Comparative Study of Sub grade Soil Strength Estimation Models Developed Based on CBR, DCP and FWD Test Results

Dr. R. Srinivasa Kumar[1], Alhammadi Shabbab Ajmi[2], Bhasker Valkati[3]

Assistant Professor, Civil Engineering Department, Osmania University, Hyderabad, India[1]

Associate Professor and Dean, Civil Engineering Department, Shaqra University, Dawadmi, KSA[2]

Lecturer, Civil Engineering Department, Shaqra University, Dawadmi, KSA[3]

Abstract: Sub grade soil strength is an essential input for design of pavements and overlays. Many models are available for determination of subgrade soil strength. Subgrade soil strength in terms of elastic modulus value is used in Mechanistic-Empirical methods and CBR value is used in empirical methods of design. Both the parameters can be used for structural evaluation of flexible pavement. Determination of elastic moduli values of pavement layers by back-calculation of deflections measured using Falling Weight Deflectometer (FWD) is a widely used scientific method. However, extensive use of FWD for large scale evaluation of in-service pavements is not feasible, especially for low volume roads, unimportant roads and roads in developing and under developed countries. In the present investigation, an attempt has been made to develop relationships that can be used to estimate subgrade soil strength in terms of elastic modulus and CBR values using in-situ test results of Dynamic Cone Penetrometer (DCP). As DCP is a portable and low-cost equipment used for quick evaluation of unbound granular and subgrade soil. Comparative studies are also carried out with similar models available from the literature review.

Keywords: Falling weight Deflectometer (FWD), Back-calculated subgrade modulus, California Bearing Ratio (CBR), Dynamic Cone Penetrometer (DCP) and Regression Models.

I. INTRODUCTION

Recent trend of flexible pavement design is based on mechanistic and empirical (M-E) methods. The M-E methods use elastic moduli values and the empirical methods use CBR values of subgrade soil. The task of selecting appropriate values of subgrade strength for the analysis of new as well as in-service pavements has been drawing the attention of pavement researchers for a number of years.

Assessment of elastic moduli of granular and subgrade soil by using Falling Weight Deflectometer (FWD) or laboratory testing are very popular methods but they could be costly procedures for evaluation of unimportant or low traffic volume roads, in developing and under developed countries. Due to this reason, Dynamic Cone Penetrometer (DCP) test has been extensively used by several countries around the globe for quick determination of an index value which represent the strength of compacted unbound granular and soil layers with reasonable accuracy at low cost.

In the present investigation, an effort has been made to develop regression models by correlating the results obtained from the evaluation of the subgrade soil of pavements using test results of CBR, Dynamic Cone Penetrometer (DCP) and Falling Weight Deflectometer (FWD).

II. HISTORICAL DEVELOPMENT OF DCP

The first Dynamic Cone Penetrometer (DCP), which is similar the one available now, known as Scala Penetrometer was developed by Scala (1956) in Australia. It was used for determination of in-situ CBR of cohesive subgrade soils. The Scala Penetrometer consists of about 9 kg hammer which drops from 510 mm height through a vertical guide rod. The hammer impact energy is ultimately applied on to a cone (having apex angle 30°) fitted at bottom end of the guide rod.

Later, a similar tool like DCP was developed by van Vuuren (1969). It consists of a 10 kg hammer which drops freely from 460 mm height. The impulse force is applied on to a cone having 30°. He has developed a relationship between the DCP test results and CBR values of subgrade soil.

For rapid evaluation of flexible pavements a DCP was used by the Transvaal Roads Department of South Africa in 1973 (Kleyn 1975). The DCP consisted of 8 kg hammer which drops from height of 574 mm. The cone's apex angles considered were 30° and 60° and a comparative study was carried out between the DCP test results (Kleyn et al 1982). Another report (Kleyn and Savage, 1982) reveals that, several investigations were made on subgrade tested with DCP using 8 kg hammer dropped from height of 574 mm and with the cone's apex angle as 60°.

III. WORKING PRINCIPLE OF DCP

The salient features of the widely used manually operated DCP are presented in Figure 1. The resistance offered by the compacted granular/soil layer to penetration of a standard cone having 20 mm diameter and apex angle 60°

driven by a 8 kg hammer dropped from a height of 575 mm, is recorded as average penetration per blow (IRC: SP:72 2007).

The average penetration of the cone per blow is reported in as an index value and it may be represented in many forms viz. DCP index (DCPI), DCP, penetration rate (PR), penetration index (PI), DCP_θ^o; or Number of blows required to penetrate a given thickness of layer as (DCPN), N_{DCP}, Blow Rate (BR) and DCP Structural Number (DSN). In this paper, the cone's average penetration per blow (in mm/blow) is denoted as DCP_θ^o, where, θ^o denotes apex angle of the cone.

During the cone penetration, the material shear strength is mainly accrued due to resistance offered by the shear displacement of soil particles. A smaller value of DCP_{60}^o indicates harder material and vice versa.

A. Testing with DCP

To begin with the testing procedure, initial reading on the DCP scale should be noted while holding the rod vertical and the cone in contact with surface of compacted soil to be tested. While one person holds the DCP handle, another person should lift the hammer to the predetermined height (i.e. 575 mm) and allow it to drop freely on to the anvil. Cumulative penetration values of the cone should be noted for each blow. Generally, the DCP testing should be stopped when the cone penetration is not more than 1 mm per blow since the cone may encounter a rock, gravel larger than 20 mm size or hard strata. After the DCP test is over, the cone-rod should be extracted out and the soil sample may be collected for conducting other tests in laboratory.

An extension rod of additional length of 1000 mm may be used in place of the standard penetrating rod where it is necessary to test the deep soil (Livneh and Livneh, 1994).

A study was conducted by Livneh (1991) for determining the change in rate of penetration due to change in the cone apex angle from 30° to 60° and suggested the following relationship to convert DCP index value (in mm/blow) in terms of the other.

$$DCP_{30^o} = 0.006 + 1.092\ DCP_{60^o} \qquad (1)$$

The DCP test can also be conducted in a laboratory on remolded material compacted in a steel mould which significantly eliminates the effect of confinement and such test can also be carried out as a prototype model (Ayers et al, 1989).

VI. REVIEW OF LITERATURE

In different parts of the world, several researchers (De Villiers 1980; Livneh and Ishai 1987; Pen 1990; Chai and Roslie 1998; Chen and Latorella 1999; Livneh et al 2000; Roy 2007) have estimated strength properties of granular and subgrade soils by correlating DCP test results with standard test parameters. Such relationships can be used to convert the DCP_θ^o value to different strength parameters such as CBR, elastic modulus and other standard test values. The following paragraphs briefly present these relationships.

A. Relationships between DCP_θ^o and CBR Values

Several relationships are available to convert values of DCP_θ^o to CBR (Table 1). Such varieties of relationships do exist since large strain penetration takes place during testing of compacted soil by both the tests. A few salient details of different investigations made for development of the relationships (Table 1) are given below.

■ Development of Scala Penetrometer (1956) for estimation of in-situ CBR of cohesive soils has led to development of the present version of DCP.

■ Scala (1956) and Kleyn (1975) initially identified straight line relationship between the DCP_θ^o and CBR values plotted on log-log chart.

■ Laboratory CBR values were used for development of the US Corps of Engineer's relationship (Webster et al 1992). Many DCP_θ^o to CBR relationships developed by different researchers around the globe were considered in this study and they were found to be in close agreement with the relationship developed by Webster et al (1992). Therefore, this relationship has been widely used by several researchers (Livneh 1995; Siekmeier et al 2000 and Chen et al 2001).

■ Ese et al (1994), extensively evaluated 23 granular base courses in Norway and correlated laboratory CBR and DCP_θ^o values. They reported that, (i) difference in confining pressure in CBR mould and prevailing in-situ condition was accounted in development of the relationship and (ii) suggested that, a critical stability value of 2.6 mm/blow may be taken for gravel base.

■ Nazzal (2003), conducted many laboratory CBR and field DCP tests on compacted granular materials, clay and soils stabilized. During the laboratory CBR testing, the moisture content and density were maintained similar to in-situ condition. He reported that, by using the equation (No.15), the estimated CBR values were well matching with the values obtained by equation (No. 10) (Webster et al 1992), when the DCP_{60}^o value is greater than 20 mm/blow.

B. Relationships between DCP_θ^o and FWD Back-calculated Subgrade Modulus Values

Elastic moduli of pavement layers are essential inputs required in the mechanistic design of new pavements and for the estimation of the remaining life of in-service pavements. Towards this aim, selection of appropriate values of base and subgrade layer moduli for the analysis of new as well as in-service pavements is most critical issue concerning the pavement/soil researchers. To achieve this objective, several researchers in different countries have developed the following relationships between back-calculated subgrade moduli and DCP index values. Such relationships can be re-used where routine evaluation of pavement by FWD is not feasible for low traffic volume roads.

■ Pen (1990) developed the following equations based on field DCP values.

$$\log(E_s) = 3.250 - 0.89\log(DCP_{60^o}) \quad 17)$$

where, E_s is the subgrade elastic modulus (in MPa) obtained from the deflections measured using PHONIX FWD.

$$\log(E_s) = 3.653 - 1.17\log(DCP_{60^0}) \quad (18)$$

Where, E_s is the subgrade elastic (in MPa) backcalculated by PEACH system.

- De Beer (1991) has prescribed the following equation.
Subgrade Elastic Modulus

$$(MPa) = 1176 \times (DCP_{60^o})^{-1.082} \quad (19)$$

Where, DCP value is in mm/blow.

- The following equations were prescribed by Chai and Roslie (1998):
Relationship between in-situ CBR and

$$DCP: E_s = 17.6(269/DCPV)^{0.64} \quad (20)$$

Based on back-calculated results:

$$E_s = 2224 \ (DCPV)^{-0.996} \quad (21)$$

Where, Es = Subgrade modulus value (in MPa) and DCPV = blows/300mm penetration of cone having apex angle 60°.

- Chen et al (1999) conducted FWD, CBR and DCP tests on six sections of pavements in Kansas. Based on FWD back-calculated subgrade moduli (E_s) and DCP test data, the following expression was prescribed.

Subgrade Resilient Modulus

$$(E_s \ in \ MPa) = 338 \times (DCP_{60^o})^{-0.3} \quad (22)$$

Where, DCP_{60^o} value was in the range of 60 to 10 mm/blow.

Chen et al (1999) found that the subgrade moduli determined from relationships between DCP to CBR and CBR to E_s were widely varying along the length of the test sections.

- George and Uddin (2000) developed the following equations (Cited in Roy 2007).
For fine grained soils:

$$\log(E_s \ in \ MPa) = 2.726 - 0.492\log(DCP_{60^0}) \quad (25)$$

For coarse grained soils:

$$\log(E_s \ in \ MPa) = 2.372 - 0.475\log(DCP_{60^0}) \quad (26)$$

- Chen et al (2005) prescribed the following equation (After eliminating outlier data).

$$E_s = 537.76 \times (DCP_{60^o})^{-0.6645} \quad (28)$$

Where, E_s = Granular Layer Resilient Modulus (MPa)

- As an indirect approach, Siekmeier et al (2000) and Chen et al (2001, 2005), converted the DCP index value to CBR value by using the equation developed by Webster et al (1992) and the obtained CBR value is converted into the resilient modulus (Es) value using the following equation of TRL-UK (Lister and Powell 1987).

Subgrade resileint modulus

$$(E_s \ in \ MPa) = 17.6 \times CBR^{0.64} \quad (29)$$

Chen et al (2001) conducted a series of FWD and DCP tests on base and subgrade layers of two pavement sections in Texas. The test results were found to be satisfactorily comparable. However, they found that the DCP index values can be affected by at least 10 percent due to variations in the test procedure.

- Several attempts were also made in India (Sood et al 1996, Srinivasa et al 2003, 2006), USA (Mi-DOT) and other parts of the world (Livneh et al 2000) for developing equations to estimate of modulus value of granular and subgrade soil using DCP index value and material parameters such as field density, density ratio, liquid limit, plasticity index, coefficient of uniformity, field moisture content, ratio between field to OMC and percent soil passing different sizes of sieves.

In addition to this, there are several relationships available for determining (i) AASHTO granular layer coefficients (cited in Roy 2007), (ii) unconfined compressive strength (UCS) of soil (De Villiers 1980; McElvaney and Djatnika 1991), using DCP test values of granular base and sub-base and (iii) shear strength of cohesion less materials using relationships available between the laboratory determined values of DCP_{60^o} and triaxial deviator stress at failure (Ayers et al 1989).

V. STRUCTURAL EVALUATION OF PAVEMENTS

As a part of a research project in India, Falling Weight Deflectometer (FWD) and Dynamic Cone Penetrometer (DCP) tests were conducted on different sections of National Highway (NH) 6, National Highway (NH) 60, Salua Road and IIT-perimeter Road. The above tests were conducted during post monsoon season because during this period the subgrade becomes soft or less stiff due to accumulation of rain and stagnated water.

NH 6 and NH 60 are two-lane highways. On NH 6, six different pavement stretches of about 300 m length each were selected and the type of subgrade soils ranged from clay to silt. The types of subgrade soil encountered at NH 60 are of clay to silty clay. Traffic volumes on Salua road and IIT-perimeter road are lower. Soil sample collected from one test pit excavated in Salua road indicated the subgrade soil as clay type. Along the IIT-perimeter road, the soil type varied from clay to silt.

A. Deflection Measurements Using Falling Weight Deflectometer

Pavement deflection measurements on selected test sections were taken using FWD (Srinivasa 2001). An impulse load of 40kN in magnitude was applied over a contact area of radius 150 mm. The FWD loading time ranges between 25 to 30 milli-seconds as usually. A load cell was used to measure the impulse load. Six geophones/deflection sensors were used to measure surface deflection basin and the sensors were placed radially at 300 mm intervals from the center of the loading plate.

B. Analysis of FWD Data

Effective layer moduli were back calculated using a Genetic Algorithm based back-calculation computer program, BACKGA (Murthy 1999). A linear elastic layered system, ELAYER program (Reddy 1993), was used for forward calculation routine of the BACKGA program. All the FWD tested pavement sections were modeled as 3 layer system.

C. DCP and CBR Tests

DCP tests were conducted on subgrade soil of the pavement sections tested with FWD. Soil samples were collected from some of the sections and were tested in laboratory for determination of their CBR values and soil classification.

The details of subgrade soil classification, DCP, FWD and CBR test results are given in Table 2-5.

D. Development of Regression Models

Regression analysis was carried out on the data collected in the present investigation and trials were made to develop different forms of relationships between the values of (a) CBR and DCP_{60}^{o} and (b) back-calculated subgrade modulus (E_s) and DCP_{60}^{o}. From the trials, the following models were found to be the most suitable.

Between CBR and DCP_{60}^{o}:

$$\log(CBR_s \text{ in per cent}) =$$
$$2.71545 - 1.3152\log(DCP_{60^0}) \qquad (29)$$

$$(N = 32, R^2 = 0.99)$$

Between E_s and DCP_{60}:

$$\log(E_s \text{ in } MPa) = 2.553 - 0.6438\log(DCP_{60^0}) \qquad (30)$$

$$(N = 96, R^2 = 0.71)$$

VI. COMPARATIVE STUDY

Comparative studies were conducted using the relationships developed in the present study with similar relationships available from the literature review. The Comparative study considered two types of relationships between (i) CBR and DCP_{60}^{o} and (ii) E_s and DCP_{60}^{o}.

(i) On relationships between CBR and DCP_{60}^{o}

Relationships between DCP_{60}^{o} and CBR values are presented in Figure 3. It is noted that, all the lines of the relationships are relatively located closer within a narrow bandwidth except the relationships prescribed by McElvaney et al (1985) and Nazzal (2003). Such disparities among relationships which were developed in different countries may be due to slight variations in the test procedure, differences in the soil moisture content, compacted density, surrounding confinement and type of soil. It is also noted that, all above equations are acceptable to the in-situ conditions prevailing and based on the adopted test conditions.

Further comparison has been made on the relationships of fine grained soils and other which were developed by Harison (1989), TRL-UK (1990), Webster et al (1992) and the Author. It can be observed from the Figure 4 that, (a) the relationships (by Eq. 6 and 7) developed by Harison

are showing almost same slope (or trend) for the soil strength considered (b) The relationships of TRL-UK, Harrison and by the present study are closely matching with each other except with small differences (c) at the lower values of CBR, all the relationships are within a close bandwidth except the relationships (by Eq. 12 and 13) developed by Webster et al (1992) which were specifically prescribed for soil types of CL (where CBR <10 per cent) and CH. This indicates that, the DCP_{60}^{o} values are sensitive at smaller CBR values and the subgrade consists of highly plastic clay.

(ii) On relationships between E_s and DCP:

Relationships between back-calculated subgrade modulus (E_s) and DCP_{60}^{o} are presented in Figure 5. Larger variations are found among these relationships. A line drawn based on indirect method using relationships of Webster et al (1992) (by Eq 10) and TRL-UK (Lister and Powell 1987) (by Eq 28) is found to be located in the middle of the lines. Such scatter and disparities are not abnormal because, in different countries the subgrade soil properties may vary with many parameters such as variations in moisture content, intensity of compaction took place during construction and traffic loading, surrounding confinement and drainage conditions. It is also noted that, all above equations are acceptable which are derived based on the test conditions and prevailing material properties.

V. CONCLUDING REMARKS

The task of selection of appropriate values of subgrade strength for the analysis of new as well as in-service pavements has been engaging the attention of pavement researchers for a number of years. Towards this objective, the DCP as a low cost and alternative tool used for evaluating unbound granular and subgrade soil of low volume and roads in developing and under-developed countries where routine evaluation of FWD is not feasible. The relationships available are useful for estimation of soil strength properties used for design and structural evaluation of unbound granular/soil layers of pavement. The disparities among the different relationships developed for different countries may be due to variations in prevailing soil type, drainage condition, layers confinement, depth of testing and variations in test procedures adopted. However, the available relationships are acceptable which were derived based on test procedure adopted and prevailing soil properties.

VI. ACKNOWLEDGEMENT

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TABLE 1-Relationships between DCP_{0^o} and CBR Values (from Srinivasa, 2009).

Reference/Developed/Source	Equation [b]	Equation Number
Kleyn and Van Heerden 1983	$Log_{10}CBR = 2.632 - 1.28\ Log_{10}(DCP_{60^o})$	2
Smith and Pratt 1983 (Developed for ARRB)	$Log_{10}CBR = 2.555 - 1.145\ Log_{10}(DCP_{60^o})$	3
McElvaney et al 1985	$Log_{10}CBR = 2.81 - 1.32\ Log_{10}(DCP_{60^o})$	4
Livneh and Ishai 1987	$Log_{10}CBR^F = 2.2 - 0.71\ (Log_{10}(DCP_{30^o}))^{1.5}$	5
Harison (1989)	For $DCP_{60^o} \geq 10$ mm/blow: $Log_{10}CBR = 2.56 - 1.16\ Log_{10}(DCP_{60^o})$	6
	Or For $DCP_{60^o} < 10$ mm/blow: $Log_{10}CBR = 2.54 - 1.12\ Log_{10}(DCP_{60^o})$	7
TRL, Road Note 8 1990	$Log_{10}CBR = 2.48 - 1.057\ Log_{10}(DCP_{60^o})$	8
Livneh, 1991 (Was obtained by substituting Eq.1 in Eq. 5)	$Log_{10}CBR = 2.2 - 0.71[Log_{10}(1.1 \times DCP_{60^o})]^{1.5}$	9
Webster et al, 1992; The Relationship developed for the US Corps of Engineers	Overall correlative relationship: $Log_{10}CBR = 2.465 - 1.12\ Log_{10}(DCP_{60^o})$	10
Ese et al 1994	$Log_{10}CBR = 2.438 - 1.065\ Log_{10}(DCP_{60^o})$	11
Webster et al 1994	For CH soil: $Log_{10}CBR = 2.542 - 1.0\ Log_{10}(DCP_{60^o})$	12
	For CL soil having CBR<10%: $Log_{10}CBR = 3.538 - 2.0\ Log_{10}(DCP_{60^o})$	13
Livneh et al 1995	$Log_{10}CBR^F = 2.14 - 0.69\ (Log_{10}(DCP_{60^o}))^{1.5}$	14
Nazzal (2003)	$CBR = 1.04 + 2559.44/(-7.35 + DCP_{60^o}^{1.84})$ [c]	15
Coonse (1999) at North Carolina State University (Cited in Roy 2007)	$Log_{10}CBR^f = 2.53 - 1.14\ Log_{10}(DCP_{60^o})$	16

[b] DCP_{30^o} and DCP_{60^o} are the penetration values in mm/blow where the cone apex angles are 30^0 and 60^0 respectively; CBR is the California Bearing Ratio (%); CBR^F indicates field CBR; and CBR^f indicates the field CBR value of cohesive residual soil.

[c] the prescribed range of $DCP_{60°}$ value is 6.31 to 66.67 mm/blow.

TABLE 2-Test Results of IIT- Perimeter Road

Test location at KM	DCP (mm/blow)	E_s (MPa)	CBR (%)	Soil Classification
4.000	36.20	42.50	4.60	A-2-6
3.985	37.50	43.60		
3.970	39.60	37.00		
3.955	40.10	35.70	4.00	A-2-6
3.940	38.80	37.50		
3.925	30.10	43.30		
3.910	33.40	41.30		
3.710	33.20	43.70	5.10	A-2-7
3.695	25.40	49.10	7.70	A-2-5
3.680	30.20	46.90	5.80	A-2-5
3.665	27.50	44.00		
3.650	28.50	44.40	6.30	A-2-5
3.400	14.00	69.40		
3.385	17.20	61.90		
3.370	13.20	72.00	17.40	A-2-4
3.355	12.10	68.60		
3.340	11.00	72.80		

TABLE 3 -Test Results of Salua road.

Test location at KM	DCP (mm/blow)	E_s (MPa)	CBR (%)	Soil Classification
2.000	28.86	35.10	6.14	A-2-6
1.985	26.48	36.50		
1.970	26.52	45.50		
1.955	27.13	48.40		
1.940	29.50	39.90		
1.925	29.20	42.60	5.99	A-2-6
1.910	28.47	39.40		
1.895	31.47	39.10		
1.880	33.42	34.10		
1.865	32.86	38.30		
1.850	32.74	39.30		
1.835	32.62	39.20		
1.820	32.31	39.10		
1.805	32.65	35.20		
1.790	31.25	36.70		
1.775	32.41	36.10		
1.760	29.10	41.30		
1.745	29.36	37.40		

TABLE 4 -Test Results of National Highway 60.

Test location at KM	DCP (mm/blow)	E_s (MPa)	CBR (%)	Soil Classification
113.000	27.58	34.60		
112.985	26.13	41.00		
112.970	24.30	42.10		
112.955	25.21	48.50		
112.940	25.45	42.80		
112.925	28.50	38.90		
112.910	28.80	37.00		
112.895	29.10	37.60		
112.880	28.50	35.80		
112.780	33.20	31.20	4.85	A-2-6
112.765	29.60	35.90		
112.750	33.30	30.20		
112.735	30.60	33.70		

TABLE 5-Test Results of National Highway 6.

Test location at KM	DCP (mm/blow)	E_s (MPa)	CBR (%)	Soil Classification
123.000	27.50	47.00	7.00	A-2-6
122.995	18.63	62.70		
122.950	30.50	34.30		
122.945	19.60	62.00	10.15	A-2-7
122.845	20.80	54.80		
122.795	21.20	50.20		
122.745	30.00	41.70	5.85	A-2-6
124.000	20.02	54.30	10.00	A-2-6
123.900	25.20	36.10		
123.850	25.00	43.50		
123.845	25.30	36.20	7.95	A-2-6
123.795	31.52	36.00	5.65	A-7-6
123.745	31.00	38.10		
134.800	29.20	47.40	6.00	A-7-6
134.815	23.00	56.80		
134.830	24.04	56.20		
134.845	29.00	42.10	6.00	A-2-6
134.860	27.50	49.40	7.00	A-6
150.000	19.80	53.90	10.00	A-2-6
150.005	27.65	41.60		
150.180	18.60	52.20	11.00	A-2-7
150.185	22.65	48.50		
150.240	24.12	42.30		
150.245	23.64	45.80		
150.300	21.12	46.50		
150.305	21.22	46.90	9.70	A-2-6
151.000	20.22	49.80	9.97	A-2-4
151.005	24.36	41.90		
151.060	18.50	51.50	11.20	A-2-6
151.065	24.13	44.00		
151.120	26.90	41.50		
151.125	25.20	46.20		
151.240	22.70	48.00		
151.300	24.30	45.50		
151.305	18.60	52.60	11.00	A-2-5
152.000	26.00	44.50	7.80	A-2-6
152.060	18.50	59.00		
152.120	17.23	67.00	12.00	A-2-6
152.125	32.20	36.60	5.20	A-2-6
152.180	28.73	44.80		
152.240	27.59	47.90	6.80	A-2-6
153.350	17.53	51.30	11.50	A-2-6
153.355	15.97	55.90		
153.415	29.68	38.00	6.00	A-2-6
153.470	25.74	46.70		
153.535	27.13	40.70	6.90	A-2-5
153.590	27.20	41.00		
153.655	24.22	46.90		

LIST OF FIGURE CAPTIONS

FIG. 1-Details of DCP
FIG. 2-Different Methods of DCP Testing
(Modified and elaborated based on Livneh et al 2000)
FIG. 3-Relationships for Converting DCP_{60} to CBR (Srinivasa 2009)
FIG. 4-Comparison Between a Few Selected Relationships of DCP_{60} - CBR.

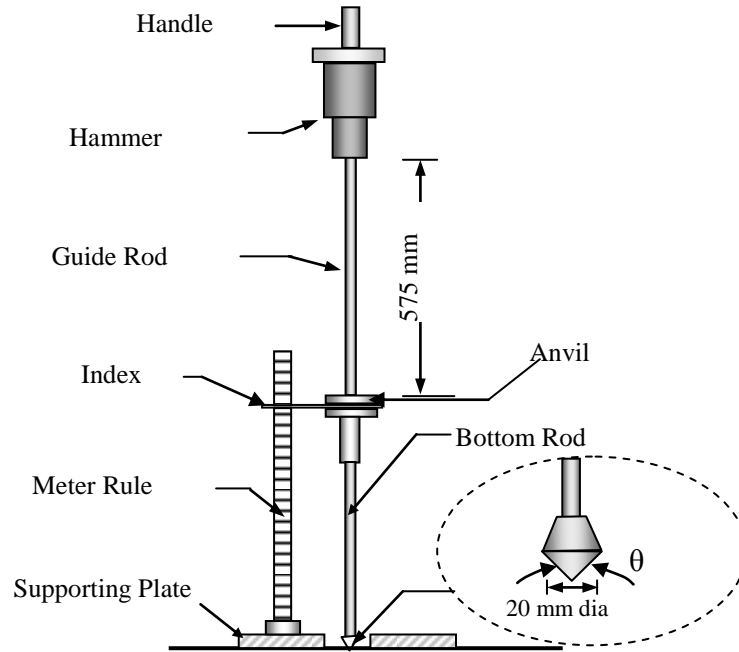FIG. 5-Relationships for Converting DCP_{60} to Elastic Modulus.
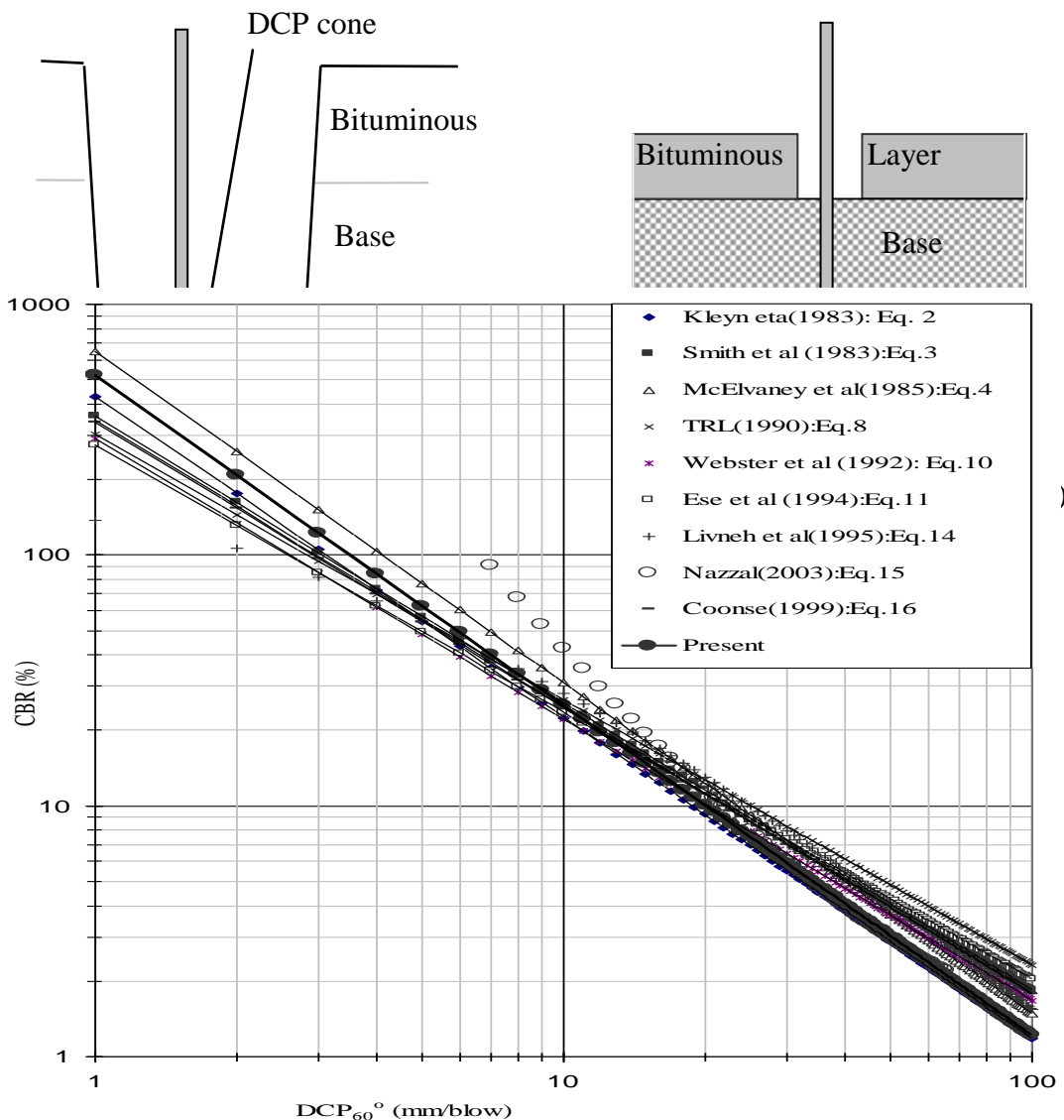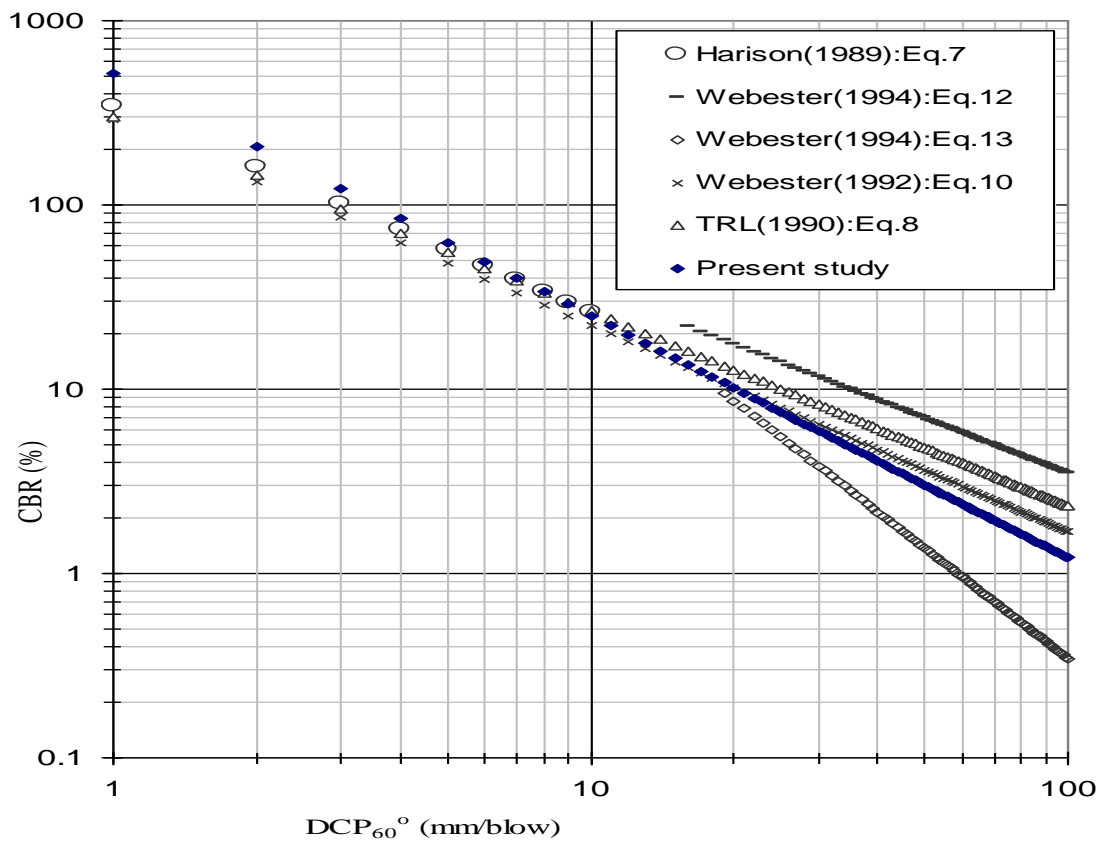

FIG. 1-*Details of DCP*.

FIG. 3-*Relationships for Converting DCP_{60}^{o} to CBR*.

FIG. 4-*Comparison Between a Few Selected Relationships of*
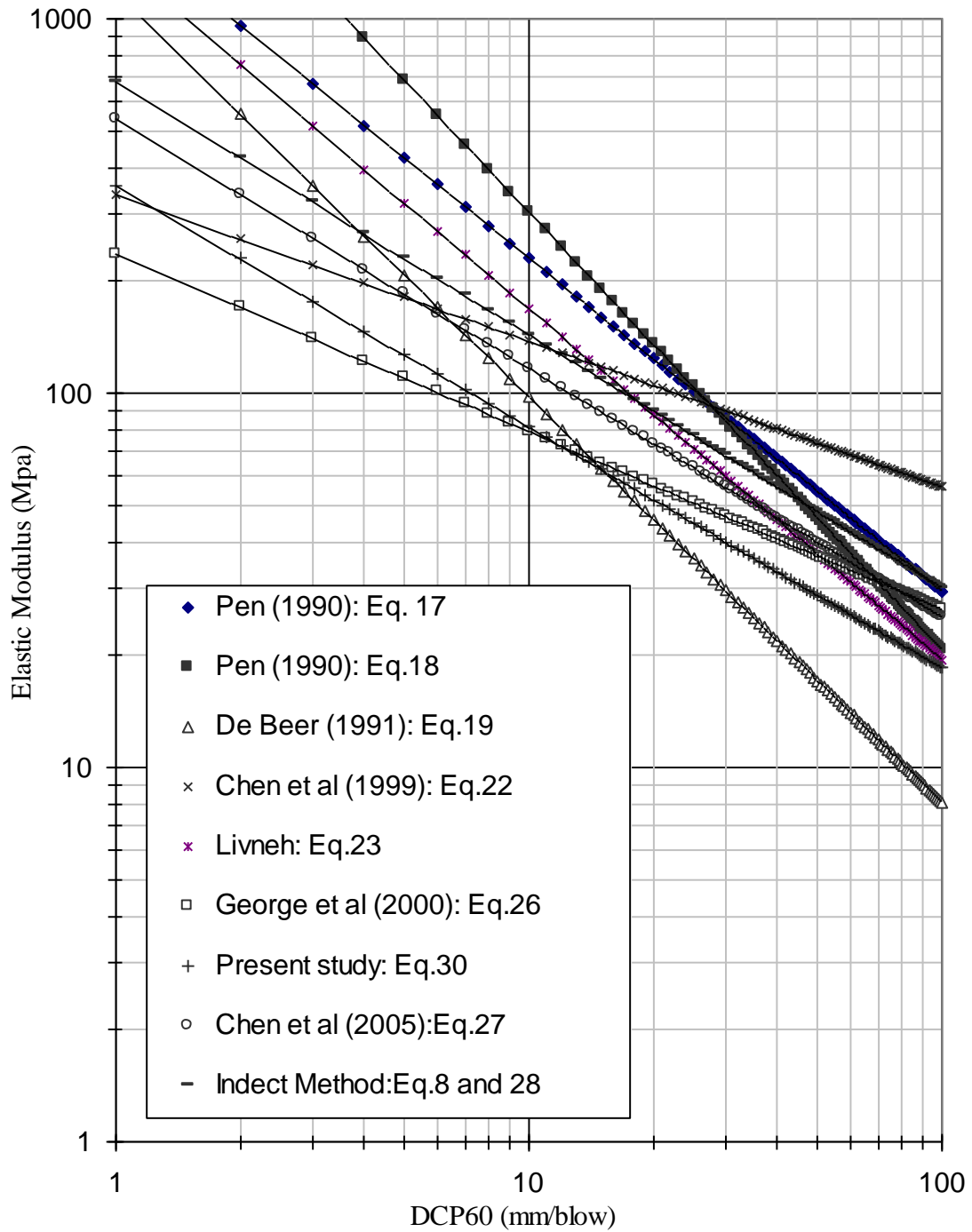$DCP_{60}{}^{o}$ *- CBR.*

FIG.5-*Relationships for Converting DCP $_{60}$ to Elastic Modulus.*