 280 mm and 380 mm depending upon the soil-strength[27]. The dimensions of the test box for tests on remoulded soil samples were arrived at based on these considerations.

Investigations were performed using a test box of 450 mm diameter and 450 mm height, made of 6 mm thick steel plates, provided with a base plate welded with 8 mm diameter mild-steel stiffeners at the bottom. An 8 mm mild steel hoop was welded at the top opening and bottom of the test-box to provide additional stability, stiffness and to prevent undue deformations on load-application as shown in Fig. 3.

The assembly was placed on a 20 mm thick sand layer while conducting PFWD such that the gaps between the bottom plate, the stiffeners and the surface of the test floor are closed, ensuring a perfect contact. The PFWD, fitted with a loading plate of 140 mm diameter, was then positioned on the soil sample and investigations were performed as shown in Fig.4(a) The results of these tests are tabulated in Table 2.

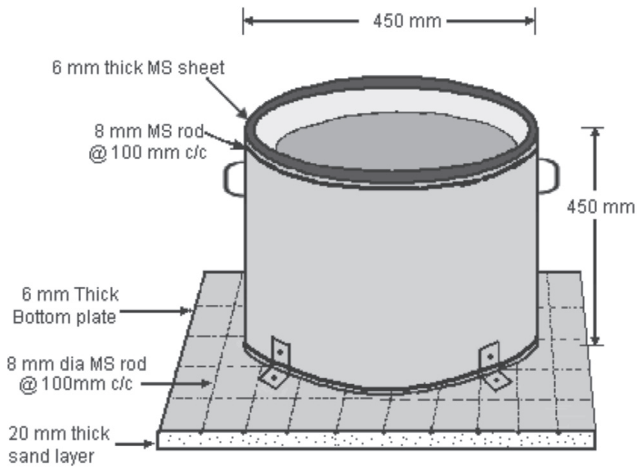

Fig. 3 Mild Steel Test Box for Unsoaked Samples (Not to scale)

5.2 Test Setup for Investigations using PBT

In the present study, PBT were performed on blended laterite soil samples prepared in the test-box fabricated for the same, as explained above. In order to perform tests on blended laterite soils at the standard proctor density for moisture-contents M_1, M_2 and M_3, the soil required to fill the cylindrical test-box of an inner diameter of 450 mm and a soil-depth of 350 mm was first calculated. One-fifth of the soil required was then noted and the soil was filled in five layers each of 70 mm thickness and compacted using a rammer to a total soil-depth of 350 mm. The diameter of the test box for the PBT satisfies the recommendations for the experimental set up and test procedure provided in the Indian Standard Code[34], where it is specified that the diameter of the loading plate should be approximately one-fifth of the diameter of the soil specimen in order to overcome the influence due to the confining of the boundary. These specifications are similar to those adopted for the Florida Method of Test for Static Plate Load Test[35]. In order to perform tests on blended laterite soils, the soil required to fill the cylindrical test-box of an inner diameter of 450 mm and a soil-depth of 350 mm was first calculated and compacted to the required thickness as mentioned in the above sections. The load was applied on the circular plate using a proving ring of 5t capacity, Fig. 4(b). Similar tests were performed for blends B_1 to B_5, for moisture contents of M_1, M_2 and M_3 for unsoaked. The results of these tests are provided in Table 2.

5.2.1 *Determination of Modulus of Elasticity for PBT*

AASHTO[35] provides information on the expression developed by Burmister[36] based on Boussinesq's theory for the determination of the modulus of deformation (E_{pbt}) for a soil sub-grade with Poisson's ratio μ for a deformation δ under a rigid plate of radius a and for a loading pressure p for static plate bearing tests as[37]:

$$E_{pbt} = 2\,pa\,(1-\mu^2) / \delta \quad ...(5a)$$

If the load is assumed to be applied by means of a rigid circular plate, it may be inferred that the stresses will not be distributed in a strictly uniform manner. Assuming a Poisson's ratio of $\mu = 0.5$, the expression for the theoretical displacement of the rigid plate, can be deduced as given below[37]:

$$E_{pbt} = 1.18\,pa / \delta \quad ...(5b)$$

5.3 Test Setup for Investigations Using e DCP

Tests using the DCP on blended laterite soils were conducted using the cylindrical test-box at the same density and moisture contents as was done in the case of tests using the PFWD and the PBT cited above. Fig. 5 provides a glimpse of the test setup for the DCP. The penetration test using the DCP was performed up to a total depth of 300 mm. The observations were then tabulated. The penetration-index measured using the DCP (DCPI) was obtained by dividing the total penetration in mm by the number of blows (mm/blow) and the same was reported for each sample. The results of DCP tests are provided in Table 2.

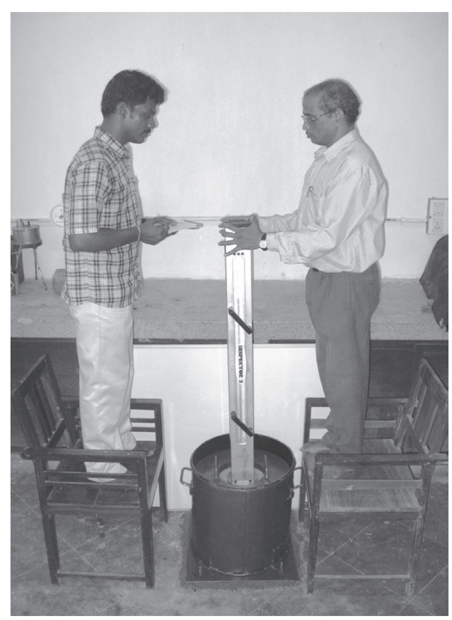

Fig.4(a) Test Box for Conducting PFWD Tests

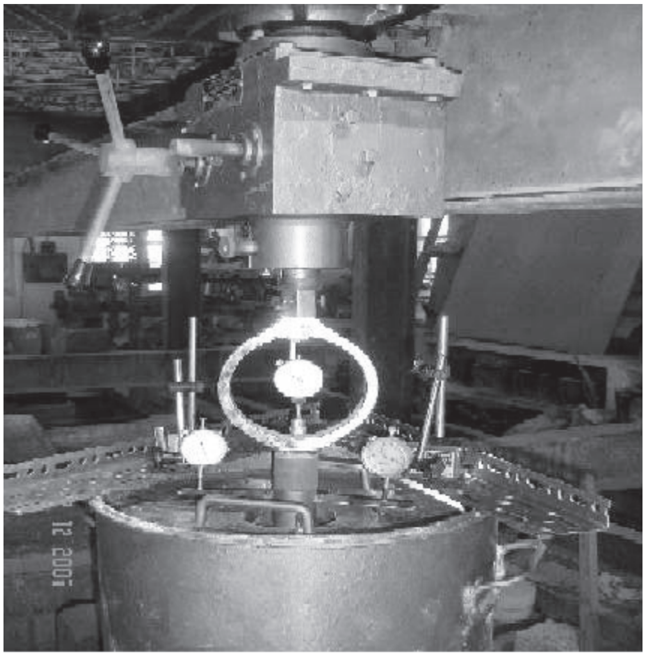

Fig. 4(b) Test Box for Conducting PBT

5.4 Test Setup for the CBR

In the present study, the CBR tests were performed on unsoaked blended laterite soil samples according to the Indian Standard test procedure[38]. CBR tests were conducted at the same density and moisture contents as was done in the case of tests using the PFWD, PBT and the DCP tests cited above. Three tests were performed for each blend and the average of three was reported as the CBR value. The results of CBR tests are summarized in Table 2.

Fig.5 Test Box for Conducting DCP Tests

Table 2 Results of PFWD, PBT, DCP and CBR Tests

Sample #	E_{pfwd} (MPa)	E_{pbt} (MPa)	DCPI (mm/ blow)	CBR (%)
B_1M_1	164.00	136.45	1.97	42.10
B_2M_1	160.00	122.80	2.37	37.00
B_3M_1	100.70	65.50	2.57	32.90
B_4M_1	48.30	28.66	4.94	20.00
B_5M_1	33.70	16.74	9.19	9.00
B_1M_2	151.70	109.00	2.30	32.80
B_2M_2	145.00	81.80	2.98	30.00
B_3M_2	71.00	46.78	4.90	16.60
B_4M_2	41.00	27.29	7.60	10.00
B_5M_2	28.00	9.44	11.00	6.50
B_1M_3	127.30	89.30	3.46	23.00
B_2M_3	125.70	81.80	4.29	19.30
B_3M_3	48.70	32.74	8.26	9.20
B_4M_3	37.00	16.43	8.25	6.43
B_5M_3	27.00	8.93	14.20	5.50

6 DEVELOPMENT OF SIMPLE REGRESSIONS BETWEEN RESULTS OF VARIOUS TESTS

This section focuses on exploring relationships connecting the observations made using the PFWD, PBT, DCP and CBR using the Statistical Package for the Social Sciences. These relationships will assist road engineers in estimating the required indices for various pavement design and pavement evaluation approaches. Thus, 15 samples were tested using each of the equipment. Details on observations of the values of the E_{pfwd}, E_{pbt}, DCPI and the CBR are provided in Table 2 for various blends and moisture contents.

7 DEVELOPMENT OF SIMPLE REGRESSIONS BETWEEN RESULTS OF VARIOUS TESTS

This section focuses on exploring relationships connecting the observations made using the PFWD, PBT, DCP and CBR using the Statistical Package for the Social Sciences and Microsoft Excel. These relationships will assist road engineers in estimating the required indices for various pavement design and pavement evaluation approaches. Thus, 15 samples were tested using each of the equipment. Details on observations of the values of the E_{pfwd}, E_{pbt}, DCPI and the CBR are provided in Table 2 for various blends and moisture contents.

7.1 Regressions Between E_{pfwd} and E_{pbt} Observations

Based on information compiled in Table 2, the following regression was developed between the modulus of resilience measured using the PFWD (E_{pfwd}) and the modulus of deformation determined using the plate bearing tests (E_{pbt}). Keeping aside 5 rows of data pertaining to samples designated as B_1M_1, B_2M_2, B_3M_3, B_4M_1 and B_5M_2 for validation, the data in the remaining 10 rows of the table were used for the development of the regression equation relating E_{pfwd} and E_{pbt} as shown in Eq. 6. The scatter plot and the best-fit curve relating them for the 10 rows of data selected are provided in Fig. 6. Thus, E_{pfwd} can be estimated based on the values of E_{pbt}. The R^2 value for the regression was 0.98 and the adjusted R^2 value was 0.97. The standard error of estimation (SE) was found to be 5.402, while the values for the F-test and t-test were 811.1 and 4.71,

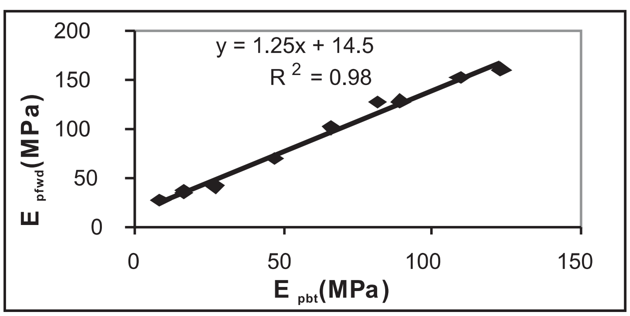

Fig. 6 Correlation between E_{pfwd} and E_{pbt} Values

respectively at a confidence level of 99%.

$$E_{pfwd} = 1.25(E_{pbt}) + 14.5 \quad ...(6)$$

Using the values of E_{pbt} in the remaining 5 rows of data pertaining to samples designated as B_1M_1, B_2M_2, B_3M_3, B_4M_1 and B_5M_2 the regression developed vide Eq.6 were validated. The respective scatter plot, Fig. 7 indicates that the predicted values of E_{pfwd} versus the actual values of the same agree with each other. It can be further observed that the regression line fits very well with an R-square value of 0.92, with a negligible intercept of 5.52. The points in the scatter plot coincide satisfactorily with the 'theoretical line of equality' that passes through the diagonals.

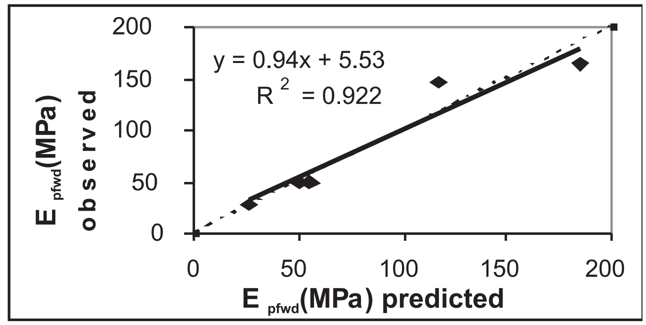

Fig. 7 Plot of E_{pfwd} observed vs. E_{pfwd} predicted from E_{pbt} values

The form of the regression equation relating E_{pfwd} and E_{pbt} developed as in Eq. 6 above for blended laterite soils, follows a similar pattern as observed in the correlations developed for other local soils, by Seyman[29], Nazzal et al[30] and Kim et al[31] as expressed vide Eq. 2, Eq. 3 and Eq. 4. A graphical comparison between the results of the present study and the investigations cited above is provided in Fig. 8. The trend indicates that the present study on blended laterite soils follow a linear trend similar to that followed by Nazzal et al[30], with

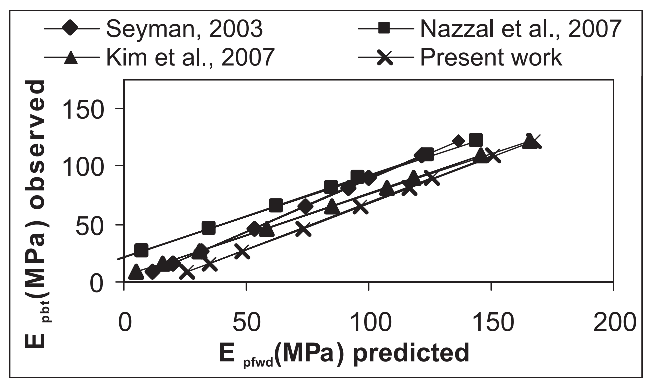

Fig. 8 Comparison of E_{pfwd} and E_{pbt} of the present study to other studies

differences in the slope and the intercept for blended laterite soils tested.

7.2 Regression Between E_{pfwd} and DCPI Observations

In a similar manner, a regression between the modulus of resilience measured using the PFWD (E_{pfwd}) and the penetration index determined using the DCP (DCPI) values can be developed as shown in Eq. 7 and the scatter plot for the same is given in Fig. 9. Thus, E_{pfwd} can be estimated based on the values of DCPI. The R^2 value for the regression was 0.91 and the adjusted R^2 value was 0.90. The standard error of estimation was found to be 0.22, while the values for the F-test and t-test were 79.45 and 5.07, respectively at a confidence level of 99%. Eq. 7 was validated using the same approach

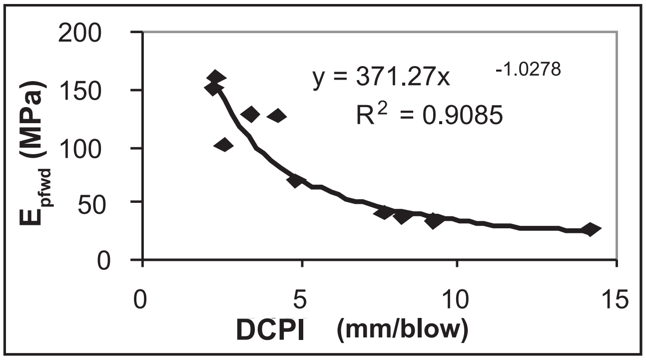

Fig. 9 Correlation between E_{pfwd} and DCPI Values

as mentioned above.

E_{pfwd} =371.0(DCPI) $^{-1.03}$ ...(7)

7.3 Regression Between E_{pbt} and DCPI Observations

A regression between the modulus of resilience measured using the PBT (E_{pbt}) and the penetration index determined using the DCP (DCPI) values is provided as shown in Eq. 8 and the scatter plot for the same is given in Fig. 10. Thus, E_{pbt} can be estimated based on the values of DCPI. The R^2 value for the regression was 0.92 and the adjusted R^2 value was 0.91. The standard error of estimation was found to be 0.28, while the values for the F-test and t-test were 90.70 and 4.03, respectively at a confidence level of 99% . Eq. 8 was validated using the same approach as mentioned above.

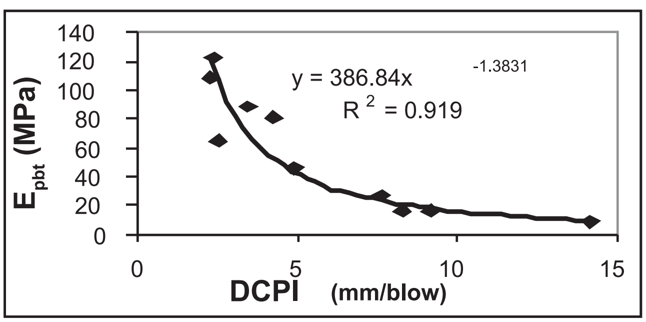

Fig. 10 Correlation between E_{pbt} and DCPI values

E_{pbt} =386.8(DCPI) $^{-1.38}$ ...(8)

7.4 Regression Between CBR and E_{pfwd} Observations

A regression between the CBR and the modulus of resilience measured using the PFWD (E_{pfwd}) values is developed as shown in Eq. 9 and the scatter plot for the same is given in Fig. 11. Thus, CBR can be estimated based on the values of E_{pfwd}. The R^2 value for the regression was 0.83 and the adjusted R^2 value was 0.81. The standard error of estimation was found to be 5.20, while the values for the F-test and t-test were 38.03 and 0.33, respectively at a confidence level of 99%. Eq.9 was validated using the same approach as mentioned above.

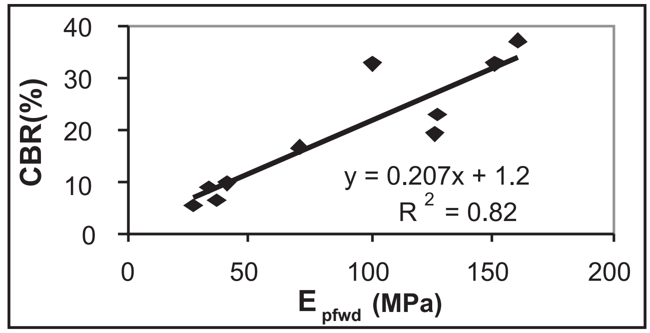

Fig. 11 Correlation between CBR and E_{pfwd} Values

CBR = 0.207 (E_{pfwd}) + 1.2 ...(9)

The form of the regression equation for blended laterite soils relating to CBR and E_{pfwd} developed as in Eq. 9 above is similar to the correlation developed by Nazzal[27] as expressed in Eq.1. A graphical comparison between the results of the present study and the investigations mentioned above is provided in Fig. 12. The graphical trend indicates that the present study on blended laterite soils follow a linear trend similar to that followed by Nazzal[27], with major differences in the slope and the intercept.

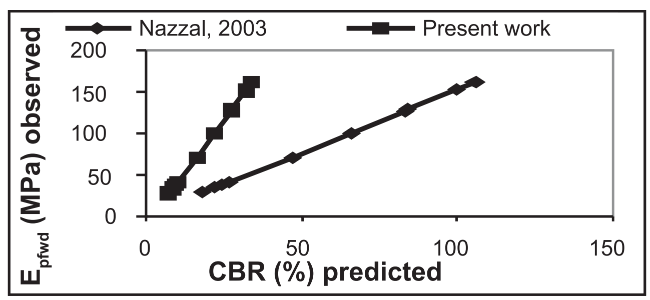

Fig. 12 Comparison of CBR and E_{pfwd} of present work to studies by Nazzal[27]

7.5 Regression Between CBR and DCPI Observations

A regression between the CBR and the penetration resistance measured using the DCP (DCPI) values was developed as shown in Eq. 10 and the scatter plot for the same is given in Fig. 13. Thus, CBR can be estimated based on the values of DCPI. The R^2 value for the regression was 0.97 and the adjusted R^2 value was 0.96. The standard error of estimation was found to be 0.14 while the values for the F-test and t-test were

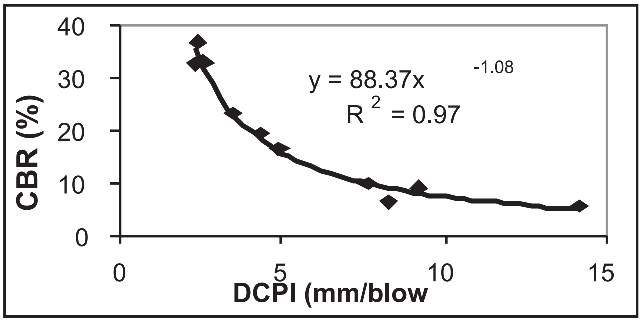

Fig. 13 Correlation between CBR and DCPI values

221.05 and 8.05, respectively at a confidence level of 99%. Eq. 10 was validated using the same approach as mentioned above.

$$CBR = 88.37(DCPI)^{-1.08} \quad ...(10)$$

7.6 Regression Between CBR and E_{pbt} Observations

A regression between the CBR and the modulus of resilience measured using the PBT (E_{pbt}) values was developed as shown in Eq. 11 and the scatter plot for the same is given in Fig. 14. Thus, CBR can be estimated based on the values of E_{pbt}. The R^2 value for the regression was 0.84 and the adjusted R^2 value was 0.83. The standard error of estimation was found to be 5.02, while the values for the F-test and t-test were

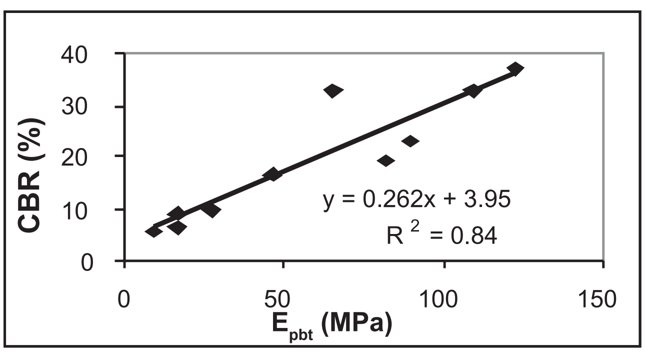

Fig. 14 Correlation between CBR and E_{pbt} values

41.63 and 1.38, respectively at a confidence level of 99%. Eq. 11 was validated using the same approach as mentioned above.

$$CBR = 0.262(E_{pbt}) + 3.95 \quad ...(11)$$

8 CONCLUSIONS

The above studies were aimed at exploring relationships connecting the observations made using various equipments- PBT, PFWD, CBR and DCP. The relationships developed are expected to be of special advantage in pavement design and evaluation of lateritic regions. The following conclusions were drawn based on the discussions on the results:

8.1 A close examination of the index properties of blended laterite soils investigated above reveals that an increase in the effective percentage of fines from 10 to 92 percent due to the blending of laterite soils has resulted in a corresponding increase in the OMC. This is mainly due to the increase in the total surface-area of the soil particles as the proportion of fines increase.

Also, the increase in the proportion of fines and the accompanied decrease in the percentage of gravel and sand for the blended laterite soil samples have resulted in an increase in the voids ratio. This has consequently decreased the MDD and the average specific gravity of the soil particles.

8.2 In the observations made in Table 2 for tests conducted using the PFWD, the PBT, the CBR and the DCP, it was found that the modulus of resilience measured using the PFWD, modulus of deformation measured using the PBT and the CBR values showed a decreasing trend as the moisture content increased from M_1 to M_3, while the penetration indices measured using the DCP showed an increasing trend indicating a decrease in the soil strength and stiffness.

8.3 For blended laterite soils investigated, the modulus of resilience measured using the PFWD and the modulus of deformation measured using the PBT are highly correlated following a linear form as in Eq. 6, in a manner similar to correlations reported by other researchers with differences in the slope and the intercept, Fig. 8.

8.4 The modulus of resilience measured using the PFWD and the penetration indices measured using the DCP are highly correlated following a power function as in Eq. 7.

In addition to the above, the correlation between the modulus of resilience measured using the PFWD and the CBR values determined are highly correlated following a linear form as in Eq. 9, in a manner slightly different from correlations reported by other researchers with major differences in the slope and the intercept, Fig. 12.

8.5 In this study, the CBR values and the penetration indices measured using the DCP are highly correlated following a power function as in Eq. 10.

8.6 Additionally, the modulus of deformation measured using the PBT and the penetration indices measured using the DCP are highly correlated and follow a power function as in Eq. 8, while the modulus of deformation and the modulus of sub-grade reaction measured using the PBT, are highly correlated to the CBR values, following linear relationships as in Eq.11 and Eq.12.

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