EVALUATION OF BASE AND SUBGRADE LAYER MODULI USING DYNAMIC CONE PENETROMETER

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Abstract: The Dynamic Cone Penetrometer (DCP) has become a cost-saving alternative for characterizing the properties of pavement layers. The goals of this study were to recommend a method to estimate modulus through DCP testing, to compare the moduli from different test methods, and to investigate any changes in stiffness using liquid stabilizers. Over 100 DCP tests have been conducted at various sites. Some were conducted on four full-scale APT sections. Several others have been done on in-service pavement sections. The FWD and SASW tests were conducted to compare the moduli from DCP measurements. The moduli from DCP tests are compatible with those from FWD-MDD tests. The moduli of base layers obtained from SASW testing were about 1.2 times higher than those from FWD-MDD tests, while subgrade moduli were about 2.3 times larger. Test results indicate the use of stabilizer did not improve the stiffnesses of base and subgrade layers.

Key Words: DCP, modulus, FWD, base, subgrade

1. INTRODUCTION

The Dynamic Cone Penetrometer (DCP) has been used to determine the bearing capacity of base and subgrade layers without digging test pits or collecting soil samples. The determination of in-situ California Bearing Ratio (CBR) by conventional methods is time-consuming and requires the use of costly equipment. The DCP is one of least expensive devices that can be used to characterize base and subgrade properties. A complete set up of the DCP equipment costs less than $1600. The DCP serves as an excellent tool for construction inspection; it has the ability to verify both the level and uniformity of compaction. In addition, the layer thickness can be determined from the changing slope of the depth vs. accumulated blows profile. Livneh *et al.* (1989) demonstrated that the results from penetration tests correlate well with in-situ CBR values. In addition, the layer thickness obtained from DCP tests corresponds reasonably to the thickness obtained in the test pits, and they concluded that the DCP tests are a reliable alternative for project evaluation. Harrison (1986) also found that there is a strong correlation between CBR and DCP results. He reported that changes in moisture content and dry density do not affect the CBR-DCP relationship. With such close relationship between CBR and DCP, pavement engineers are now able to use the DCP for rapid field inspection.

More than a dozen DOTs and federal agencies are currently using the DCP to assess the strength and uniformity of highway structures (Siekmeier *et al.*, 2000). For example, MnDOT (Burnham, 1996) adopted a requirement that the subgrade CBR should be at least 6 to minimize rutting damage to the finished grade (prior to paving) and to provide adequate subgrade support for proper compaction of the base and other layers. Soils with CBR values of less than 8 may need remedial procedures, such as sub-cutting, drying and compaction, backfilling with granular borrow, or lime treatment. They also adopt the same equation developed in 1992 by the US Army Engineers Waterways Experiment Station to compute the CBR value (Webster *et al.*, 1992). They found that the effects of soil moisture content and dry density influence both CBR and DCP values in a similar way and are considered negligible for the correlation.

The Texas Department of Transportation (TxDOT) has used stabilized subgrades and bases extensively. In fact, subgrade stabilization is almost routine in many districts, especially in those with clay subgrades. A pressing need exists to determine the effectiveness of stabilization of subgrades and base courses, to evaluate the current mixtures and thickness design approaches and to suggest realistic structural properties associated with these stabilized pavement layers. The Texas Transportation Institute conducted extensive research using the Falling Weight Deflectometer (FWD) and DCP to understand the mechanism of stabilization and engineering improvements in calcareous bases and subgrade layers [Little *et al.* 1995; Chen *et al.* 2001b]. The Kansas Department of Transportation has been using the FWD and DCP for pavement evaluation since the early 1990s (Chen, J. *et al.* 1999). The DCP was used to verify FWD measurements and moduli backcalculated from the deflection data. The DCP helps researchers to provide recommendations for modifications to current TxDOT mixture and thickness design approaches. The goal is to minimize structural damage within the stabilized base layer due to cracking unrelated to load, and load-associated fatigue cracking.

2. BACKGROUND

Rapid in-situ strength testing provides transportation agencies the opportunity to conduct quality assurance programs based on strength or modulus measurements. As a result, different devices will be applied in the field, and the correlation among those devices will be important. Also, the DCP is useful when the back-calculated moduli from Falling Weight Deflectometer (FWD) data is in question, such as when the AC thickness is less than 3 inches, or when shallow bedrock is present. These two situations often cause a misinterpretation of FWD data. The DCP can be applied in these two situations to increase the accuracy of the stiffness measurement. In addition, an FWD test may not be conducted directly on weak subgrade and/or base layers due to the large deflections that can exceed the equipment's calibration limit. In addition, many backcalculation programs are based on a linear-elastic concept, and testing on weak subgrade and base layers may cause nonlinear deformation.

Before the DCP can evaluate layer stiffness, an empirical correlation needs to be established. Many equations have been proposed in the past to correlate DCP results to CBR values, and

CBR values to moduli. Those empirical equations were reviewed, evaluated and compared against the results obtained from the Mobile Load Simulator (MLS) project. The MLS is a full scale, accelerated pavement testing (APT) device. In this study, over 60 DCP tests have been conducted on or near four test sections located on the south and north bound lanes of US281 near Jacksboro, Texas. From the same test site, FWD, laboratory, Spectral Analysis of Surface Waves (SASW), and in-situ instrumentation results are available for comparison. In addition, 40 more DCP and FWD tests have been conducted on several in-service pavement sections.

3. DCP TESTING PROGRAM

3.1 Test Section

The majority of the test results (FWD, SASW, DCP) used in this study were from US281. Thus, a detailed description is presented for this test site only. US281 is a two-lane highway (in each direction, a total of four lanes) in the Fort Worth District. The Fort Worth District Pavement Engineer indicated that there was an average of 3,100 vehicles per day (1,550 per direction) in 1994. The percentage of trucks is approximately 17.4%. The first asphalt layer of the test section was constructed in 1957. There were four major overlays/rehabilitations that were completed in 1971, 1976, 1986, and 1995. There were four major upgrades/rehabilitations that were completed in 1971, 1976, 1986, and 1995. The last major rehabilitation was done in 1995, with asphalt concrete processes. The layers most tested by DCP for this study are the 380mm flexible base and soft (average modulus 86 MPa) subgrade. Neither of these layers has been reworked since the road was originally constructed in 1957. Accelerated pavement testing was applied on south and north bound lanes of US281. Approximately 972,000 and 388,800 ESALs were applied by the Texas MLS to the south and north bound lanes, respectively. Details of this research can be found in references (Chen and Hugo, 1998; 2001a).

The DCP and FWD were applied to three job sites in the Dallas District (IH635FR, FM 2818, IH30) and one site in the Austin District (US290). These four projects were chemically treated with liquid stabilizer. The effectiveness of chemical treatments was evaluated based on stiffness measurements of the stabilized layers and adjacent non-stabilized layers.

Two sections of US290 were tested. Both sections consist of 2 inches of AC over 12 inches of crushed limestone base. The only difference between the two sections is that the base of the first section was treated with EN1 liquid stabilizer.

The pavement structure of frontage road (FR) IH635 consists of a 4-inch AC on top of 24 inches of liquid-stabilized (EMC squared/EMS) subgrade. Figure 1 shows the preparation of implementing DCP and FWD tests on IH635.

The pavement of IH 30 consists of a 7 inches of AC and 8 inches of EMC Squared/EMS treated subgrade.

The FM2818 pavement structure also has 4 inches of AC. Two different sections were tested with the DCP and FWD. The first pavement section included 4 inches of AC over 12 inches of subgrade modified with EMC Squared/EMS. This section was constructed on approximately 15 feet of fill material. Down approximately 10 feet (within the fill material) there are another 12 inches of subgrade modified with EMC Squared/EMS. The second section of FM2818 included 4 inches of HMAC over 6 inches of subgrade treated with EMC Squared/EMS.

3.2 DCP Testing

Livneh *et al.* (1995) reported that vertical confinement effects on DCP values (due to upper asphalt layers) do exist. Since this is the true effect of the pavement structure, any DCP measurement for pavement evaluation purposes should be performed through a narrow hole in the asphalt layers and not after removal of a wide strip of asphalt (Livneh *et al.*, 1995).

Three types of DCP tests were conducted on US281 to investigate the effects of test procedure on DCP results. The most common procedure involved drilling through the AC layers, about 200mm (7.5~8 inches) in this case. Then, the DCP was started on the top of the base layer. DCP tests were also conducted when the asphalt layers were removed for another reason, such as to collect core samples, install sensors, or remove larger blocks of AC for Nuclear Density Gauge tests. These tests were considered similar to the drilled-AC tests, though the removal of overburden pressure may have raised the penetration rate. The third type of DCP test was conducted with no drilling or removal of the AC layers. As this method was very labor-intensive and damaging to the equipment, only 9 such tests were run. Some of the DCP tests were conducted after traffic load was applied to observe any effects on the DCP values. Some modifications in both the testing apparatus and the testing procedure procedures have been reported by Livneh, M. *et al.* (2000) and Livneh, M. (2000).

Tables 1 through 4 show the DCP results obtained in this study. As observed in Tables 1 to 4, Coefficients of Variation (COVs) from subgrade data are higher than from base data. Since varied test procedures would affect results by at least 10%, it is preferable to conduct all DCP tests through a drilled hole. Figure 2 shows the DCP values for base and subgrade layers of all test sections; each different mark represents the average value for individual DCP testing.

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4. RELATIONSHIP BETWEEN DCP AND CBR VALUES

4.1 Correlation Equation Between DCP and CBR

For a wide range of granular and cohesive materials, Corps of Engineers found a relationship as in Eq. 1 (Webster *et al.*, 1992). Eq. 1 has been adopted by many researchers and practitioners (Livneh 1995, Webster *et al.*, 1992, Siekmeier *et al.*, 2000) and will be used in this study as well.

$$\log CBR = 2.465 - 1.12\,(\log PR) \quad or \quad CBR = 292/PR^{1.12} \tag{1}$$

Where CBR: California Bearing Ratio
PR: the penetration through the layer in units of mm/blow

4.2 Equations to Relate CBR to Modulus

One of the most commonly required inputs in pavement design is the modulus value. Thus, the relationship between CBR and modulus becomes essential to implement the DCP in pavement evaluation. The 1993 AASHTO Guide for Design of Pavement Structures adopted Eq. 2 for calculating moduli (E), which was proposed by Huekelom and Klomp (1962)

$$E(psi) = 1500 * CBR \quad or \quad E(MPa) = 10.34 * CBR. \tag{2}$$

The moduli from which this correlation was developed ranged from 750 to 3000 times the CBR. Also, the formula is limited to fine-grained soil with a soaked CBR of 10 or less. Powell *et al.* (1984) indicated a relationship between modulus and CBR as

$$E(psi) = 2500 * CBR^{0.64} \; or \; E(MPa) = 17.58 * CBR^{0.64} \tag{3}$$

Eq. 3 was selected to compute modulus values in this study. A relationship between CBR and modulus has been reported by Van Til *et al.* (1972). This study also compared the moduli obtained from all CBR-Modulus relationship.

5. COMPARISON OF MODULI FROM DCP AND OTHER TEST METHODS

5.1 Moduli from FWD-MDD Tests

Deflection profiles from FWD tests provide information valuable for the assessment of pavement layer moduli through a backcalculation process. The use of the backcalculated moduli is often obtained from a best fit to the measured deflection profile. The verification of backcalculated moduli has been done in two ways: (1) engineering judgment; and (2) comparison with other test results such as laboratory and field seismic testing. Kim *et al.* (1992) and Uzan and Scullion (1990) applied FWD tests on top of a Multi-Depth Deflectometer (MDD) and measured the resulting surface deflections and depth deflections simultaneously using both the FWD geophones and the MDD. They found that the depth deflections measured by the MDD could be a powerful tool in evaluating the accuracy and dependability of backcalculated moduli values from FWD data. Fig. 3 shows a schematic of the FWD-MDD test.

Four different loads, 25, 40, 52, and 67 kN, were applied in close proximity to the MDDs with each load repeated three times to examine the repeatability of the results. Only the results from 40 kN loads were reported here. Detailed discussion of the FWD-MDD test is given in Chen *et al.* (1999).

Moduli were found through several iterative computations to reconcile deflections measured by the FWD (on the surface) and MDD (at depth). Note that the base and subgrade layers were constructed in 1953. Only small changes in base and subgrade moduli during trafficking were expected.

5.2 Moduli from SASW Tests

In this study, the data from the SASW tests were used to estimate the modulus of each layer of the pavement at each test section. The SASW method can be used to determine the modulus profile of a pavement section. Detail of the theoretical and experimental aspects of the SASW method can be found in Nazarian *et al.* (1995) and Yuan *et al.* (1998). Table 5 presents the moduli from the FWD-MDD and SASW methods for both before and after trafficking.

5.3 Moduli from Laboratory Tests

Ten Shelby-tube samples were collected for lab testing. The lab tests (for resilient modulus) were performed at the Texas Transportation Institute. Three different deviator stresses and two confining pressures were applied in the testing. The tests were conducted under triaxial conditions to obtain modulus values and permanent deformation properties up to 20,000 load applications. It is not the scope of this study to discuss the permanent deformation properties.

The modulus values presented here are determined at the 200th load repetition. The determination of the modulus values through laboratory testing is well documented in the literature. The modulus values and the corresponding deviator stresses and confining pressures are given in Table 6. From analyses using the program BISAR, it was found that the deviator stresses and confining pressures for the pavement structure under a 40 kN load are approximately 20 to 35 kPa (3 to 5 psi) and 7 to 14 kPa (1 to 2 psi) respectively. For the deviator stresses and confining pressures the pavement will encounter, the modulus values are approximately 96 to 103 MPa (14 to 15 ksi).

5.4 Comparison of Moduli for DCP and Other Tests

Comparisons of the modulus values from different test methods are presented in Fig. 4. The DCP-determined moduli of base and subgrade layers were very close to those obtained from FWD-MDD tests. The comparison shows the DCP-determined moduli obtained from Eq. 3 were much better than those from others. In addition, the moduli from Eq. 2 were much higher than those moduli from other Equation or chart especially for base layers. Using Eq. 1 to compute CBR and then using Eq. 3 to compute moduli values agreed well in this case. Eq. 3 has been recommended to TxDOT for further evaluation and routine analysis.

The laboratory determined subgrade moduli were higher than those from DCP and FWD-MDD tests. No correction factor was required for the backcalculated moduli to match the laboratory moduli.

The effects of MLS loading on base and subgrade layers of the 281S site were investigated. Prior to MLS loading, the average CBR values of 69 and 12 (moduli values of 262 and 86 MPa) were found for base and subgrade, respectively. After approximately 972,000 ESALs of MLS loading, the average CBR values of 57 and 12 (moduli values of 232 and 83 MPa) were found for those layers. The DCP tests were conducted from the top of the base through holes drilled in the AC. There were insignificant changes (less than 10%) in the coefficient values before and after loading. Also, the average moduli values for the subgrade were approximately the same before and after trafficking. DCP testing indicated a reduction in base modulus value from 262 to 232 MPa. The subsurface layers did not deteriorate much due to the surface loading, probably because of the thick AC cover and the fact that they have been in service since 1957.

FWD-MDD test results at the above section also indicated the subgrade moduli were approximately the same before and after MLS testing. The reduction in base moduli due to loading, according to FWD-MDD tests, was 241 to 220 MPa. After 972,000 ESALs no cracking had been observed, and the average rut depth was approximately 4mm.

The laboratory determined subgrade moduli were only slightly higher than those from DCP and FWD-MDD tests. The moduli of base and subgrade obtained from SASW testing were about 1.2 and 2.3 times as large as those from FWD-MDD testing, respectively.

6. STIFFNESS EVALUATION OF STABILIZED BASE AND SUBGRADE

The cost effectiveness of the lime, cement and flyash stabilized base and subgrade has been well recognized. However, the limitations of each treatment are equally real. Normally, cement is used to treat sandy soils and lime is applied to stabilize clay materials. However, the presence of sulfate may render a lime treatment ineffective and cause the soil to expand. Premature failures of lime stabilization projects have been reported by the Dallas District in areas known to have clays with a high-sulfate content. Lime treatment will generally increase the pH of a soil. However, due to non-ionic and neutral pH properties of this blend of stabilizers, the soil pH does not radically change.

Chemical treatment could be a viable alternative to lime when treating the sulfate-rich expansive clay. One of objectives of this study was to evaluate the effectiveness of two chemical treatments using EN1, and a blend of EMC Squared and EMS stabilizers. FWD and DCP tests were conducted on four liquid-stabilized projects to determine any changes in stiffness due to application of the liquid stabilizers.

For IH 635, an effort was made to determine the untreated subgrade modulus value below the treated layer, as shown in Fig.5. Based on the DCP results, no gain in stiffness can be observed in this EMC Squared-treated subgrade. It is not uncommon to have a lime-stabilized subgrade layer exceeding 1000 MPa. Both DCP and FWD backcalculated moduli indicated that the modulus of the EMC Squared-treated subgrade layer was less than 173 MPa. Comparison of the treated subgrade and the underlying untreated subgrade of FM2818 was found no gain in strength, as shown in Fig. 6. The modulus from DCP results was approximately 76 MPa for both treated and untreated sections. The average modulus (117 MPa) of the treated subgrade in this area is higher than from IH635FR (62 MPa) and FM2828 (76 MPa). This conclusion was based on DCP test results. The higher strength in this area was not due to the treatment but due to higher quality of the existing material. This observation was derived from the comparison of the treated and untreated subgrade below. For IH30, Figure 7 shows how the treated layer may be stiffer or softer than the underlying subgrade. The test results from the EN1 treated base, and EMC Squared/EMS treated subgrade indicate that no gain in stiffness was observed by using the liquid stabilizers. The stabilizers may improve the base and subgrade materials in other ways (such as permeability, frost heave, etc.) but the in-situ tests in this study indicate no improvement in stiffness.

7. CONCLUSIONS

For this study, over 60 DCP tests were conducted on or near two test pavements located on the south and north bound lanes of US281 near Jacksboro, Texas. The same sites have been used for the MLS project, so the moduli values from many other test methods are available. The effects of testing conditions on the DCP values were studied and the DCP moduli were compared with those from FWD-MDD, SASW and laboratory tests. In addition, 40 more DCP and FWD tests have been conducted at several in-service pavement sections. The conclusions are given as follows:

- It was found that using Eq. 1 to compute CBR and then using Eq. 3 to compute moduli from DCP tests yielded compatible results with those from FWD-MDD tests. Eq. 3 has been recommended to TxDOT for further evaluation and/or adoption into routine analysis.

- The laboratory-determined subgrade moduli were only slightly higher than those from DCP and FWD-MDD tests. The 1993 AASHTO Design Guide suggested that a factor of 0.33 should be applied to backcalculated moduli to match laboratory moduli. The factor is not applicable in this case.

- The moduli of base and subgrade obtained from SASW testing were about 1.2 and 2.3 times higher than those from FWD-MDD testing, respectively.

- Based on test results from treated and non-treated materials, there is no evidence to support the claim that the chemical treatments lead to increased stiffness. Using these liquid stabilizers did not lead to a consistent, measurable increase in stiffness.

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Table 1. DCP Results for 281S (Through a Drilled Hole)

Before MLS Loading							
	Base (10 Tests)				Subgrade (10 Tests)		
	mm/blow	CBR	Modulus (MPa)		mm/blow	CBR	Modulus (MPa)
Average	3.76	69	262	Average	18.59	12	86
St.Dev.	0.67	14	34	St.Dev.	7.26	4	20
COV	18%	20%	13%	COV	39%	36%	23%
After MLS Loading							
	Base (9 Tests)				Subgrade (9 Tests)		
	mm/blow	CBR	Modulus (MPa)		mm/blow	CBR	Modulus (MPa)
Average	4.46	57	232	Average	19.49	12	83
St.Dev.	0.77	11	30	St.Dev.	5.97	4	17
COV	%17	20%	13%	COV	31%	33%	21%

Table 2. DCP Results Before MLS Loading for 281N (Through a Drilled Hole)

	Base (11 Tests)				Subgrade (10 Tests)		
	mm/blow	CBR	Modulus (MPa)		mm/blow	CBR	Modulus (MPa)
Average	3.75	70	265	Average	14.05	18	110
St.Dev.	0.77	20	46	St.Dev.	4.82	6	24
COV	21%	28%	17%	COV	34%	34%	22%

Table 3. DCP Results After MLS Loading for 281S

Through Removal of AC Slab							
	Base (3 Tests)				Subgrade (3 Tests)		
	mm/blow	CBR	Modulus (MPa)		mm/blow	CBR	Modulus (MPa)
Average	5.29	46	203	Average	17.75	14	92
St.Dev.	0.71	7	21	St.Dev.	7.19	8	33
COV	13%	16%	10%	COV	41%	57%	36%
Through AC Surface							
	Base (9 Tests)				Subgrade (9 Tests)		
	mm/blow	CBR	Modulus (MPa)		mm/blow	CBR	Modulus (MPa)
Average	3.78	70	264	Average	13.52	17	106
St.Dev.	0.79	21	48	St.Dev.	3.40	4	18
COV	21%	30%	18%	COV	25%	26%	17%

Table 4. DCP Results for other Pavement Sections

IH 365 (Subgrade, 7 Tests)				FM2818 (Treated Subgrade, 7 Tests)			
	mm/blow	CBR	Modulus (MPa)		mm/blow	CBR	Modulus (MPa)
Average	27.94	9.35	10.29	Average	20.98	9.68	10.90
St.Dev.	4.22	6.60	4.20	St.Dev.	1.06	0.53	0.39
COV	15%	71%	41%	COV	5%	6%	4%
FM2818 (Untreated Subgrade, 7 Tests)				IH 30 (Subgrade, 5 Tests)			
	mm/blow	CBR	Modulus (MPa)		mm/blow	CBR	Modulus (MPa)
Average	20.96	70	265	Average	16.61	22.71	17.44
St.Dev.	4.70	20	46	St.Dev.	12.17	21.98	10.70
COV	22%	28%	17%	COV	73%	97%	61%
US290/281 Intersection (Base, 2 Tests)							
	mm/blow	CBR	Modulus (MPa)				
Average	0.22	1316	251				

Table 5. Moduli from FWD-MDD and SASW Tests

	Modulus (MPa)							
	Base				Subgrade			
Site	FWD-MDD		SASW		FWD-MDD		SASW	
	before	after	before	After	before	after	before	After
281S	241	220	290	293	76	69	168	172
281N	262	255	308	252	69	69	170	140

Table 6. Moduli from Laboratory Triaxial Tests

Site	Laboratory Triaxial Testing		
	Deviator (kPa)	Confining (kPa)	Modulus (MPa)
281S	34	14	105
	34	14	97
	21	7	96
	69	14	25

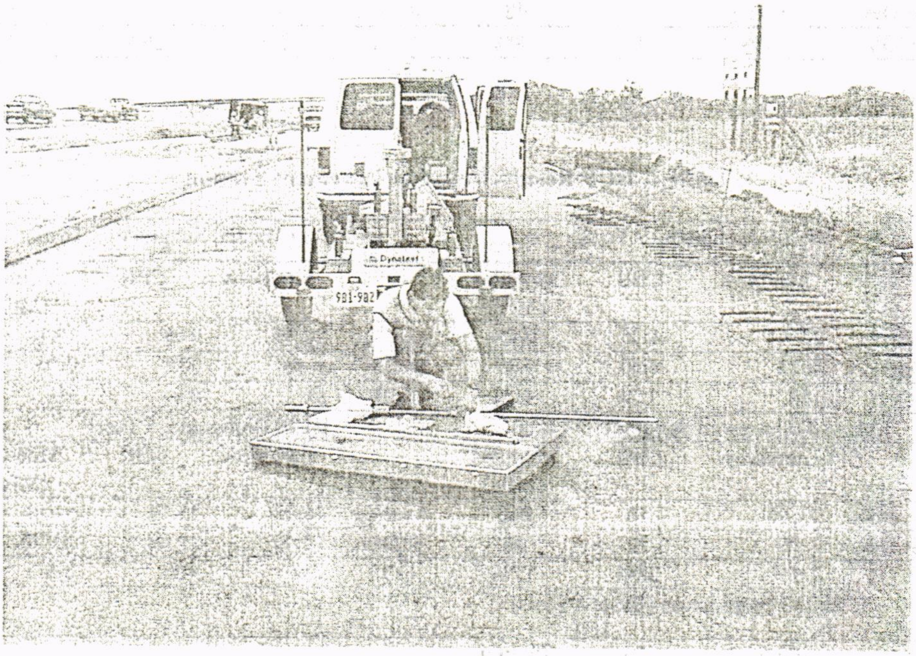
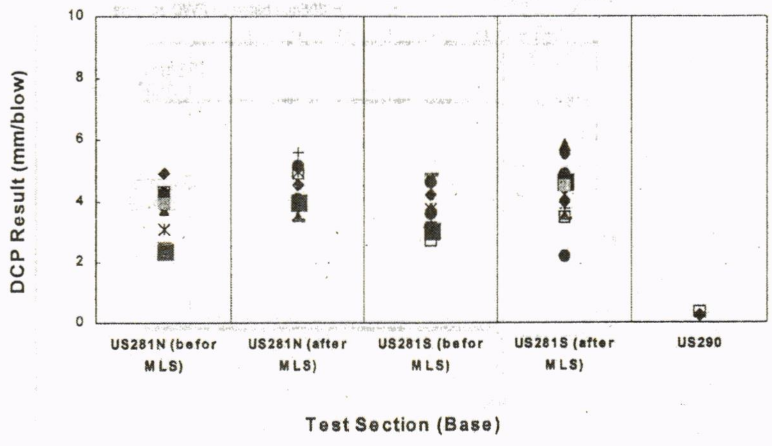
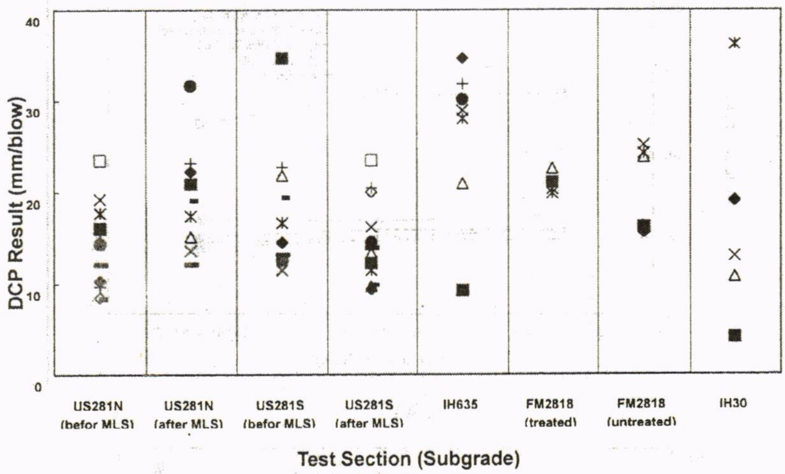


Figure 1. Preparation for FWD and DCP Tests on IH635



(A)



(B)

Figure 2. DCP Measurements for (A) Base (B) Subgrade

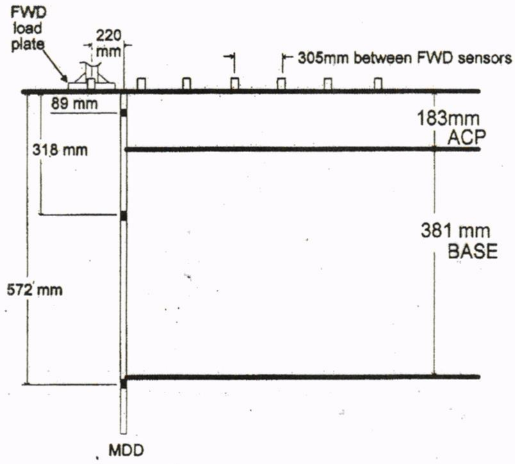
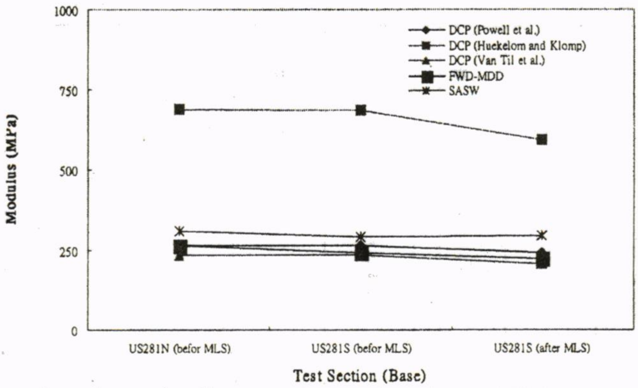
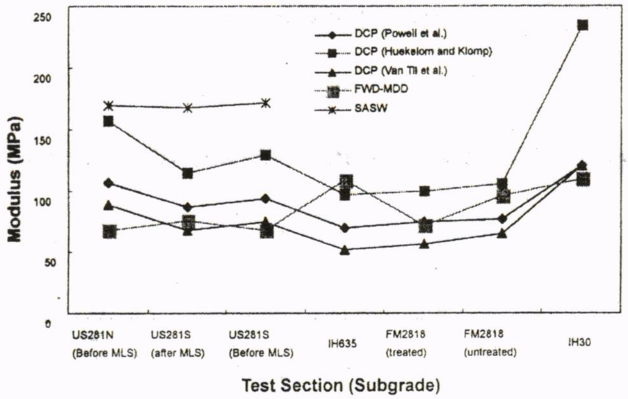


Figure 3. Schematic of FWD-MDD Test



(A)



(B)

Figure 4. Comparison of Modulus Values for Different Testing (A) Base (B) Subgrade

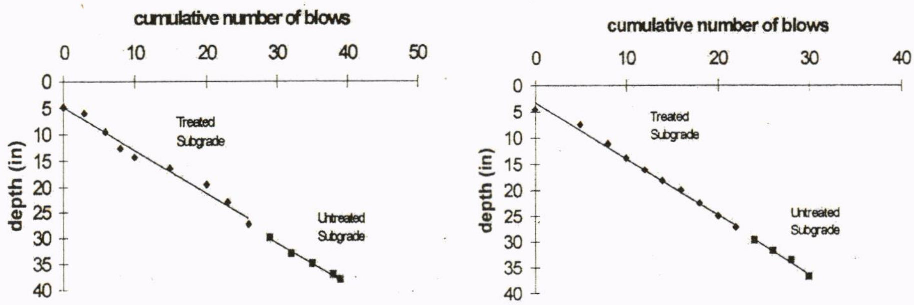


Figure 5. DCP Results from IH635 for Treated and Untreated Layers

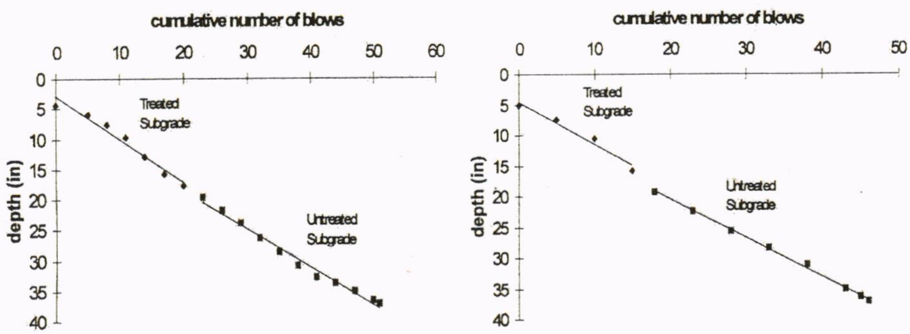


Figure 6. DCP Results from FM2818 for Treated and Untreated Layers

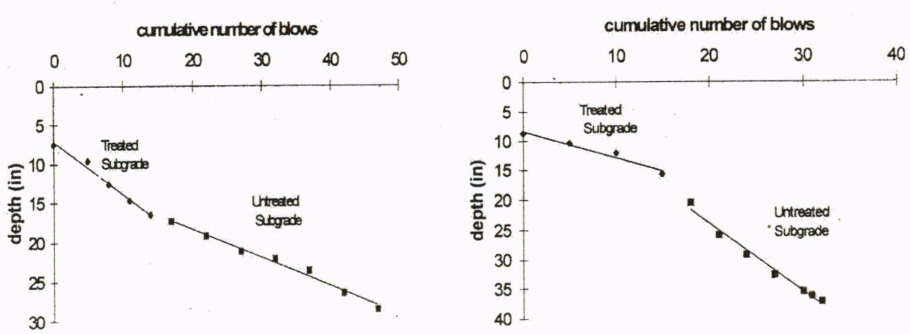


Figure 7. DCP Data from IH30 for Treated and Untreated Layers