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LTPP Data Analysis: Feasibility of Using FWD Deflection Data to Characterize Pavement Construction Quality

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SUMMARY OF FINDINGS

Good materials and construction practices are vital to producing high quality and long lasting pavements. Quality control and assurance techniques can provide the owner of the roadway the means to ensure that these desired ends are achieved. The objective of this research study is to determine whether the Falling Weight Deflectometer (FWD) can effectively be used to assist in the QC/QA process during pavement construction.

The Long Term Pavement Performance (LTPP) study provided the data needed to accomplish this research. LTPP began, in about 1988, as the Strategic Highway Research Program (SHRP), and it included a subset of newly constructed or rehabilitated pavements called the Specific Pavement Studies (SPS). Some of these were targeted at pavement maintenance and rehabilitation, while others were for new construction. New construction was subdivided into three LTPP experiments, one for asphalt concrete (AC) surfaced pavements (SPS-1), one for Portland Cement Concrete (PCC) surfaced pavements (SPS-2), and one for both AC and PCC pavements under the influence of light traffic loads (SPS-8). In each case, several thickness and material designs were developed in an experimental design matrix. These SPS experimental design matrices were constructed under a variety of climatic conditions and material sources in several states.

During construction of the SPS-1, -2 and -8 projects, a large number of field and laboratory tests were performed on each structural layer. These tests included densities, moisture contents, layer thickness measurements and others, along with FWD tests conducted at each layer interface.

This report shows that the FWD data is of reasonable quality and certainly of sufficient quantity to carry out the research. The traditional test results, although not as extensive as FWD data, were also reasonable and appeared to cover most of the wide variety of pavement designs and construction locations represented by the LTPP/SPS database.

Sixteen SPS-1, SPS-2 and SPS-8 test sites (each site represents between 2 and 12 test sections) were utilized for analyses during this research project. These sites represent a wide variety of materials, climates, and construction dates.

It was found that the FWD deflections tracked (followed a similar deflection pattern as a function of stationing) from layer to layer as construction proceeded. It was also possible to document the material variations encountered in each layer. Three early pavement failures were also detected with the FWD.

This study used only very simple, straightforward methods of analyzing the FWD load-deflection data. These methods included the calculation of the "composite" modulus, or apparent stiffness, of the materials under each layer interface for unbound material tests, and an approximate calculation of the stiffness of a single bound layer for the bound layer tests. The stiffness of any bound layer was derived through a simple formula, using the composite modulus, E_o, and the "AREA" of the deflection basin. AREA is similar to a basin shape factor or curvature index of the FWD's deflection basin out to a distance of 3 feet (~0.9 m). The research confirms that these values represent reasonably well the material properties and their variations, in terms of modulus or stiffness, along the pavement sections studied.

In one SPS site, using FWD tests conducted on a series of new SPS-1 projects (12 sections total), the results showed that two of the twelve AC surfaced sections had no FWD data from the AC surface at all, even though data from all the other structural layers were present. A third section exhibited extremely high deflections (and correspondingly low stiffness values), while a fourth section had somewhat questionable values. It was subsequently found that the two missing sections from the database had been prematurely taken out of service, and were removed from the SPS experimental design matrix due to a rapid pavement failure (rutting and lack of bond between layers). The third section, where the FWD detected a noticeable problem with the surface course, had to be taken out of service a short time later, while the fourth suspect test section survived for about a year and was then taken out of service — all for essentially the same reasons: rutting, shoving, and surface distortion.

It was also found that the FWD test results on the unbound materials were reasonably well related to material properties determined from laboratory and other in-situ field tests. It was also possible to use FWD-derived parameters to detect the difference between stronger and weaker (nominal) flexural strength PCC.

To conclude, FWD test results provide data that can be used with confidence to estimate material properties — mainly stiffnesses or moduli — and their variations at each layer interface during new or reconstructed pavement construction. These values generally “track” well from layer to layer as construction proceeds, and they are moderately well correlated to other measures of pavement quality.

The results of this project should be useful for further research, to develop a load-deflection test protocol (or test method), and to finalize a means of analyzing load-deflection data in the field as construction proceeds.

CHAPTER 1—INTRODUCTION AND RESEARCH APPROACH

1.1 Research Goals

As stated in the Research Problem Statement, use of the Falling Weight Deflectometer (FWD) has become an integral part of the process for structural evaluation of pavements in recent years. In addition, there is a potential for using FWD data to characterize pavement construction quality. For example, FWD deflection data obtained shortly after constructing each pavement layer may reveal variations in deflections and shape of deflection basins, along a project or among projects, that can be caused by differences in the properties of pavement layers resulting from construction variability. Therefore, procedures that take into account the effects of loading schemes, deflection measurement locations, deflection basin parameters, and other related factors could be used to analyze deflection data and characterize construction quality. However, the feasibility of developing such procedures has not been established.

Research is needed to evaluate the feasibility of developing methods to characterize pavement construction quality based on FWD deflection data. If proven feasible, the research will encourage further validation and implementation by highway agencies and lead to effective means for characterizing construction quality that can be used to enforce pavement quality specifications. The data for the Long Term Pavement Performance (LTPP) studies are expected to aid in such an evaluation.

The objective of this research is to evaluate, based on the data available from the LTPP studies, the feasibility of developing procedures to characterize construction quality of new and reconstructed flexible and rigid pavement based on FWD deflection data and, if feasible, recommend such procedures. The research shall use the data available in the LTPP Information Management System (IMS) database classified as "Level E" and, if needed, be supplemented with relevant data from other sources.

1.2 Introduction

In this report, it is assumed that the ultimate or ideal goal (as stated above) " … of developing procedures to characterize construction quality" during pavement construction is to eventually use these procedures for quality control, and quite possibly quality assurance, for either the contractor in the first instance or the contracting agency in the second instance. This is the reason that the research (also as stated above) " … will encourage further validation and implementation by highway agencies and lead to effective means for characterizing construction quality that can be used to enforce pavement quality specifications." This ultimate goal, if proven feasible, goes far beyond merely characterizing pavement layers for quality, and even beyond quality control without any enforcement or absolute criteria, to what is commonly known as quality assurance with enforceable specifications based on a deflection-based test and analysis method. The goal here is therefore to ascertain, through this research, whether or not such a goal is feasible.

Although traditional Quality Control and Quality Assurance (QC/QA) tests on unbound materials have been successfully used in the past during pavement construction, these traditional tests do have some shortcomings, for example:

- Traditional test methods are time-intensive, and therefore often have a sampling frequency that is inadequate for highly variable construction materials (such as natural subgrades).
- Many of the traditional QC/QA tests on unbound materials produce indirect measures of pavement quality, which are then, in turn, related to the stiffness or strength (the bearing capacity) of these pavement layers.
- The "raw" density (in kg/m^3 or lbs/ft^3) of an unbound material is not particularly relevant by itself, but only as it relates to the maximum dry density (Proctor or Modified Proctor) of that particular material, commonly known as degree of compaction.
- The moisture content (in percent of dry weight) of an unbound material is also not very relevant by itself; rather, the moisture content needed to achieve a maximum or optimum dry density of that particular material is more important.
- Other unbound material tests, apart from density and moisture content, are generally related to a material's immediate or single-load shear strength on a very localized sample. Examples include: CBR, R-value, DCP, or unconfined compressive strength, etc. Shear strength, in turn, is only indirectly related to modulus or stiffness and, therefore, to repeated load bearing capacity.

Traditional QC/QA tests on *bound* pavement layers, on the other hand, are more direct and therefore more informative, although sampling frequency is oftentimes a shortcoming. Based on sampling frequency alone, the Falling Weight Deflectometer (FWD) could well provide more accurate information that can be used for QA or QC during pavement construction than the traditional test methods are able to provide.

Although FWD tests can be conducted quickly and with a greater sampling frequency than most traditional QC/QA tests, this approach may also have its own drawbacks, for example:

- For unbound materials, changes in moisture content or other factors that occur after construction can affect the deflections and, by extension, the results and conclusions that can be drawn from deflection readings.
- Care is needed to insure that the unbound material surface under test is level enough to accommodate a standard FWD loading plate.
- During construction, the layer of the pavement of most concern, for each successive structural layer, is the upper portion of the material(s) under test. All structural layers, including the deeper portions of the subgrade, influence FWD deflections. This confounds many analysis procedures, or requires complex and oftentimes non-unique backcalculation techniques that may not be readily acceptable as part of a QC/QA test method for new pavement construction.

For all of the above reasons, an ideal approach to "total quality control" during pavement construction may well be a combination of the more traditional procedures and newer FWD (or other deflection-based) test methods.

The LTPP study comprises a large database that includes FWD tests conducted during and shortly after pavement construction, along with a very large volume of other FWD tests conducted on the pavement surface at various times during the pavements' lifetime. The subset of test results associated with new pavement construction is primarily associated with the Specific Pavement Studies (SPS), specifically SPS-1 (AC-surfaced), SPS-2 (PCC-surfaced) and SPS-8 (AC- or PCC-surfaced under light traffic loads) that were constructed "from the bottom up” and tested during and immediately after construction. The volume of FWD data that alone exists in the SPS-1, -2 and -8 database was easily sufficient to conduct the necessary research for this project.

In contrast, the more "traditional" QC/QA types of data elements in the SPS experimental database appear to be somewhat limited. Two reasons for this have been detected: first, because fewer traditional tests were conducted in general, and second because some of the data may not yet exist at Level E (data that has undergone sufficient QC checks) in the IMS. Nevertheless, adequate data was found in the IMS to draw conclusions about the use of the FWD for new pavement construction, both to compare the FWD data obtained at the various layer interfaces and to compare FWD data with the traditional QC/QA test data available.

1.3 Background and Research Approach

One of the many potential uses of data gathered during the LTPP program is to ascertain the usefulness or effectiveness of new pavement construction data, including current or potential QC or QA data.

Currently, the following QC/QA related data categories are present in the LTPP Information Management System (IMS):

<u>Unbound material-related data elements</u>:

The following are the key subgrade and granular base data types that are used to characterize the quality of layers and are, or will be, available for LTPP’s SPS-1, SPS-2 and SPS-8 projects:

1. FWD Deflection Data from lane "S-" and "G-" tests (S = Subgrade, G = Granular Base)
2. Backcalculated Resilient Modulus of Subgrades and Bases
3. Layer Thicknesses (of any Layer above the Subgrade)
4. Field Densities (Nuclear) & Moisture Contents
5. Proctor or Modified Proctor Densities
6. Laboratory Resilient Moduli
7. Other Unbound Material Properties (e.g., gradations, liquid limit, etc.)

<u>AC-related data elements</u>:

The following are the key data types that are used to characterize the quality of the asphalt layers and are, or will be, available for the SPS-1 and SPS-8 projects:

1. Backcalculated Resilient Moduli of Asphalt Layers
2. AC Layer Thicknesses
3. Resilient Moduli from Cores, Test Method P07
4. AC Temperatures During FWD Testing

<u>PCC-related data elements</u>:

The following are the key data that are used to characterize the quality of the concrete (or lean concrete) layer and are available for the SPS-2 and SPS-8 projects:

1. Backcalculated Resilient Moduli of PCC Layers
2. Concrete compressive strength
3. Concrete flexural strength
4. Concrete split tensile strength
5. Concrete modulus of elasticity from cores or cylinders

The above data provide benchmarks for evaluating the results of FWD data analyses conducted in this research. Based on the data listed above, these three primary questions are addressed in this report:

1. Do FWD-based data, as obtained during construction of the various layers, relate to the FWD deflections taken immediately after construction has been completed?
2. Do FWD-based parameters or variables, as obtained during construction, relate to other traditional QC/QA tests conducted during construction or planned for as part of the pavement design process?
3. Based on the responses to the above two queries, can FWD deflection data be used to determine the quality of pavement layers during construction? Or more simply stated: Can the deflection-based parameters and their variability (for example, due to poor compaction or construction control) be quantified for use in a QC or QA test procedure?

It should be mentioned that a surprisingly large volume of literature already exists on the subject. Unfortunately, few if any of these references actually concluded, one way or the other, whether the FWD could be used as a surrogate for traditional QC/QA tests or as an additional QC/QA tool for new pavement construction. Some of the useful findings from recent studies are summarized, as follows:

The importance of using "SQA" (Statistical Quality Assurance) specifications and procedures when conducting QA or QC testing for new construction is encouraged [see Reference (1)]. Some examples of the magnitude of and variation in QA parameters are shown, for both unbound and bound materials; however, few if any of these are related to the types of variables studied in this research project. The use of SQA approaches, however, will be emphasized as needed in this report.

Many methods for in-situ measurements on unbound soils and subgrades have been shown (see Reference (2)]. The first conclusion provided in this reference is: "As agencies change from purely empirical to mechanistic-empirical design procedures, in-situ test and analysis methods must be developed to provide the parameters required for design and for verification during construction by using new approaches." The FWD is included among the ways to characterize the mechanical properties of soils. The main advantage of the FWD is the speed of testing, which will allow for a much higher number of tests to be conducted, compared to sampling for laboratory testing for example.

The moduli calculated directly from center deflection measurements for two types of in-situ plate loading field tests were essentially equivalent for embankment-type soils in Denmark (mainly cohesive moraine deposit clays), while for naturally occurring subgrades (also moraine deposit clays), the moduli derived from Static Plate Bearing Test results (rebound method) were approximately 75% of the corresponding moduli derived from FWD load-deflection tests conducted at the same points directly on the subgrade [see Reference (3)]. From the point of view of comparing a more traditional QC/QA test to FWD tests, this is certainly encouraging; however very few Static Plate Bearing tests were conducted during the LTPP program.

A Portuguese standard test protocol #06.01 [see Reference (4)] has already been developed and is in widespread use in Portugal. The procedures outlined in the reference cited (in Portuguese) show how the FWD should be used and interpreted for testing both existing pavements and new pavement construction, including tests conducted directly on unbound materials. There are also two more applicable Portuguese test methods, one for calibration (Procedure #06.02) and one that describes the FWD field test procedure for equipment operators (Procedure #06.03). One drawback, however, is that the Portuguese method is based on Heavy Weight Deflectometer (HWD) loads and test configurations, which is probably fine for bound pavement surfaces. However, these loads may be too large for the center deflection on unbound materials. Accordingly, the Portuguese procedure only utilizes the HWD sensors positioned at some distance away from the loading plate to interpret the strength of unbound materials when testing directly on the subgrade or granular base layers.

In this research, only data present in the Level E IMS, or data tables reduced from these data, were used. "Level E" data means that the data has passed all of the necessary QC checks, and is what is generally releasable to the public per request or on DataPave. The data used in the Interim Report utilized only data from DataPave 2.0, which includes data collected through early to mid-1999. For this research project, this data was supplemented by additional data uploaded to Level E after the release of DataPave 2.0 but prior to the release of DataPave 3.0.

1.4 SPS-1, -2 and -8 Experimental Design

LTPP data available from new pavement construction, where FWD deflections were measured at all, or most, layer interfaces during construction, are associated with SPS-1 (AC surfaced), SPS-2 (PCC surfaced) and SPS-8 (either AC or PCC, light traffic). A variety of thicknesses and material types were chosen by LTPP for these new construction experiments. The experimental design for these test sites is indicated in Table 1, Table 2 and Table 3. These tables list the planned layer thicknesses and material types/strengths. The actual as-built layer thicknesses and the measured material strengths and properties may differ from the planned values; the actual values were used in all calculations made in this research.

Table 1. SPS-1 Experimental Design Thicknesses and Materials

SPS-1 Experiment Number	Dense Graded Aggregate Base (inches)	Permeable Asphalt Treated Base (inches)	Dense Graded Asphalt Treated Base (inches)	Asphalt Concrete Surface Course* (inches)
01	8			7
02	12			4
03			8	4
04			12	7
05	4		4	4
06	4		8	7
07	4	4		4
08	8	4		7
09	12	4		7
10		4	4	7
11		4	8	4
12		4	12	4
13	8			4
14	12			7
15			8	7
16			12	4
17	4		4	7
18	4		8	4
19	4	4		7
20	8	4		4
21	12	4		4
22		4	4	4
23		4	8	7
24		4	12	7

* Asphalt Concrete Surface Course includes Asphalt-Bound Surface Friction Course (if any). Note: 1 inch = 25.4 mm.

Table 2. SPS-2 Experimental Design Thicknesses and Materials

SPS-2 Experiment Number	Dense Graded Aggregate Base (inches)	Permeable Asphalt Treated Base (inches)	Lean Concrete Base (inches)	PCC Surface Course (inches)
01	6			8*
02	6			8
03	6			11*
04	6			11
05			6	8*
06			6	8
07			6	11*
08			6	11
09	4	4		8*
10	4	4		8
11	4	4		11*
12	4	4		11
13	6			8*
14	6			8
15	6			11*
16	6			11
17			6	8*
18			6	8
19			6	11*
20			6	11
21	4	4		8*
22	4	4		8
23	4	4		11*
24	4	4		11

* Denotes sections with nominal 550-psi PCC; other sections have nominal 900-psi PCC. Note: 1 inch = 25.4 mm; 550 psi = 3.8 MPa; 900 psi = 6.2 MPa.

Table 3. SPS-8 Experimental Design Thicknesses and Materials

SPS-8 Experiment Number	Dense Graded Aggregate Base (inches)	PCC Surface Course (inches)	Asphalt Concrete Surface Course* (inches)
01	8		4
02	12		7
03	8		4
04	12		7
05	8		4
06	12		7
07	6	8	
08	6	11	
09	6	8	
10	6	11	
11	6	8	
12	6	11	

* Asphalt Concrete Surface Course includes Asphalt-Bound Surface Friction Course (if any). Note: 1 inch = 25.4 mm.

1.5 Units Used and LTPP Definitions of Terms

For ease of reading and to facilitate understanding in this report, wherever possible SI units have been used to express the results of, and relationships between, the various material properties reported. However in certain instances, U.S. Customary units were retained when the data or charts used already were created in these units (see for example Tables 1-3).

The following abbreviations were used in this report:

- Lane 1 = A line between the wheel paths.
- Lane 3 = A line approximately in the right-hand wheel path.
- Lane S1 = Tests conducted along Lane 1 on subgrade.
- Lane G3 = Tests conducted along Lane 3 on granular base.
- Lane P1 = Tests conducted along Lane 1 on permeable asphalt base.
- Lane F3 = Tests conducted along Lane 3 on asphalt concrete surface course.
- Lane J1 = Tests conducted at center slab positions along Lane 1on jointed PCC.
- Section 39-01xx = SPS-1 (AC) Section xx, in State 39 (Ohio).
- Section 04-02yy = SPS-2 (PCC) Section yy, in State 04 (Arizona).
- Section 39-0101 = SPS-1 (AC) Section 01, in State 39 (Ohio).
- Section 04-0213 = SPS-2 (PCC) Section 13, in State 04 (Arizona).

- SPS-1 Site = Usually 12 test sections, numbered 0101-0112 or 0113-0124.
- SPS-2 Site = Usually 12 test sections, numbered 0201-0212 or 0213-0224.
- SPS-8 Site = Usually 2 test sections, numbered consecutively (AC or PCC).

Other abbreviations that are used in this report include:

- AC = Asphalt Concrete (surface course).
- PCC = Portland Cement Concrete.
- LCB = Lean Concrete Base.
- JCP = Jointed PCC Pavement.
- DGAB = Dense Graded Aggregate Base (unbound).
- PATB = Permeable Asphalt Treated Base (open-graded, dissimilar to the AC surface course).
- ATB = Asphalt Treated Base (dense graded, generally similar to the AC surface course).

All data reported herein are based on LTPP's protocol sensor spacings of 0, 8, 12, 18, 24, 36 and 60 inches (0, 203, 305, 457, 610, 915 & 1524 mm), respectively. In the very few cases in the SPS-1, -2 and -8 database where sensors were, in all likelihood, not positioned according to LTPP's normal protocol, these sensor positions were corrected for use in this research. The corrected sensor positions were determined as part of an FHWA-funded study of LTPP deflection data (presently pending publication).

1.6 Report Organization

The data and analyses presented in the following sections are organized as follows:

Following this introductory chapter, Chapter 2 presents the use of specific pavement test parameters from DataPave and the National Information Management System (IMS) database that can be used to characterize pavement construction quality. These include both the FWD deflection data, and analyses thereof, and traditional construction quality data categories.

Chapter 3 covers the relationships between FWD-associated data and conventional QC/QA data types. Relationships between all of these data are developed, discussed and plotted.

Chapter 4 consists of the conclusions and research recommendations in relation to the research performed herein.

The last page of the report is a list of references.

CHAPTER 2—THE USE OF VARIOUS TEST PARAMETERS TO CHARACTERIZE PAVEMENT CONSTRUCTION QUALITY

2.1 Pavement Construction Quality Data Used in this Research

Traditional pavement construction quality control procedures include a variety of laboratory and in-situ sampling and test methods, depending on the type of material in any given pavement layer, from the subgrade (whether natural or embankment) upwards to the surface course. In addition, FWD load-deflection data was added to the potential methods for characterizing the quality of the pavement under, or immediately after, construction.

2.1.1 Unbound Material Density, Moisture, Proctor and Other Test Data

The most commonly used methods of quality control during construction of unbound materials in pavements are material densities and moisture contents. Most agencies include standard specifications on compaction to achieve certain densities and optimum moisture contents to assist in achieving these densities.

The procedure generally starts with a laboratory-specified density and moisture test called a "Proctor". Both standard and modified Proctors are employed, depending on the jurisdiction and the material type, or layer. Briefly, a Proctor curve represents the dry density of a given material versus its moisture content. The moisture content where the maximum dry density is achieved using a prescribed compactive effort in a Proctor test mold defines that material's maximum density and optimum moisture content.

Subsequently in the field, the contractor is supposed to achieve a predetermined degree of compaction compared to these maximum and optimum values. Typical specified values are 95% for subgrades and 98% for granular bases; however, these are not universal and also the specified degree of compaction required depends on whether a Standard or Modified Proctor procedure was used to quantify the benchmark density and moisture content. For most of the SPS-1, -2 and -8 experiments, both the Proctor "maximum" densities and the in-situ field densities are recorded and stored in the IMS, for each unbound layer.

An additional test that was carried out on select unbound materials was the resilient modulus test, which was performed on both disturbed and undisturbed subgrade and granular base material types. There are a limited number of cases in the IMS where both P46 lab moduli and direct FWD tests on the unbound materials are available from the same test section.

2.1.2 FWD Deflection Testing on Unbound Pavement Layers

During construction of the SPS-1, -2 and -8 experimental sections, FWD load-deflection tests were generally conducted at each unbound layer interface along two test lanes: S1 and S3 for the subgrade layer, and G1 and G3 for the granular base layer (where present). In most instances, the test spacing along each lane of testing was 50 feet (15.24 m). For unbound layers, two drops per drop height and up to four consecutive drop heights were employed, per station (for bound layer tests, four drops per drop height and up to four consecutive drop heights, were employed). Since most sections are 500 feet (152.4 m) in length, this usually resulted in 10 or 11 test points, per lane, per section, and per layer interface.

In a previous LTPP study [pending publication], most of the IMS Level-E FWD data on bound layers were screened, normalized and averaged for easy use. In addition, the unbound layer data were screened for use as part of this research project.

When FWD tests are carried out on unbound materials, it is generally desirable to achieve a plate pressure roughly the same as that expected under heavy axle loads passing over the finished pavement section. With the FWD, a lighter load package and a larger loading plate are oftentimes used to achieve this goal. In the LTPP database, the large 450 mm (~18") diameter loading plate was used in one instance. In other cases, the 300 mm (~12") standard loading plate was left on the FWD for unbound layer testing. The normal protocol sensor positions were used throughout most of the entire SPS-1, -2 and -8 database. In most instances a smaller weight (mass) package was used when testing unbound materials, which did in fact help to achieve somewhat lower plate pressures.

For drop height one (the lowest possible drop height and resulting plate pressure), this generally resulted in pressures of some 20- to 30-psi (140-200 kPa) on the subgrade, and somewhat more on the granular base (due to larger weight packages used when that layer was tested). These pressure levels were, in fact, still somewhat high—considering that even a high 700-kPa (100-psi) tire pressure will be drastically reduced once the pressure bulb emanating from the surface reaches the base or subgrade. However, drop height one FWD plate pressures most often resulted in reasonable deflection basins. The data obtained at the lowest drop height were generally consistent from drop to drop, and with decreasing deflections from the center sensor outwards, as expected.

2.1.3 Testing and Properties of Unbound Materials – Precautions

Naturally, during construction and prior to the placement of any bound layer(s), changing weather conditions can affect the in-situ properties of the materials. For example, immediately after grading and compaction of a subgrade surface, precipitation may occur, thus affecting the in-situ readings from any given in-situ test or sampling method. In the opposite direction, if a given unbound layer “dries out” after compaction, it may either strengthen the material if soil suction increases its tensile strength or weaken the material if it becomes too “loose”, like dry sand.

Therefore, the timing of the tests used on unbound materials, the timing of the construction activity, the ambient weather conditions prevailing over the entire duration of time from the beginning of construction to actual in-situ testing and sampling, and placement of the subsequent layer(s) can affect the test results as well as the ultimate performance of the pavement section.

With these confounding effects in mind, we can nevertheless look at overall trends and relationships, and anticipate that the general correlations may be somewhat weaker but none-the-less valid.

2.1.4 Bound Material Thickness and Strength Parameters

In the case of asphalt- and cement-bound materials, very useful data exists in the LTPP database that was gathered during construction of the SPS-1, -2 and -8 test sites. One of the useful data elements is layer thickness; these data were usually gathered through the use of "grid" elevation measurements, taken layer by layer throughout the construction process. In this research, the average of these (usually 33) grid thickness measurements taken along or near the lanes tested was used to describe the thickness of each structural layer, at each test section.

Select "strength" parameters associated with the bound layers were also measured and stored in the IMS database, generally based on laboratory tests carried out on field samples. Some of these involved cores, while others were reconstituted samples, such as PCC cylinders and beams. From cores, elastic moduli were measured in some cases, both for AC (resilient modulus) and PCC (static modulus) surface types. Most of the measured strength parameters involved portland cement concrete surfaced pavements, including lean concrete base (where present). Cores from both AC and PCC pavement types, however, were occasionally used to measure modulus of elasticity in the laboratory.

Other potentially useful in-situ parameters included AC temperature at the time of FWD testing (computed parameters in the IMS) and backcalculated modulus of bound surface layers (also computed parameters).

2.1.5 FWD Deflection Testing on Bound Pavement Layers

During and shortly after construction of the SPS-1, -2 and -8 experimental sections, FWD load-deflection tests were carried out on the surface layer, and occasionally at an intermediate bound-base layer interface. For AC surfaced pavements or layers, FWD testing was conducted along two test lanes: F1 and F3, similar to the tests conducted for the unbound layers. For PCC surfaced pavements, FWD tests were carried out at joints, corners, etc., and also at the interior slab position, denoted Lane J1. Lane J1 data was the only PCC data from FWD testing that was used in this research. Similar tests were occasionally carried out along Lanes P1 and P3 of the PATB layer or along Lane L1 and L3 of the LCB layer, if present. As was the case with FWD tests on unbound materials, Lane 1 tests represent the line *between* the two wheel paths, while Lane 3 tests represent the line in the right-hand or outside wheel path of the finished pavement section.

For bound pavement layers, four FWD drops per drop height, for up to four consecutive drop heights, were employed, per station. Since most sections are 500 feet (15.24 m) in length, with tests every 50 feet, there usually were 11 test points, per lane, per section, and per layer interface. The exception to this was the Lane J1 tests, where interior slab tests were conducted. Accordingly, test intervals of exactly 50 feet (~15m) could not be achieved. In a previous study (pending publication), most of the IMS Level-E FWD data on bound layers were screened, normalized to the target load level, and averaged for each drop height, for easy use. These data, where available, were used in this research.

2.1.6 FWD Data Available from SPS-1, -2 and -8 Sites

The FWD database chosen for this research consists of all SPS new construction sites where FWD testing was conducted on at least one of the unbound layers, as well as testing on the surface course and, occasionally, any bound base course as well. In this manner it was possible to compare the FWD test results from one new construction layer to the next, and also to compare the results to the results of other quality control tests run on the various materials during pavement construction.

A list of SPS sites (with several test sections represented within each site) utilized in the study is shown in Table 4. Three of LTPP's four regions are represented, covering a variety of climatic zones within the USA. Specific climatic zone information associated with each SPS site may be found in the current version of DataPave. Table 5 provides a detailed list of all selected SPS test sections and the various layers tested using the FWD within these sections.

Table 4. Selected SPS Sites with Adequate FWD Test Results used in Research Study

State (State Code)	LTPP Region	Pavement Surface (SPS Experiment)
Kansas (20)	2	AC (01)
Ohio (39)	2	AC (01)
Ohio (39)	2	PCC (02)
Arkansas (05)	3	AC (01)
Louisiana (22)	3	AC (01)
Mississippi (28)	3	AC (08)
Arizona (04)	4	AC (01)
Arizona (04)	4	PCC (02)
Colorado (08)	4	PCC (02)
Montana (30)	4	AC (01)
Montana (30)	4	AC (08)
Nevada (32)	4	AC (01)
Nevada (32)	4	PCC (02)
Utah (49)	4	AC (08)
Washington (53)	4	PCC (02)
Washington (53)	4	AC (08)

Table 5. Selected SPS Sections with Adequate FWD Results used in Research Study

State ID	Section ID	FWD Test Dates for Structural Layers					
		Subgrade	DGAB	PATB	LCB	AC	PCC
4	113	08-Jun-93	30-Jun-93	06-Aug-93[1]		17-Feb-94	
4	114	07-Jun-93	17-Jun-93			16-Feb-94	
4	115	08-Jun-93		22-Jul-93[1]		16-Feb-94	
4	116	07-Jun-93		26-Jul-93[1]		17-Feb-94	
4	117	08-Jun-93	17-Jun-93			16-Feb-94	
4	118	07-Jun-93	17-Jun-93	26-Jul-93[1]		17-Feb-94	
4	119	09-Jun-93	17-Jun-93	No Data		16-Feb-94	
4	120	08-Jun-93	30-Jun-93	No Data		17-Feb-94	
4	121	No Data	30-Jun-93	06-Aug-93		17-Feb-94	
4	122	07-Jun-93		26-Jul-93		17-Feb-94	
4	123	09-Jun-93		23-Jul-93		16-Feb-94	
4	124	09-Jun-93		23-Jul-93		16-Feb-94	
4	213	29-Jul-93	11-Aug-93				08-Feb-94
4	214	28-Jul-93	10-Aug-93				31-Jan-94
4	215	29-Jul-93	10-Aug-93				02-Mar-95
4	216	29-Jul-93	10-Aug-93				02-Feb-94
4	217	29-Jul-93			09-Sep-93		07-Feb-94
4	218	28-Jul-93			09-Sep-93		01-Feb-94
4	219	29-Jul-93			09-Sep-93		03-Feb-94
4	220	28-Jul-93			09-Sep-93		01-Feb-94
4	221	30-Jul-93	11-Aug-93	No Data			08-Feb-94
4	222	28-Jul-93	10-Aug-93	No Data			31-Jan-94
4	223	29-Jul-93	11-Aug-93	No Data			03-Mar-95
4	224	28-Jul-93	10-Aug-93	No Data			01-Mar-95
5	113	28-Jul-93	30-Sep-93			17-Mar-94	
5	114	28-Jul-93	30-Sep-93			16-Mar-94	
5	115	28-Jul-93				16-Mar-94	
5	116	28-Jul-93				16-Mar-94	
5	117	28-Jul-93	30-Sep-93			16-Mar-94	
5	118	28-Jul-93	30-Sep-93			16-Mar-94	
5	119	27-Jul-93	30-Sep-93	No Data		15-Mar-94	
5	120	27-Jul-93	01-Oct-93	No Data		15-Mar-94	
5	121	27-Jul-93	01-Oct-93	No Data		15-Mar-94	
5	122	27-Jul-93		No Data		15-Mar-94	
5	123	28-Jul-93		No Data		15-Mar-94	
5	124	28-Jul-93		No Data		15-Mar-94	
8	213	05-Oct-93	10-Oct-93				30-Mar-94
8	214	07-Oct-93	11-Oct-93				30-Mar-94

[1] May have been ATB tests – not a PATB section.

State ID	Section ID	FWD Test Dates for Structural Layers (continued)					
		Subgrade	DGAB	PATB	LCB	AC	PCC
8	215	07-Oct-93	11-Oct-93				01-Apr-94
8	216	05-Oct-93	08-Oct-93				30-Mar-94
8	217	No Data			No Data		N/A
8	218	13-Oct-93			No Data		31-Mar-94
8	219	13-Oct-93			No Data		31-Mar-94
8	220	No Data			No Data		N/A
8	221	04-Aug-93	No Data	No Data			01-Apr-94
8	222	04-Aug-93	No Data	No Data			29-Mar-94
8	223	03-Aug-93	No Data	No Data			29-Mar-94
8	224	09-Aug-93	No Data	No Data			25-Mar-94
20	101	No Data	[30-Jun-93][3]			27-Oct-93	
20	102	No Data	20-Jul-93			27-Oct-93	
20	103	[29-Jun-93]				28-Oct-93	
20	104	[29-Jun-93] & 20-Jul-93				28-Oct-93	
20	105	[29-Jun-93]	21-Jul-93			28-Oct-93	
20	106	No Data	20-Jul-93			27-Oct-93	
20	107	[29-Jun-93]	No Data	No Data		27-Oct-93	
20	108	[29-Jun-93]	20-Jul-93	No Data		27-Oct-93	
20	109	[29-Jun-93]	20-Jul-93	No Data		27-Oct-93	
20	110	[29-Jun-93] & 21-Jul-93		No Data		27-Oct-93	
20	111	[29-Jun-93] & 21-Jul-93		No Data		28-Oct-93	
20	112	[29-Jun-93] & 21-Jul-93		No Data		28-Oct-93	
22	113	24-Apr-96	16-May-96			21-Sep-98	
22	114	24-Apr-96	16-May-96			21-Sep-98	
22	115	24-Apr-96				18-Sep-98	
22	116	24-Apr-96				18-Sep-98	
22	117	24-Apr-96[2]	16-May-96			21-Sep-98	
22	118	23-Apr-96[2]	16-May-96			18-Sep-98	
22	119	24-Apr-96	21-May-96	No Data		17-Sep-98	
22	120	07-May-96	21-May-96	No Data		17-Sep-98	
22	121	06-May-96[2]	No Data	No Data		17-Sep-98	
22	122	06-May-96		No Data		17-Sep-98	
22	123	07-May-96		No Data		17-Sep-98	
22	124	07-May-96		No Data		18-Sep-98	
28	805	17-Sep-96	No Data			21-Apr-97	
28	806	17-Sep-96	No Data			21-Apr-97	
30	113	No Data	21-Aug-98			10-Nov-98	
30	114	No Data	23-Aug-98			11-Nov-98	

[2] Probably Granular Base (G-) tests, although labeled S-.

State ID	Section ID	FWD Test Dates for Structural Layers (continued)					
		Subgrade	DGAB	PATB	LCB	AC	PCC
30	115	No Data	22-Aug-98[3]			11-Nov-98	
30	116	No Data	22-Aug-98[3]			11-Nov-98	
30	117	No Data	22-Aug-98			11-Nov-98	
30	118	No Data	22-Aug-98			10-Nov-98	
30	119	02-Jul-98	23-Aug-98	No Data		12-Nov-98	
30	120	No Data	23-Aug-98	No Data		12-Nov-98	
30	121	02-Jul-98	23-Aug-98	No Data		12-Nov-98	
30	122	02-Jul-98	23-Aug-98[3]	No Data		12-Nov-98	
30	123	No Data		No Data		11-Nov-98	
30	124	No Data	23-Aug-98[3]	No Data		11-Nov-98	
30	805	21-Apr-94	03-Jun-94			23-Aug-94	
30	806	21-Apr-94	03-Jun-94			23-Aug-94	
32	101	15-May-95 & 6-Jul-95	12-Jul-95			27-Mar-96	
32	102	17-May-95 & 11-Jul-95	24-Jul-95			04-Apr-96	
32	103	17-May-95 & 11-Jul-95				04-Apr-96	
32	104	16-May-95 & 6-Jul-95				27-Mar-96	
32	105	17-May-95 & 11-Jul-95	20-Jul-95			04-Apr-96	
32	106	16-May-95 & 6-Jul-95	14-Jul-95			02-Apr-96	
32	107	16-May-95 & 10-Jul-95	18-Jul-95	No Data		03-Apr-96	
32	108	16-May-95 & 10-Jul-95	18-Jul-95	No Data		03-Apr-96	
32	109	16-May-95 & 6-Jul-95	18-Jul-95	No Data		03-Apr-96	
32	110	16-May-95		22-Aug-95		03-Apr-96	
32	111	16-May-95 & 11-Jul-95		25-Aug-95		25-Aug-95	
32	112	16-May-95 & 11-Jul-95		25-Aug-95		03-Apr-96	
32	201	25-Apr-95 & 14-Jun-95	28-Jun-95				25-Mar-96
32	202	12-May-95 & 20-Jun-95 & 28-Jun-95[2]	No Data				01-Apr-96
32	203	12-May-95 & 21-Jun-95	No Data				26-Mar-96
32	204	12-May-95 & 20-Jun-95 & 28-Jun-95[2]	No Data				27-Mar-96

[3] Probably Subgrade (S-) tests, although labeled G-.

	Section	FWD Test Dates for Structural Layers (continued)					
State ID	ID	Subgrade	DGAB	PATB	LCB	AC	PCC
32	205	25-Apr-95 & 14-Jun-95 & 21-Jun-95			No Data		25-Mar-96
32	206	15-May-95 & 30-Jun-95			No Data		02-Apr-96
32	207	21-Jun-95			No Data		27-Mar-96
32	208	11-May-95 & 21-Jun-95			No Data		27-Mar-96
32	209	25-Apr-95	28-Jun-95	No Data			26-Mar-96
32	210	12-May-95 & 20-Jun-95 & 28-Jun-95[2]	No Data	No Data			27-Mar-96
32	211	11-May-95 & 15-Jun-95 & 21-Jun-95	28-Jun-95	No Data			26-Mar-96
32	212	No Data	No Data	No Data			No Data
39	101	29-Aug-95	12-Sep-95			05-Nov-96	
39	102	29-Aug-95	12-Sep-95			22-Apr-96	
39	103	24-Aug-95				04-Nov-96	
39	104	19-Jul-95				05-Nov-96	
39	105	No Data	No Data			05-Nov-96	
39	106	01-Aug-95	17-Oct-95			05-Nov-96	
39	107	29-Aug-95	12-Sep-95	19-Oct-95		No Data	
39	108	28-Aug-95	05-Oct-95	No Data?[4]		04-Nov-96	
39	109	25-Aug-95	11-Sep-95	20-Sep-95		04-Nov-96	
39	110	25-Aug-95		17-Sep-95		04-Nov-96	
39	111	19-Jul-95		30-Aug-95		06-Nov-96	
39	112	19-Jul-95		30-Aug-95		06-Nov-96	
39	201	01-Aug-95	18-Oct-95				31-Dec-96
39	202	10&11-Jul-95	05-Sep-95				15-Dec-96
39	203	22-Aug-95	05-Sep-95				03-Jan-97
39	204	26-Jun-95	17-Aug-95				02-May-96
39	205	19-Jul-95			29-Aug-95		30-Dec-96
39	206	19-Jul-95			29-Aug-95		30-Dec-96
39	207	23-Aug-95			17-Oct-95		04-Jan-97
39	208	23-Aug-95			17-Oct-95		04-Jan-97
39	209	23-Aug-95	11-Sep-95	02-Oct-95			31-Dec-96
39	210	26-Jun-95 & 10-Jul-95	17-Aug-95	28-Aug-95			14-Dec-96
39	211	23-Aug-95	18-Sep-95	22-Sep-95			02-Jan-97
39	212	26-Jun-95 & 17-Aug-95[5]	17-Aug-95	28-Aug-95			14-Dec-96
49	803	12-Aug-96	18-Jul-97			31-Oct-97	

[4] Lane F3 tests are listed for 29-Jul-96; these were probably PATB tests.
[5] Probably incorrect date; should be 26-Jun-95?

State ID	Section ID	FWD Test Dates for Structural Layers (continued)					
		Subgrade	DGAB	PATB	LCB	AC	PCC
49	804	12-Aug-96	18-Jul-97			31-Oct-97	
53	201	08-Jul-95	20-Aug-95				17-Nov-95
53	202	07-Jul-95	20-Aug-95				15-Nov-95
53	203	07-Jul-95	20-Aug-95				14-Nov-95
53	204	08-Jul-95	20-Aug-95				17-Nov-95
53	205	08-Jul-95			No Data		17-Nov-95
53	206	07-Jul-95			No Data		19-Nov-95
53	207	07-Jul-95			No Data		19-Nov-95
53	208	08-Jul-95			No Data		18-Nov-95
53	209	07-Jul-95	20-Aug-95	No Data			16-Nov-95
53	210	07-Jul-95	20-Aug-95	No Data			15-Nov-95
53	211	07-Jul-95	20-Aug-95	No Data			16-Nov-95
53	212	07-Jul-95	20-Aug-95	No Data			16-Nov-95
53	801	22-Aug-95	No Data			13-Nov-95	
53	802	22-Aug-95	No Data			13-Nov-95	

[…] Not used: data gathered using the 450 mm FWD plate size (data not extensive enough to be useful).

As indicated in Table 4, a total of 16 SPS sites provided adequate FWD load-deflection data, from a variety of climatic zones, for use in the analysis database. Please note that in Table 5, the footnotes and brackets indicate the data that was either not used or was changed for various reasons, as stated. Please note as well that some of the footnotes refer to more than one page of this 5-page table.

2.2 Analysis of SPS-1, -2 and -8 Deflection Data

2.2.1 Example of FWD Center Deflections for an SPS-1 Site

A clearer indication of the general deflection levels encountered on SPS-1 or -2 sites, with generally 12 contiguous sections per site, is shown in Figure 1. Only the center deflection is plotted, with averages for each section shown on the same graph. This example SPS-1 site is from Ohio (State 39). In this case, the measured deflections have been converted from the SI units used in the IMS (microns) to U.S. Customary units (mils).

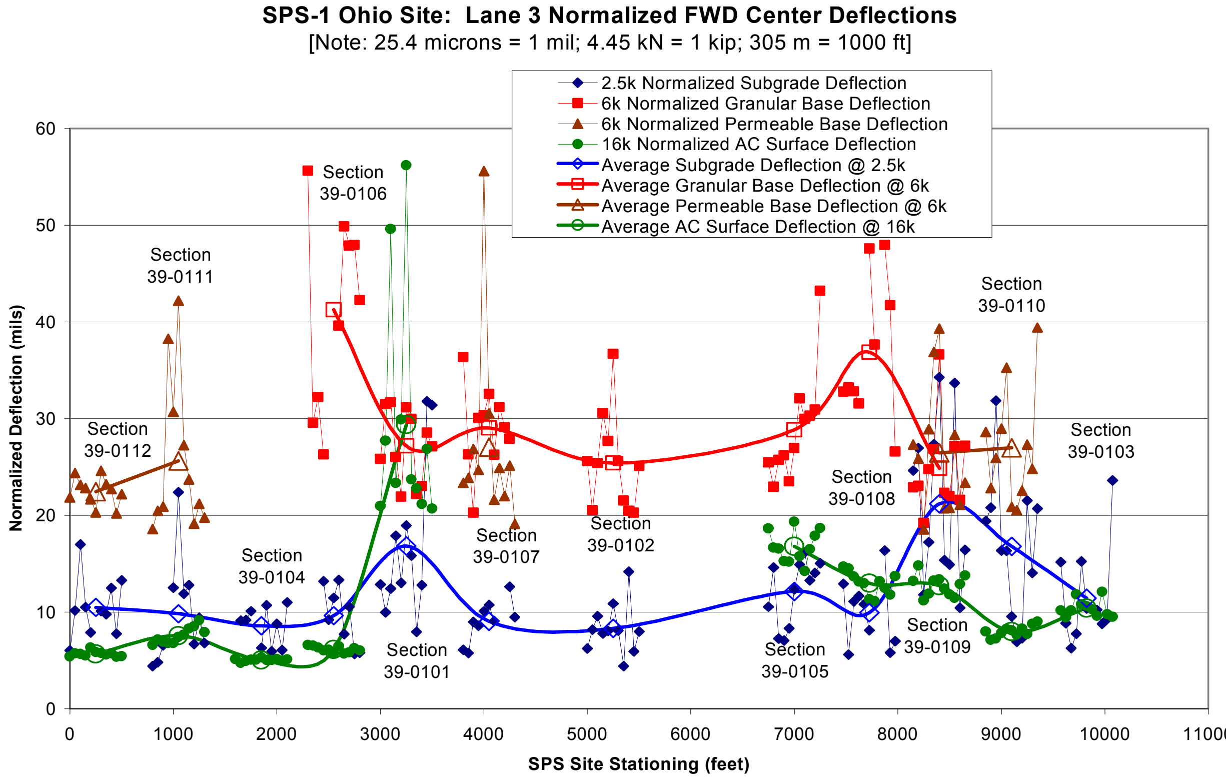

Figure 1. Composite Pavement Layer Deflections for SPS-1 Site in Ohio

In order to view the data on the same vertical scale in Figure 1, it was necessary to scale, or "normalize", the plotted FWD center deflections. Normalizing the FWD deflection data means that the deflections have been scaled up or down to a specific target load. For example, if the actual FWD load level was 10% higher than the targeted, the plotted deflection was reduced (or divided) by a factor of 1.10. Accordingly, the following normalized FWD load levels were utilized in the preparation of Figure 1:

- Subgrade Layer Tests 2,500 lbs (11 kN)
- Granular or Permeable Base Layer Tests 6,000 lbs (27 kN)
- Asphalt-Bound Surface Layer Tests 16,000 lbs (71 kN)

On the horizontal scale, the SPS site (not section) stationing is plotted. As can be seen, there are varying distances between the twelve SPS-1 sections shown (500 feet or 15.24 m in length for each). Also as can be seen, the overall pattern of deflections carries through (is roughly parallel) to the following layer(s), from the subgrade upwards — notice the shape of the best-fit section averages from layer to layer. Section 39-0101 is a clear exception to this, however.

Figure 1 also shows that there are two complete sets of data that stand out among all the rest: Section 39-0101, which is one of the thicker AC sections (nominally 7" AC over 8" DGAB), showed very high AC surface deflections, even though the subgrade and base deflections were *not* abnormally high. Section 39-0105 (nominal 4" AC over 4" ATB over 4" DGAB) also showed fairly high AC surface deflections, in spite of similarly reasonable subgrade and base deflections. These deflections levels alone would indicate that the AC surface in Section 39-

0101 is probably substandard in one way or another. It should also be noted that no surface course deflections were taken after these sections were opened to traffic (on 15 August 1996) for the two succeeding sections along the stationing of this SPS-1 site, i.e. 39-0107 and 39-0102.

It was subsequently discovered that Sections 30-0107 and -0102 had in fact failed prematurely. The asphalt layers on these two sections were removed and replaced (with thicker AC sections) prior to the date FWD surface course tests were scheduled and soon after construction was completed. After FWD tests on the AC surface of the remaining sections were conducted, about a month later, it was also necessary to remove and reconstruct Section 39-0101—for essentially the same reasons. Reportedly, the bond between the various lifts of ATB and/or AC was poor, and some rutting and other surface distress had already occurred, even prior to the first winter of service. Finally about two years later, it was also necessary to remove and replace Section 39-0105, once again for much the same reasons—in this case, rutting and cracking. At this time (2001), all four of these test sections have been taken out of the SPS-1 experimental design matrix due to these premature pavement failures. The first two had failed even before FWD tests were conducted, while the last two failed in inverse order of the average magnitude of the FWD deflections conducted on the surface course. In all likelihood, all four failures could have been predicted by FWD deflections alone, even though the subgrade and base course tests appeared quite normal.

Sections 39-0106 and 39-0108, which are among the thicker AC sections, both showed fairly large deflections on the aggregate base. These somewhat large base course deflections, however, did not seem to "carry through" to the rather thick surface courses subsequently placed. A couple of other sections at this SPS-1 site showed somewhat large subgrade deflections as well. Based on performance to-date, in fact, it appears that these sections have a good AC layer and are still performing adequately, after about five years of service.

2.2.2 Other Direct Uses of Load-Normalized FWD Deflections

FWD load-deflection data were used in a global (meaning LTPP-wide) manner, using all available test loads and deflections, where other pavement construction quality test results were also available, to show promising trends or relationships. These data will be presented in subsequent sections in this report, mainly in *Chapter 3—Relationships Between FWD-Derived Parameters and Traditional Pavement Construction Quality Data.*

2.2.3 Deflection-Derived Layer Quality Characterization

Apart from the deflection values themselves, it is possible to calculate approximate moduli, or stiffnesses. The "composite" layer modulus, based on FWD tests conducted directly on any given pavement layer, may be calculated from Equation (1) [see e.g. Reference (5) in the textbook by Ullidtz, *Pavement Analysis,* on Page 33]:

$$E_o = (1.5 \bullet a \bullet \sigma_o) / d_o \qquad (1)$$

where: E_o = "surface" or composite modulus of the subgrade beneath the loading plate;
a = radius of FWD loading plate;
σ_o = (peak) pressure of FWD impact load under loading plate;
and d_o = (peak) center FWD deflection reading.

Equation (1) is the most commonly used version. It is based on an evenly distributed and uniform FWD load, and a Poisson's ratio of 0.5. Generally, Poisson's ratio will be less than 0.5

(usually thought to be between 0.35 and 0.45 for most unbound materials), while the distribution of the load under the FWD plate will not be exactly uniform (rather it will be somewhat non-uniform due to the rigidity of the loading plate). These two counteracting factors have a tendency to offset one another; hence the simple "1.5 times" composite modulus formula is the one most often used in practice and, also, in this research.

The term "surface" or composite modulus is also used to define the overall (or apparent) modulus of the subgrade layer beneath some depth and under the sensor used in the calculations. This depth is approximately equal to the distance the deflection sensor is placed from the center of the FWD load plate. The apparent or composite subgrade modulus derived from any FWD sensor at offset "r" may be calculated from Equation (2) [from Reference (5)]:

$$E_{o,r} = (0.84 \bullet a^2 \bullet \sigma_o) / (d_r \bullet r) \tag{2}$$

where: $E_{o,r}$ = "surface" or composite modulus of the subgrade beneath the sensor used;
a = radius of FWD loading plate;
σ_o = (peak) pressure of FWD impact load under loading plate;
d_r = (peak) FWD deflection reading at offset distance "r";
and r = distance of deflection reading d_r from center of loading plate.

The suggested constant of 0.84 assumes that Poisson's ratio is 0.4 (from the calculation $1 - \mu^2$). If d_r is a reasonably large distance from the edge of the loading plate, the load may be assumed to be a "point" load, so the plate pressure distribution does not matter. Furthermore, small changes in Poisson's ratio have only minimal impact on Equation (2).

Both Equations (1) and (2) were derived from Boussinesq's equations for a linear-elastic, semi-infinite half space. In light of the limitations mentioned, plus the fact that unbound (or any other) materials are not perfectly linear-elastic to infinite depths, these equations certainly are not perfect; however, they are very easy to use and understand. They are "forward" calculating, so the results are unique to the input variables (as opposed to backcalculation results, which are non-unique).

When dealing with a bound layer, the use of composite moduli using Equations (1) and (2) becomes less relevant, since there are several layers with appreciably different stiffnesses that make up the "composite" moduli calculated using the Boussinesq relationships. In most cases, there is a substantially stiffer layer above the subgrade and base layer(s). Thus the underlying materials would confound any composite modulus or stiffness calculation that was designed to ascertain the quality of the bound layer. As a result, in some cases backcalculation is used to derive the layered elastic response of a mutilayer system with a bound upper-most layer.

A relatively small percentage of backcalculated values for the SPS-1, -2 and -8 sites are populated with backcalculated moduli in the IMS. Nevertheless, if the FWD is to be a serious candidate as a potential QC/QA device during pavement construction, backcalculation may not be the most desirable approach, since it is both excessively time consuming and generally produces non-unique solutions. One backcalculation program does not necessarily produce the same results as the next (usually not, in fact), and the process of backcalculation is more an engineering "art" than a "science".

A new method of determining the effective stiffness of the upper (bound) layer in the case of new construction was therefore developed for use during Phase I of this research. The proposed method is forward calculating (and therefore produces unique values). It is based on the AREA concept (a deflection basin curvature index) and the overall composite modulus of the entire pavement structure, E_o, as previously defined by Equation (1).

The "AREA", for example as reported in the AASHTO Design Guide (6), is calculated as:

$$A = 6 * [1 + 2(d_{1'}/d_0)+2(d_{2'}/d_0)+ (d_{3'}/d_0)] \tag{3}$$

where: A = The "AREA" beneath the first 3 feet (914 mm) of the deflection basin;
d_0 = FWD deflection measured at the center of the FWD load plate;
$d_{1'}$ = FWD deflection measured one foot from the center of the plate;
$d_{2'}$ = FWD deflection measured two feet from the center of the plate;
and $d_{3'}$ = FWD deflection measured three feet from the center of the plate.

When calculating AREA, the diameter of the loading plate must be between 300 mm (11.8") and 305 mm (12"). An AREA calculation of 36 is achieved if all four deflection readings, at the 0-, 1-, 2- and 3-foot (0-, 305-, 610- and 914-mm) offsets, are identical, which is tantamount to an infinitely stiff upper layer.

A series of forward calculations were made to see what the AREA term becomes if *all* layers in a multilayered elastic system have identical stiffnesses or moduli (and Poisson's ratios). This can be carried out using, for example, the ELSYM5 or BISAR multilayered elastic programs. It turns out that, no matter which modulus value is selected, as long as all of the layers are assigned the same identical modulus of elasticity, the AREA term is always equal to 11.037. It therefore follows that if FWD deflection measurements carried out in the field result in this particular value for the AREA term, then the effective modulus, or stiffness, of the upper layer is identical to that of the underlying layer(s).

This number is important in the following two equations, because it now can be used to ascertain whether the upper layer has a higher stiffness than the underlying layer(s). If the AREA term is much larger than 11.037, for example, then the upper (bound) layer is appreciably stiffer than the underlying (unbound) layer(s). The value 11.037 is therefore used in Equation (4), below, while Equation (3) is tantamount to a "radius of curvature" index, based on the stiffness of the upper bound layer compared to the composite stiffness of the underlying unbound layers.

The calculation of E_o was explained in connection with the presentation of Equation (1), above. Once again, this value is a composite, effective stiffness of *all* the layers under the FWD loading plate. If these two terms are combined such that the boundary conditions are correct and the logic of the AREA concept is adhered to, the following equations result:

$$AF = [(k_2 - 1)/\{k_2 - (AREA/k_1)\}]^2 \tag{4}$$

where: AF = AREA factor, i.e. the "improvement" in AREA from 11.037 squared;
k_1 = 11.037 (the AREA when the stiffness of the upper layer is the same as that of the lower layers);
k_2 = 3.262 (maximum possible improvement in AREA = 36/11.037).

$$ES = [E_o * AF * k_3^{\{(1/AF) - 1\}}] \tag{5}$$

where: ES = Effective Stiffness of upper (bound) layer;
E_o = as defined by Equation (1);

AF = as defined by Equation (4);

k_3 = thickness of upper layer / load plate diameter = $h_1/(2*a)$;

and a = radius of FWD load plate.

Although Equation (5) has not been tried extensively or independently verified, for the purposes at hand it can be effectively used to *approximate* the relative stiffness of the upper (bound) layer in a pavement cross section. The advantage of using Equations (4) and (5), or similar equations developed elsewhere, is that forward-calculation techniques together with commonly used deflection-based quantities (such as AREA) can be employed. Only the composite modulus or stiffness of the pavement system, the AREA, and the pavement thickness normalized to the diameter of the loading plate, are needed to calculate the relative stiffness of the bound upper pavement layer. Elsewhere in this report, the forwarded-calculation method of deriving relative stiffnesses through Equations (4) and (5) is referred to as the AREA method.

An alternate method of calculating the stiffness or apparent modulus of PCC layers was also investigated. This method was proposed by Hossain and Yang [see Reference (7)] in a publication entitled *"Determination of Concrete Pavement Surface Layer Modulus and Thickness Using Deflection Testing."*

$$E_{PCC} = 2 * (1 - 0.15^2) * a * \sigma_o / (d1 - d2) \qquad (6)$$

where: E_{PCC} = stiffness, or modulus, of PCC layer;

a = radius of FWD load plate;

σ_o = (peak) pressure of FWD impact load under load plate;

d_1 = FWD deflection measured at the center of the plate;

and d_2 = FWD deflection measured at a distance of 8" (203 mm) from center of load plate.

Equation (6) however is limited to PCC layers around 11" thick. A comparison of the results of Equations (5) and (6) for interior slab deflections of the thicker (nominal 11") PCC sections is plotted in Figure 2. Both the thicker best-fit correlation line and the thinner (and longer) line of equality are shown in the figure.

The results shown in Figure 2 indicate that the relative or effective stiffnesses obtained through each method are well correlated and, in fact, quite close to the line of equality. However the values obtained through the AREA method were selected for the rest of the analyses in this research, because the standard deviation of all the values used in the comparison was smaller. Given that these values were obtained from new PCC pavement construction, it is expected that the stiffnesses should be closer to each other than is indicated by Equation (6). Secondly, in a few instances, the Hossain/Yang method showed significantly higher values than were reasonable (>80,000 MPa). Finally, the comparisons were obtained only for those slabs where the thickness was close to 11 inches (280 mm) so that the relationship between the distance of the #2 sensor and the slab thickness would satisfy the requirements of the Yang/Hossain method of forward calculation. This illustrates another advantage of the AREA method of calculating relative stiffness: Any bound surface course (or intermediate) layer, of any thickness or material type, can be calculated using the proposed AREA method.

A comparison of the stiffnesses derived from the AREA method versus the static lab moduli will be presented in *Section 3.2.3 – Bound PCC Surface Course Properties versus FWD-Based Test Results* (see Figure 33 in particular).

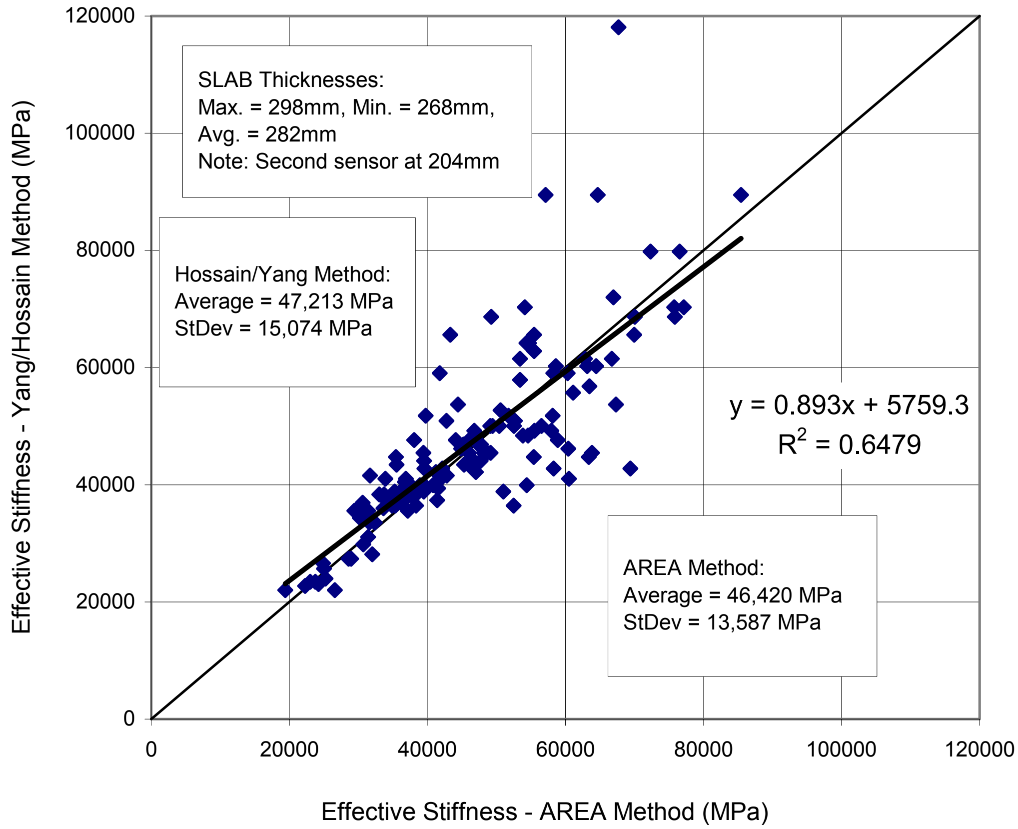

Figure 2. PCC Stiffnesses Derived through Two Forward Calculation Methods

2.2.4 Comparison of FWD Test Results from Subsequent Layers

An important question that needs to be asked is: Are FWD test results taken at one layer (for example, the subgrade) related to test results from subsequent layers? [It is necessary to assume that the same pavement section is tested twice within a reasonably short period of time surrounding construction of the subsequent layer(s).] This is important because it is necessary to know if the average stiffnesses and the variability thereof "carry through" with the same pattern of deflections from one layer to the next.

All available flexible pavement section data from the nationwide SPS-1 and -8 tests were organized in parallel data tables where subgrade, granular base and asphalt-bound surface FWD tests were conducted at each layer interface (see Table 5). Using the composite modulus Equations (1) and (2) shown in *Section 2.2.3 – Deflection-Derived Layer Quality Characterization*, it was possible to compare all of the data between these three types of layers. Many combinations of these values were compared, including Equation (1) for the center deflections and Equation (2) for offset sensors 2 through 4 for unbound layers and 2 through 7 for bound layer tests. In some cases, average values of two or three of the appropriate values were also compared to similar averages from other layer interfaces.

The purpose of these comparisons was to verify (or reject) the hypothesis that the FWD test results are correlated, both with respect to section averages (in terms of composite moduli) and standard deviations (spatial variability) from one layer to the next. In Figure 1, it was shown how the load-normalized deflections, from one layer to the next, "track" (or are parallel to) one another. In this subsection, it will be shown that the inverse of the deflections (expressed as moduli or stiffnesses) “track” one another, on a nationwide basis.

The best correlations found between the subgrade and granular base test results (in terms of modulus) utilized offset Sensors 3 or 4. In terms of section averages, the composite moduli derived using Sensor 4 for the subgrade and Sensor 4 for the granular base showed the best correlation between test sections ($R^2 = 0.69$), while in terms of variability (section standard deviations), Sensor 3 for the subgrade and Sensor 4 for the granular base offered the best correlation (also $R^2 = 0.69$). The plots of and relationships between these data are shown in Figure 3 and Figure 4, respectively.

As can be seen in Figure 3, the calculation of the unbound material (composite) modulus using Sensor 4 for each layer produced similar results for average moduli of around 200 MPa (~30,000 psi), a result that is fairly typical in connection with both forward- or back-calculation for unbound materials that are confined. Also as can be seen, at lower modulus values the composite modulus as calculated from Sensor 4 on the base course tends to be higher than the same sensor on subgrade tests. On the other hand, the opposite is true for the highest values shown in Figure 3 (i.e., the subgrade tests indicate a slightly higher composite modulus). In terms of standard deviations, as shown in Figure 4, the granular base tests always produce a slightly improved standard deviation in comparison with similar subgrade tests. This is also as expected, since the consistency of the overall structure should improve as the various structural layers are placed and compacted. This will become more obvious when the surface course tests are compared to the underlying tests on unbound materials later on in this subsection of the report.

The correlations between granular base and subgrade section moduli for some of the other combinations of sensors were also quite good, in many cases producing R^2-values between 0.5 and 0.6. The point here is that the FWD test results are reasonably well related to one another, in this case when tests are run on the subgrade layer prior to construction of the granular base layer, and after the base layer is placed and compacted.

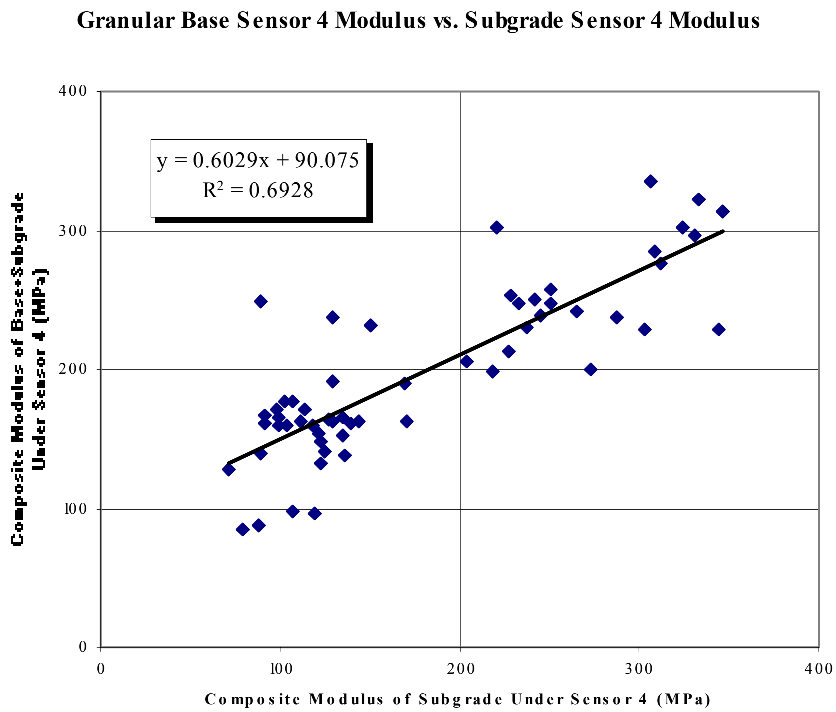

Figure 3. Average Composite Granular Base versus Subgrade Section Moduli

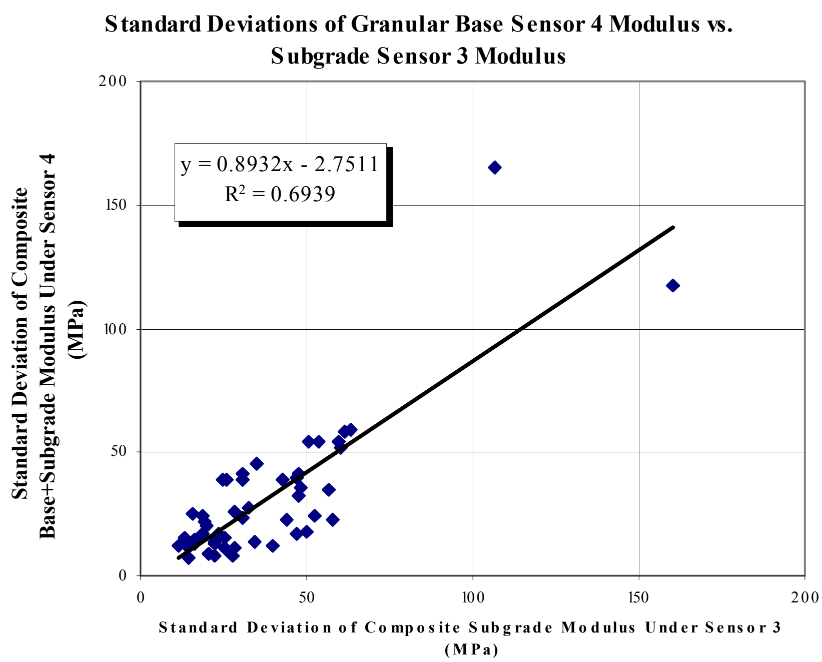

Figure 4. Average Composite Granular Base versus Subgrade Standard Deviations

It is also important to look at the correlations between FWD test results after the surface course layer(s) have been constructed versus those from the underlying layers, such as the subgrade and granular (unbound) base. The FWD has already been established as a useful device to characterize the bearing capacity of finished pavement structures. State and Provincial DOTs in North America have routinely used the FWD for over 20 years by now, and even longer in Europe. Therefore, if the tests conducted on the unbound layers below are well correlated with the FWD test results obtained at the pavement surface, this can lend even more credibility to FWD tests conducted on unbound materials.

The composite moduli (section averages and standard deviations) derived from all FWD tests conducted on AC surfaced pavements in the Table 5-listed database were compared to the companion FWD tests taken on the subgrade and granular base, respectively. These results are shown in Figure 5, Figure 6, Figure 7 and Figure 8.

The best correlations found between the granular base course and AC surface test results (in terms of modulus) utilized offset Sensors 4, 5 and 7. In terms of section averages, the composite moduli derived using Sensor 4 for the granular base and Sensor 5 for the AC surface showed the best correlation between test sections ($R^2 = 0.58$), while in terms of variability (section standard deviations), Sensor 4 for the granular base and Sensor 5 for the AC surface offered the best correlation (also $R^2 = 0.77$). The plots of and relationships between these data are shown in Figure 5 and Figure 6, respectively.

The best correlations found between the subgrade and AC surface two or more layers above (in terms of modulus) utilized offset Sensors 4, 6 and 7. In terms of section averages, the composite moduli derived using Sensor 4 for the subgrade and Sensor 7 for the AC surface showed the best correlation between test sections ($R^2 = 0.48$), while in terms of variability (section standard deviations), Sensor 4 for the subgrade and Sensor 6 for the AC surface offered the best correlation (also $R^2 = 0.67$). The plots of and relationships between these data are shown in Figure 7 and Figure 8, respectively.

Please note that in cases where AC surface test results using the outer sensors are correlated with unbound material tests below using Sensor 4 @ 18" (~450 mm) offset, the standard deviations correlate much better than the absolute (section average) values. This suggests that the *variability* of the unbound materials may be a more important, and probably more accurate, measure of the construction quality of a given unbound layer than the absolute moduli or average modulus. It also appears logical that the sensors situated farther from the loading plate for surface course tests correlate best with one of the nearby sensors for the granular base and subgrade tests, based on the conical shape of the compression wave generated by the FWD under load.

Further, it can also be seen that that the Sensor 6- or 7-derived unbound material (composite) modulus from the surface is somewhat higher than the values derived from FWD tests conducted directly on the unbound material layers below. This was more so in the case of AC surface versus subgrade than AC surface versus granular base tests. Undoubtedly, this is primarily due to the increase in confining pressures and the decrease in stress level on the base or subgrade after the AC surface course is placed.

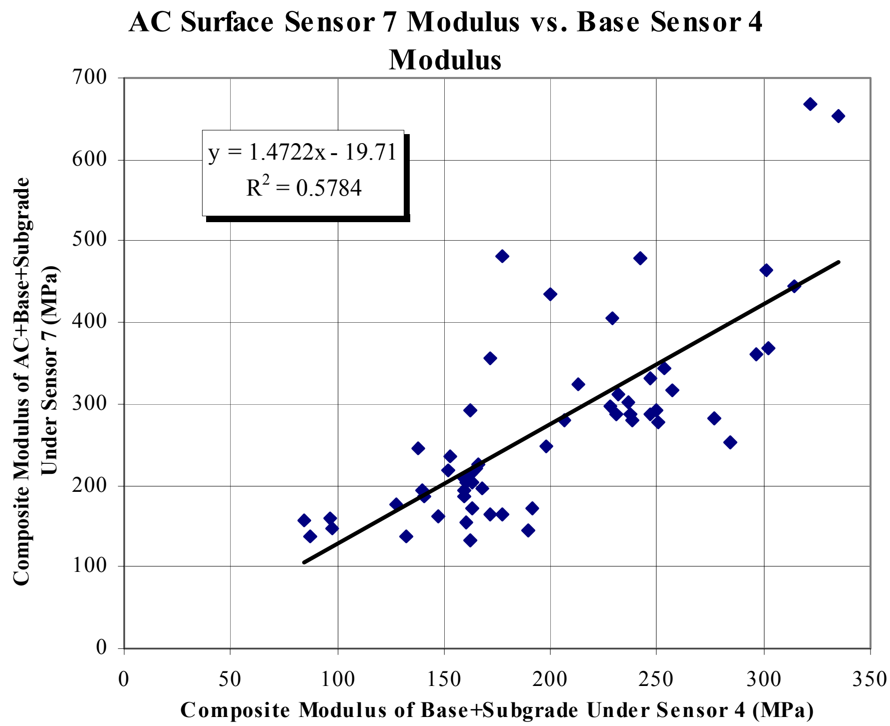

Figure 5. Average Composite AC Surface versus Granular Base Section Moduli

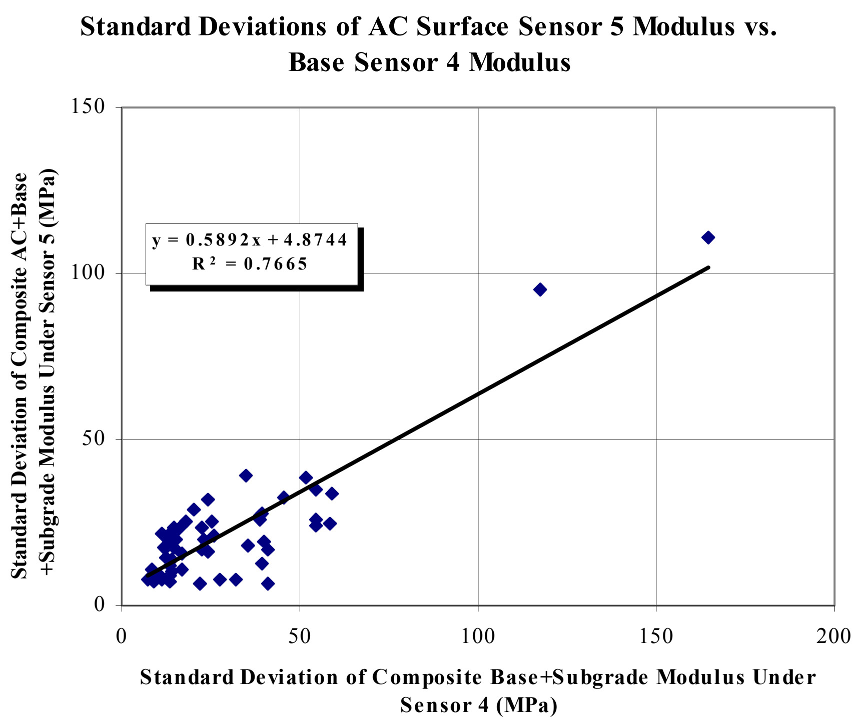

Figure 6. Average Composite AC Surface versus Granular Base Standard Deviations

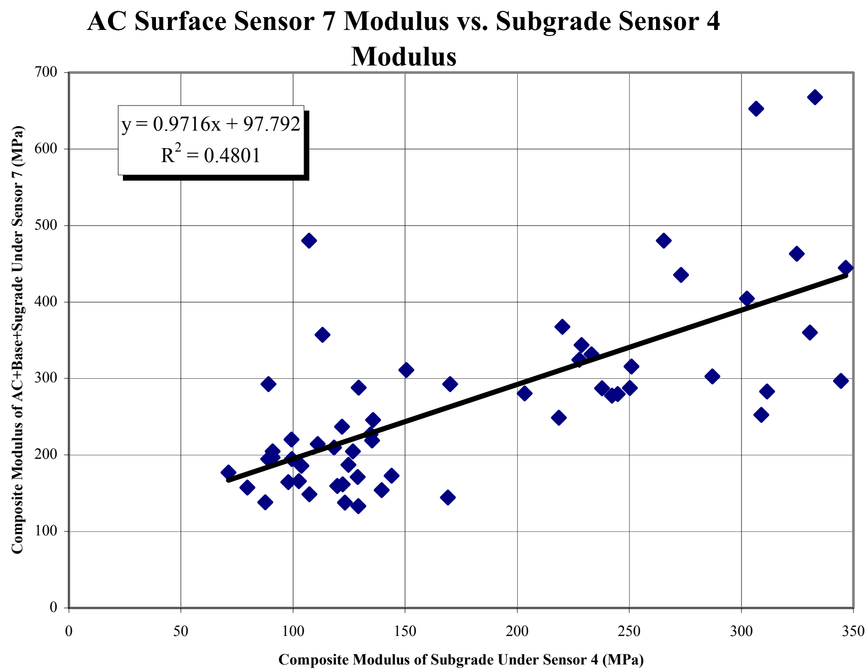

Figure 7. Average Composite AC Surface versus Subgrade Section Moduli

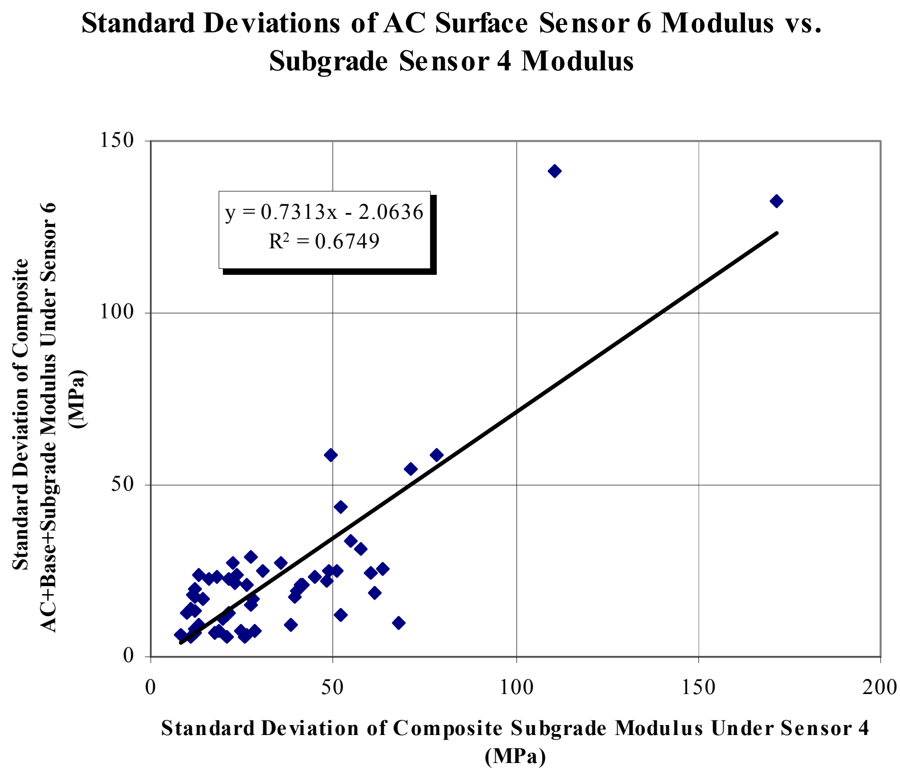

Figure 8. Average Composite AC Surface versus Subgrade Standard Deviations

On the other hand, the relationships between the unbound material FWD tests and the corresponding tests conducted on PCC surfaces showed less promise. In fact, after correlating all possible combinations of composite moduli, the only relationship that was over $R^2 = 0.15$ was the relationship between the standard deviation of the section moduli from Sensor 7-derived PCC surface course tests and the corresponding values for the Sensor 4-derived subgrade tests. This relationship is shown in Figure 9. As can be seen, the R^2-value is only 0.19, and there is a lot of scatter in the data (with one extreme outlier). It is suspected the reason for the poor correlations between PCC surface course tests and any measure of unbound material quality from tests taken directly on the bases and subgrades is the very high confining pressures and the very low stress levels involved when FWD tests are conducted on PCC surfaces. In other words, the PCC layer itself, being much stiffer than the underlying layers, "masks" the material properties of the subgrade, for example, when tests are run on the PCC surface and Sensors 6 or 7 with very small deflections are used in the analysis of subgrade properties.

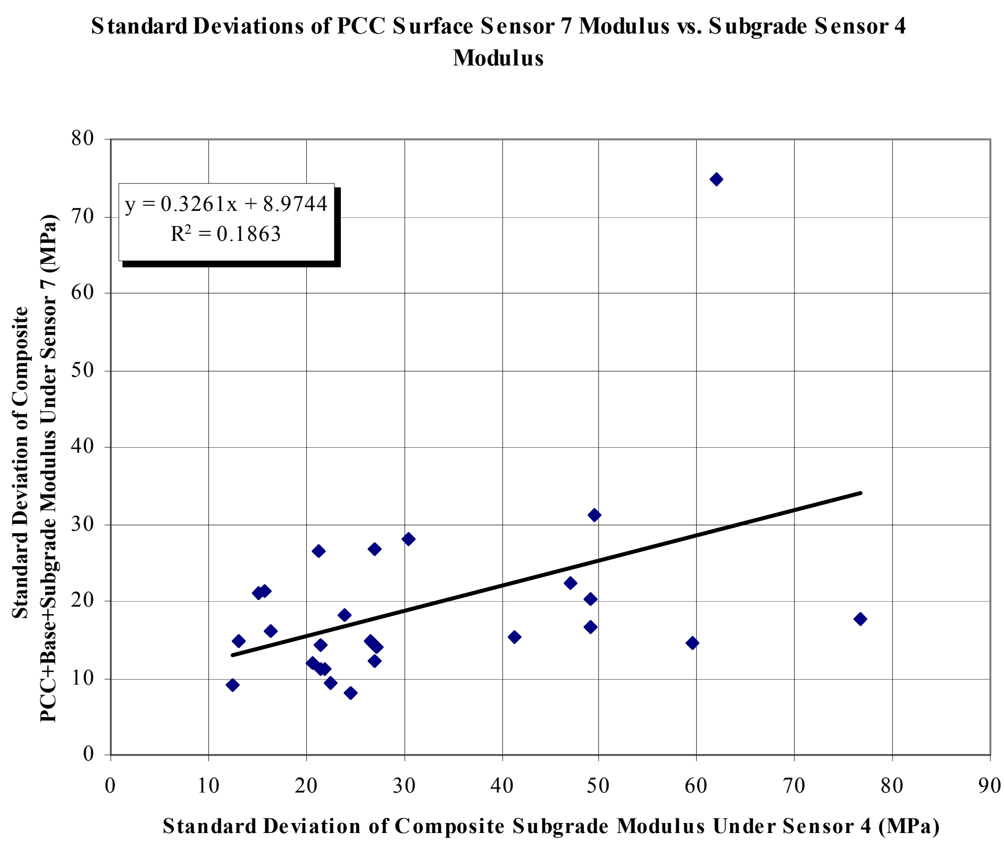

Figure 9. Average Composite PCC Surface versus Subgrade Standard Deviations

It is suspected that Sensor 4 for the unbound layers was more indicative of the unbound material properties compared to those tests taken at the surface than a closer-to-the-load-plate sensor, mainly due to the constraints associated with the unbound material test results in the LTPP IMS database, as previously discussed in *Sections 2.1.2 and 2.1.3* on FWD testing of unbound materials.

2.2.5 Examples of Bound Layer Stiffnesses Derived from Equation (5)

To further illustrate the use of Equation (5), as presented in *Section 2.2.3 – Deflection-Derived Layer Quality Characterization,* examples of the stiffnesses derived for a variety of bound material types in the SPS-1 and SPS-2 database (one SPS site for each example) are shown in Figure 10, Figure 11, Figure 12 and Figure 13, respectively.

As can be seen in Figure 10, the LCB stiffnesses, in this example from the SPS-2 site in Ohio, are fairly typical — some 1 thousand ksi (7 thousand MPa). In all cases, even using the statistically derived value of average less one standard deviation, the lowest stiffness is still around 700 ksi (for Section 39-0206), which is within the normally acceptable range of moduli for LCB in new pavement construction.

In Figure 11, from data taken along the same SPS-2 site in Ohio, the effective Permeable Asphalt Treated Base stiffnesses derived using Equation (5) appear somewhat low—on average around 30 ksi (~200 MPa), with the lowest statistical averages less one standard deviation around 18 ksi for Section 39-0211. Nevertheless, these stiffness values may still be acceptable for a lightly compacted, open-graded mix with low binder content that is intended as a drainage layer under the PCC slabs.

For the same SPS-1 site shown in Figure 1 (see *Section 2.2.1 – Example of FWD Center Deflections for an SPS-1 Site),* the effective AC stiffnesses are shown in Figure 12 for the Lane 1 FWD test results. From the data plotted in both Figure 1 and Figure 12, it is evident that Section 39-0101 has abnormally low stiffnesses, especially in light of the low pavement temperature at the time of test (~48^{O}F or 9^{O}C). These values were in the 200-500 ksi (1,500-3,500 MPa) range, which is unacceptable, also in terms of the variations encountered. Section 39-0105 also shows fairly low stiffness values, in the 500-800 ksi (3,500-5,500 MPa) range, especially considering the AC mat temperature of about 43^{O}F (6^{O}C) at the time of testing. From the results shown in Figure 12, it would also appear that Section 39-0108 is not much better in terms of quality compared to Sections 39-0101 (already replaced, as mentioned previously) and 39-0105. However, the AC mat temperature at the time of FWD testing was somewhat higher in Section 39-0108 (~55^{O}F or 13^{O}C), in addition to which the stiffnesses were higher (800-1000 ksi or 5,500-7,000 MPa), and they showed less variation. Section 39-0105 was also replaced about two years after construction. All other AC sections along Ohio's SPS-1 site, on the other hand, appear to have reasonable AC layer stiffnesses.

For the Ohio SPS-2 site, the effective PCC layer stiffnesses, derived using Equation (5), are shown in Figure 13 for Lane J1 tests (Lane J1 is the same as Lane 1, except that FWD tests are only conducted at centerslab positions). None of the PCC stiffnesses shown in Figure 13 appear to be abnormally low. In fact, some sections exhibit questionably high PCC stiffnesses (up to ~10 million psi), which may indicate excessive scatter in the data or a slight over-estimation based on Equation (5), or both. However, the values obtained appear reasonable.

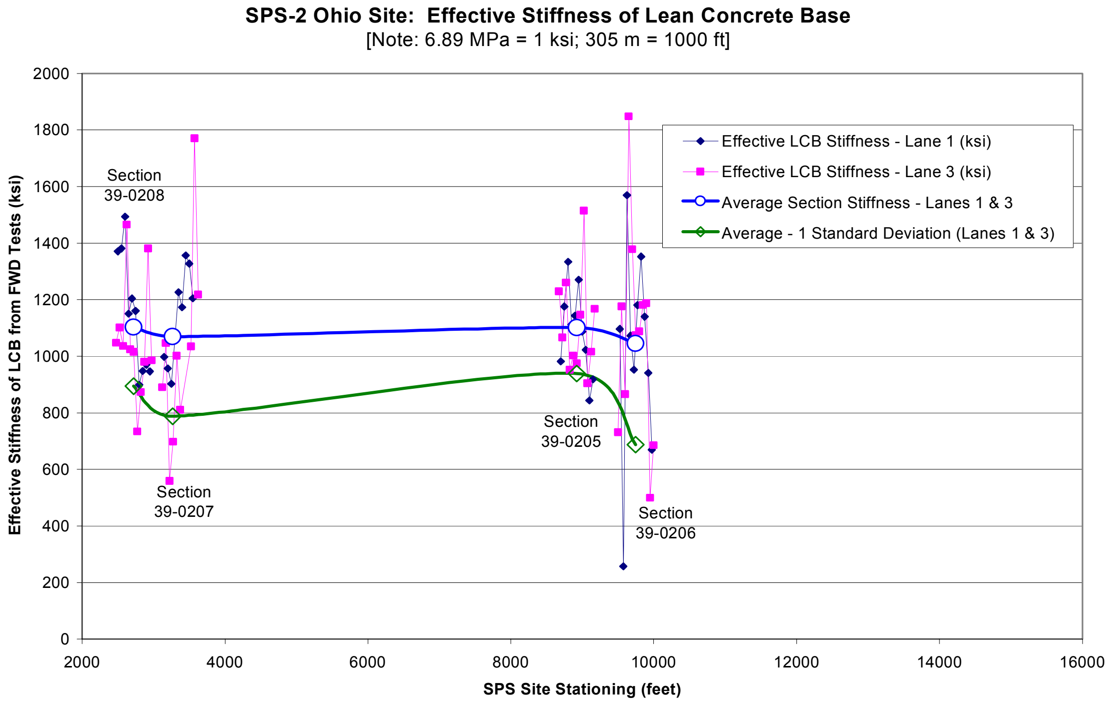

Figure 10. Relative Stiffnesses from LCB Tests for SPS-2 Site in Ohio

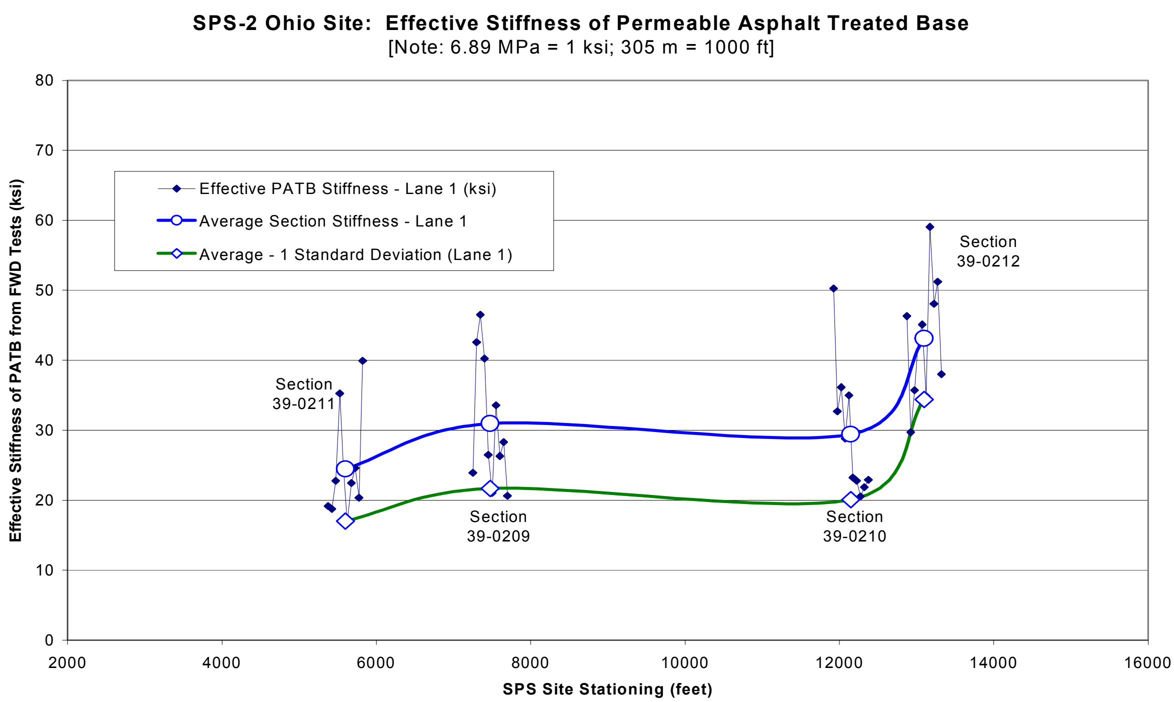

Figure 11. Relative Stiffnesses from PATB Tests for SPS-2 Site in Ohio

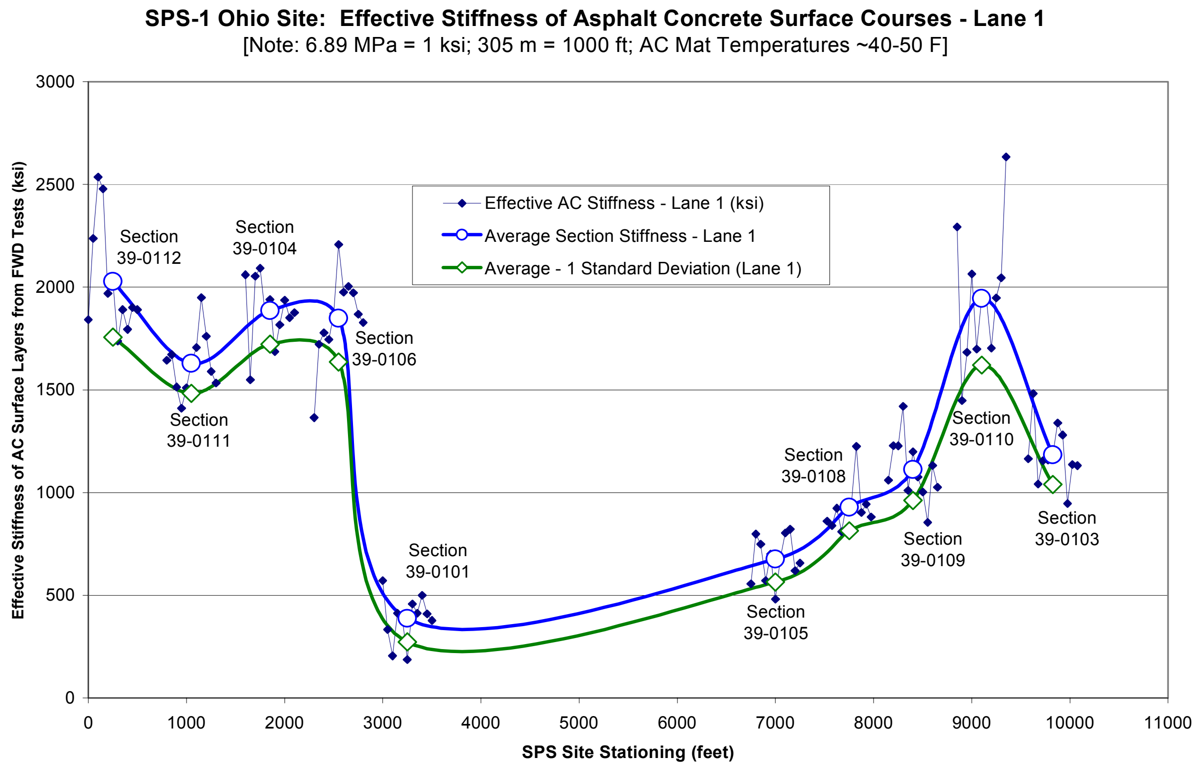

Figure 12. Relative Stiffnesses from AC Surface Course Tests for SPS-1 Site in Ohio

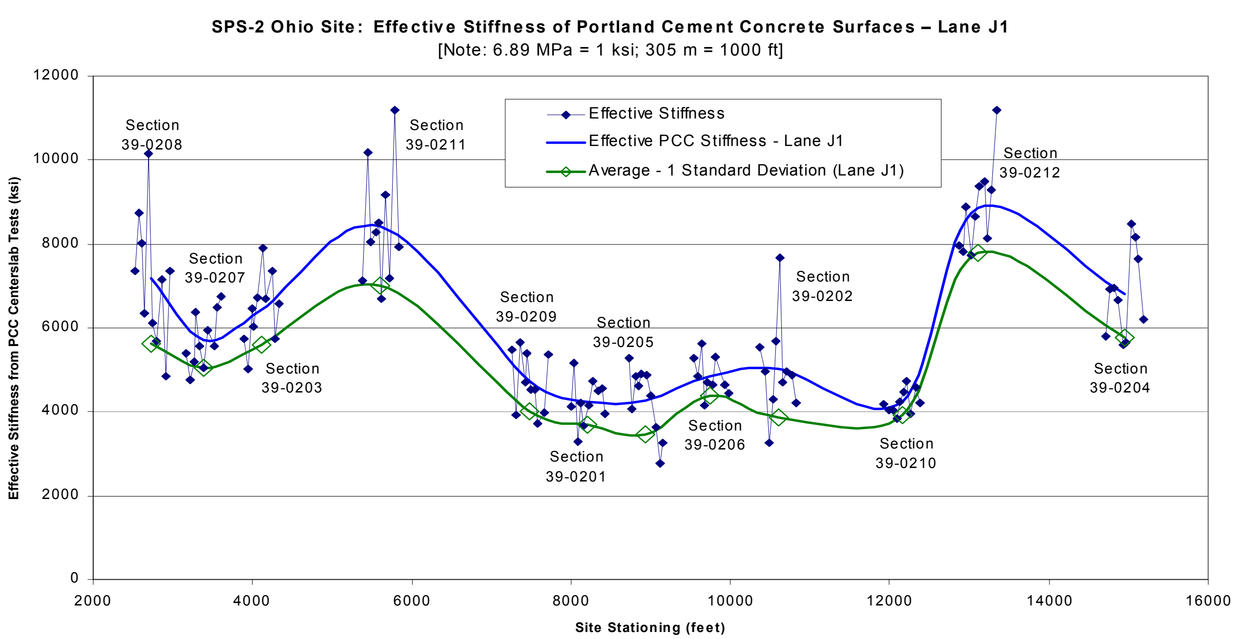

Figure 13. Relative Stiffnesses from PCC Centerslab Tests for SPS-2 Site in Ohio

2.3 Traditional Pavement Construction Quality Data

This subsection summarizes the extent and quality of the various traditional pavement construction-associated QC/QA data, from LTPP tests other than the FWD.

For unbound materials (subgrade soils and granular bases), the following data are available in the IMS:

1. In-situ material density
2. In-situ material moisture content
3. Laboratory Proctor or Modified Proctor maximum density & optimum moisture
4. Relative in-situ density as a percentage of maximum (Proctor) density (calculated)
5. Percent retained on the 2 mm sieve
6. Percent "coarse" sand
7. Percent "fine" sand
8. Percent silt
9. Percent clay
10. Liquid Limit (LL)
11. Plastic Limit (PL)
12. Plasticity Index (PI)
13. Percent passing the 0.074 mm sieve (P200)
14. Unconfined compressive strength
15. Laboratory modulus of elasticity

Generally, the current practice for quality control during the construction of unbound layers is to monitor moisture and density during placement and the method of placement. The practice is essentially a "pass-fail" method. It doesn't address bearing capacity directly as a deflection measurement would. Accordingly, existing practices have several disadvantages, the most obvious of which are:

- Relative density tests are relevant only as long as the soil being tested belongs to the same soil that was tested in the lab to establish the base line (the Proctor test). If the maximum density drifts lower, the field-testing would provide passing results with less compaction effort. If the maximum density drifts higher, the contractor will have more difficulty in obtaining compaction, most likely leading to obtaining a new maximum Proctor density value. The relationship between specified densities and the soil support values used during the design of the pavement is understood to be a positive one, but not one that is normally applied during construction.
- Test strip or control strip methods are influenced by the overall combination of equipment and the underlying soil support. The control becomes a density that is some percentage of the best the contractor could get, given the equipment and field conditions. The relationship between the test strip method and the soil support values used during the design of the pavement is understood to be a positive one, but is even more removed from actual design values than specified density methods, because no laboratory tests are carried out to characterize soil-moisture-density characteristics.
- Proof-rolling methods generally identify areas of excess moisture, poor underlying support, or low compaction of the upper layers. This practice is relevant to particular soils and local conditions, but it might not relate to the assigned subgrade support value. Most proof-rolling procedures are such that the weakest soils, if properly placed, will

pass. This method has the least connection of the three typically used to actual soil support values used during the design of a pavement structure.

For bound materials, data elements available in the database that may have some relationship to bearing capacity include:

1. Asphalt layer nuclear density
2. PCC layer strength parameters (compressive strength, tensile strength, modulus of rupture, and modulus of elasticity)
3. AC layer strength parameters (primarily lab modulus from test protocol P07)
4. Grid layer thicknesses for all structural layers
5. Backcalculated modulus for all structural layers (where available)

There are other data elements that describe the constituent properties of asphalt-bound materials, such as binder properties and aggregate properties that may relate to bound layer stiffness. Due to a lack of adequate data, these properties were not compared to the deflection results in this study. For the asphalt, a relationship between the laboratory moduli and deflections was evaluated. For concrete, laboratory moduli and several strength properties were also evaluated. Overall, the results for the concrete did not provide very good correlations with the deflection results, nor did the asphalt properties in the preliminary trials carried out.

On the other hand, the concrete flexural design strengths versus the associated deflection-based forward calculated stiffnesses ranked in the same order in 20 out of 26 cases evaluated (with one virtual tie), indicating a very strong connection; however, the magnitude of the values are likely influenced by many other parameters that are not found in the LTPP database.

A useful reference that discusses LTPP's backcalculated modulus (computed parameter) tables compared to other test data in the IMS is Reference (8). According to this document, the backcalculated moduli in the IMS have very little relationship to other lab-associated data, for the same materials and at the same test sections (both with respect to SPS and GPS test section data). This was possibly due to the fact that not all of the deflection data resulted in acceptable Level E backcalculated moduli (as computed parameters in the IMS database), while virtually all the lab data were represented. This explanation is purely conjecture, although some of the results presented in *Chapter 3.2.3 —Bound PCC Surface Course Properties versus FWD-Based Test Results* indicate that this could explain at least part of the problem.

CHAPTER 3—RELATIONSHIPS BETWEEN FWD-DERIVED PARAMETERS AND TRADITIONAL CONSTRUCTION QUALITY DATA

3.1 Comparison of Unbound Material Data with Deflection-Associated Data

As a first step in the analysis of related unbound material data, field densities, relative densities, normalized deflection values, and layer moduli were associated, by stationing, for all available unbound layers in the SPS-1, -2 and -8 database. The subgrade may consist of up to three layers, depending on the site, since multiple subgrade layers were constructed on some sites. On occasion, the subgrade consisted of native soil followed by one or two fill layers, while in others the subgrade consisted of native material with treated soil above. As previously mentioned, unbound material deflection data was designated as Lane S1 or S3 for subgrade testing and G1 or G3 for aggregate base testing. In the case of multiple layers of subgrade material, it was not known exactly which layer the deflection tests were conducted on. The data were not used where there was a reasonable doubt regarding what subgrade layer the FWD tests were on.

As of approximately August 2001, the LTPP IMS database contained 9,648 sets of FWD deflection basins, from 4,824 test locations, conducted on unbound materials. This represents data from 144 test sections at 15 SPS sites distributed over 11 states. Of these, there are 1,964 deflection basins (at 982 test locations) on 51 test sections that have both in-situ density and moisture data, along with parallel maximum density and optimum moisture (Proctor) values. The association criteria used was that the in-situ measurement had to be within 51 feet of the deflection data, to allow up to two deflection test locations on either side of any utilized in-situ density measurement taken on the subgrade. This introduced some spatial variation, but it provided more deflection data with which to compare material properties versus FWD deflections.

The first calculations conducted were intended to develop a table of mean deflections and deflection-based values, field densities and moistures, and Proctors for each of the test sections, resulting in an array of 51 test sites and 26 deflection and material parameters for all unbound materials, including granular bases and subgrade soils. A correlation matrix was calculated for the array of data to provide an initial indication of relationships of individual variables. The single deflection parameter that had the best relationship with all of the unbound material parameters was the "Delta2" value, which is the center sensor deflection (d1) minus the second sensor deflection (d2), i.e. a kind-of indication of the strength of the upper-most portion of the unbound materials under and near the FWD load plate. Specifically, as the Delta2-values decrease, the apparent unbound material strengths increase.

Delta2 correlated with the available unbound material properties, as follows:

Material Parameter	Correlation Coefficient, R
In-situ Dry Density	-0.422
In-situ Moisture Content	0.479
Maximum Density	-0.429
Optimum Moisture Content	0.485
Relative Density	0.056

Although these are not very strong correlations (they are not R-squared), they do indicate that the FWD-derived values still have some relationship to the conventional QC/QA properties, for all unbound materials in the correlation matrix. The signs are also as expected, for instance: as the dry densities increase (better compaction or stronger materials), the Delta2-values decrease (stiffer unbound material). The relative density does not, in itself, relate well with deflection parameters with the variety of subgrade materials that this data represents. If we could confine the measurements to truly the same material, then perhaps the relative densities would have a good (negative, as expected) correlation with the Delta2-values, but the variations of the other parameters evidently overpower any possible correlation with relative densities in this particular nationwide data set.

The plot in Figure 14 shows the relationship between load-normalized Delta2 and the in-situ moisture content for all available data pairs. It clearly shows that the high Delta2-values tend to be associated with higher moisture contents, as expected. The plot shown in Figure 15 shows that the density declines as the Delta2-value increases, also as expected.

Both Figure 14 and Figure 15 reflect quite a bit of scatter. However, the three data points in Figure 14 that are the furthest from the trend line correspond to three of the four points in Figure 15 that are also the furthest from the trend line. The additional point in Figure 15 is from the same site as two of the other "outliers." The exact cause of these outliers is not known. It is possible that the subgrade was stabilized, or had "crusted", causing the Delta2-values to be much lower than the densities and moisture contents would predict.

In the next phase of the analysis, the granular base properties were extracted from the IMS, in an effort to see if the correlations shown in Figure 14and Figure 15 could be improved by including only one broad unbound material type.

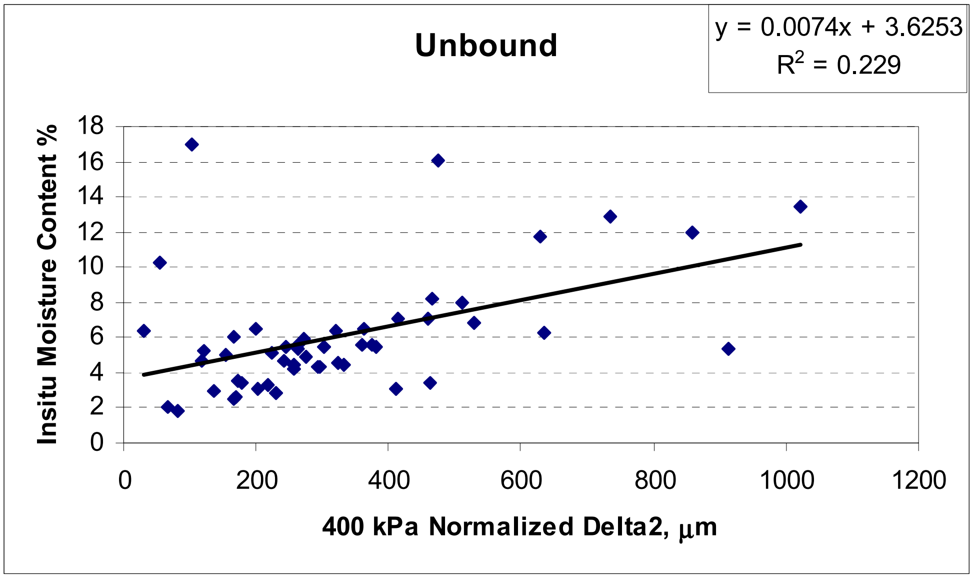

Figure 14. Relationship between In-situ Unbound Moisture and Delta2 (d1-d2)

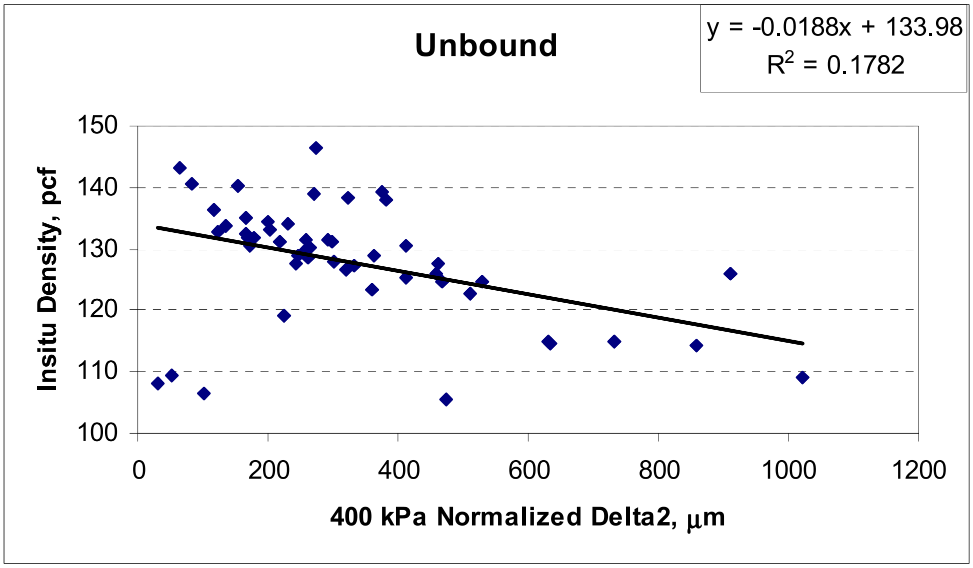

Figure 15. Relationship between In-situ Unbound Density and Delta2 (d1-d2)

3.1.1 Granular Base Properties versus FWD-Based Test Results

In an effort to improve upon the overall relationships derived for all unbound materials combined, without differentiating between the various subgrade types and granular bases, the granular base properties alone were analyzed to see if they would result in better correlations between FWD-derived properties and other traditional QC/QA test results. A dataset of granular base materials was thus created and the same type of analyses were conducted (deflection and material properties, averaged by section) as was done for the entire unbound data set, and a correlation matrix of the resulting average values was developed.

Figure 16 shows a plot that corresponds to the best correlation in the matrix, i.e. that of the previously discussed AREA factor versus the in-situ dry density of the granular base. [The AREA factor is defined by Equation (3) in *Section 2.2.3 – Deflection-Derived Layer Quality Characterization.*] If density is an approximate – albeit indirect – indicator of layer stiffness (and we believe it is), then the relationship shown in Figure 16 is rational. The correlation is also better than when the entire unbound material data set was used, with an R-squared of 0.32 — better but still not entirely convincing. In theory, the AREA factor is most closely associated with the ratio of upper layer stiffness to underlying layer stiffness, so the underlying subgrade strength should have an impact on the relationship. To confirm this hypothesis, the residuals of the AREA versus in-situ density were compared to some of the offset sensor deflections, which in theory also respond to the underlying stiffness. Interestingly, these residual checks showed very little relationship; the residuals themselves, however, showed a reasonably strong relationship to the relative densities, as shown in Figure 17.

Next, the in-situ relative densities were added to the AREA factor, and both of these were compared to the in-situ dry densities using a multiple regression analysis. This may provide additional evidence of a significant relationship between deflection (or stiffness) and the traditional granular base QC/QA properties, although not a particularly useful one since some form of in-situ density is on both sides of the regression. It should be pointed out, however, that relative density and in-situ density have virtually no relationship to one another in this data set, and in all likelihood in any data set that could be assembled from different materials. Table 6 shows the results of the multiple regression analysis conducted. The AREA versus relative density regression is significant at an R-squared of 0.76, while the intercept and independent variable coefficients are strongly significant (P-value < 0.001).

A search for independent variables was then conducted that could show a useful tie between the relative density of the base material and some deflection-based or other material parameter. Table 7 is one such relationship that shows that relative density can be satisfactorily predicted by two deflection parameters plus one other unbound material property. The deflection parameters were the normalized deflections at 8 inches (~200 mm) from the center of the load plate, and the difference between the deflection at the center of the load plate and the deflection eight inches from the center of the load plate, “Delta2”. The material parameter that was most significant in this relationship was the optimum moisture content found in the laboratory Proctor density tests (at maximum density). In this case, the adjusted R-squared of the proposed relationship was a respectable 0.71.

The optimum moisture content, in this case, is an interesting material parameter in terms of how it relates to the relative in-situ densities. The sign of the coefficient is positive, indicating that the relative densities are higher when the optimum moisture content is higher. One could speculate that a base with enough fines to serve as a binder compacts better. The optimum moisture content typically has a positive relationship with the fines content of the base aggregate and actually relates to the maximum density when a large number of test results are evaluated.

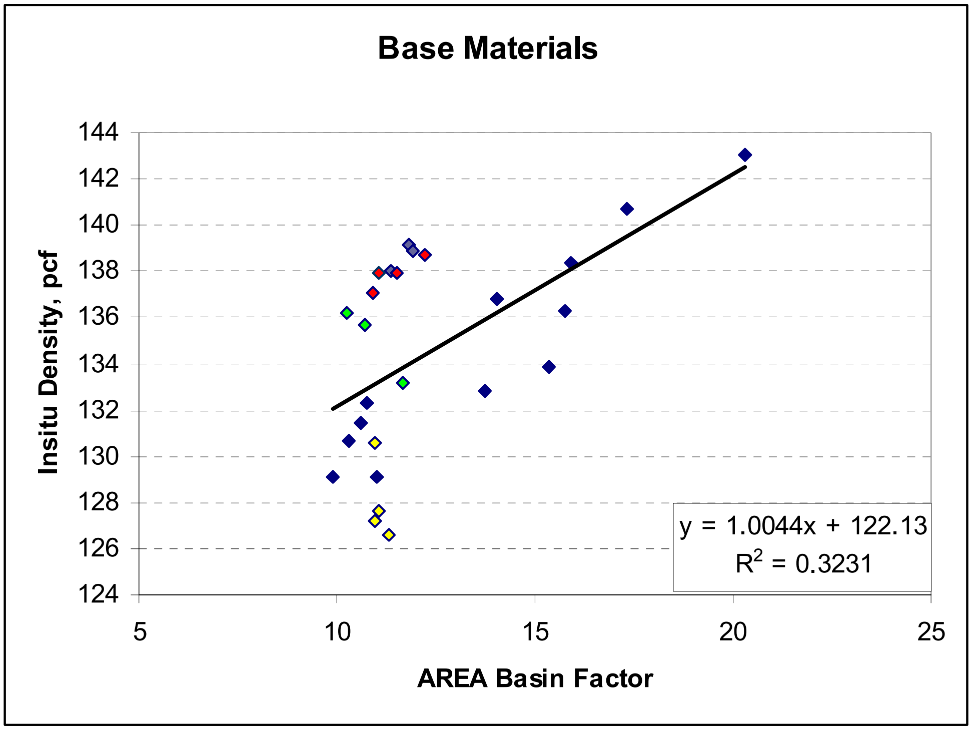

Figure 16. Relationship between In-situ Base Dry Density and the AREA Factor

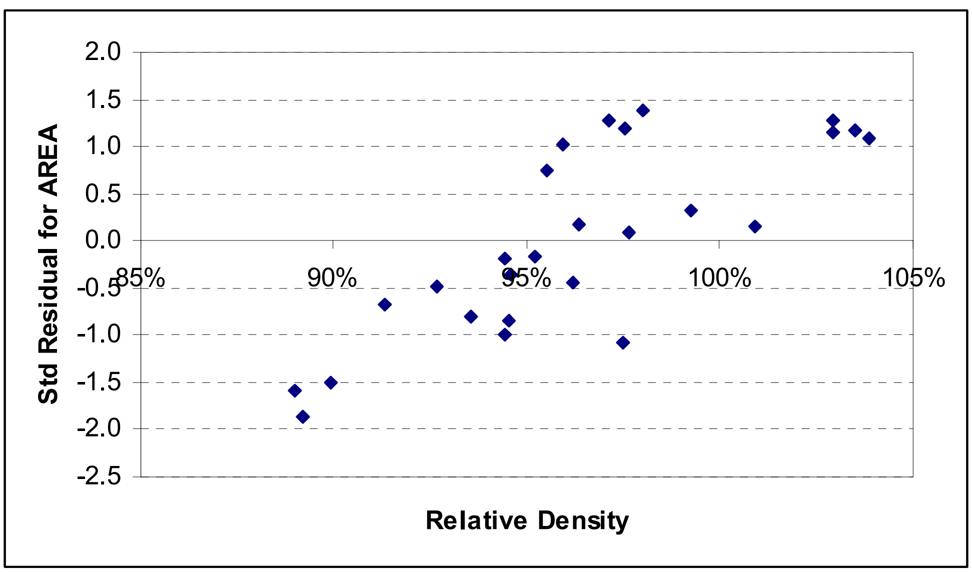

Figure 17. Relationship between In-situ Base Relative Density and AREA Residuals

Table 6. In-situ Density as a Function of AREA Basin Factor and Relative Density

Regression Statistics	
Multiple R	0.885
R Square	0.783
Adjusted R Square	0.764
Standard Error	2.193
Observations	26

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	398.52	199.26	41.44	0.00
Residual	23	110.59	4.81		
Total	25	509.11			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	53.502	10.075	5.310	0.000	32.660	74.344
AREA	0.669	0.178	3.751	0.001	0.300	1.038
% Rel. Den	75.587	10.835	6.976	0.000	53.173	98.001

Table 7. Relative Densities as a Function of Deflection and Material Properties

Regression Statistics	
Multiple R	0.8620
R Square	0.7430
Adjusted R Square	0.7063
Standard Error	0.0218
Observations	25

ANOVA

	df	SS	MS	F	Significance F
Regression	3	0.0289	0.0096	20.2356	0.0000
Residual	21	0.0100	0.0005		
Total	24	0.0389			

	Coefficients	Std. Error	t Stat	P-value	Lower 95%	Upper 95%
Intercept	0.882908	0.0153	57.5515	0.0000	0.8510	0.9148
n.defl.2	0.000175	0.0001	3.4306	0.0025	0.0001	0.0003
Delta2	-0.000224	0.0001	-4.3226	0.0003	-0.0003	-0.0001
Opt. MC	0.013147	0.0022	5.8952	0.0000	0.0085	0.0178

Figure 18 shows the observed relative densities from the IMS plotted against the predicted relative densities from the above-derived multiple regression relationship, thus providing an alternate view of the goodness-of-fit. As previously indicated, the nature of the data collected for LTPP was not designed specifically for this type of analysis, but the data presents an opportunity to explore how deflection data relates to traditional materials tests used for quality control. Relative densities are expected to relate to the material stiffness, at least from a "conventional wisdom" standpoint. Many materials studies have shown strong relationships between density and stiffness and, therefore, deflections should also relate to relative densities in the field, and the data evaluated here provides very strong evidence that this is, in fact, the case.

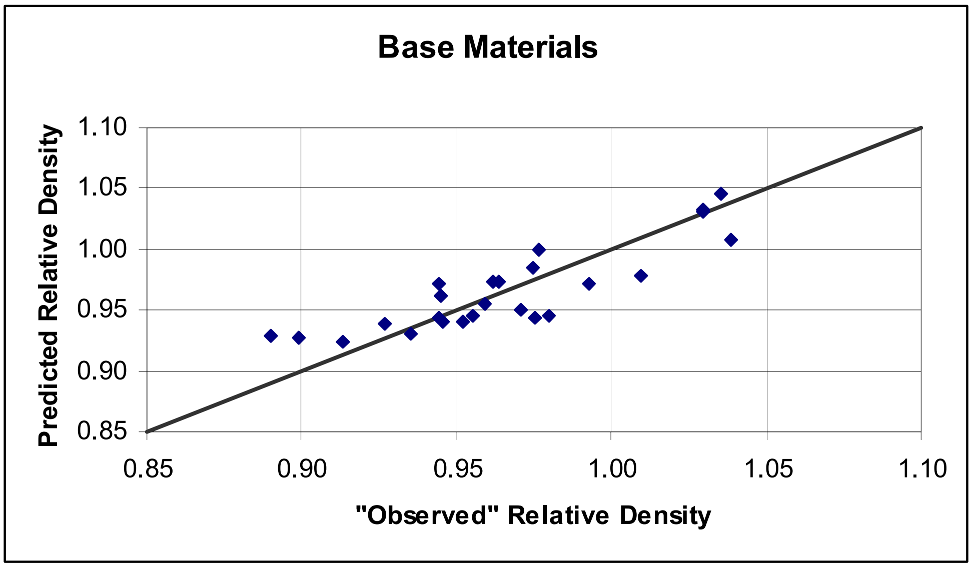

Figure 18. Relationship between Predicted and Observed Base Course Relative Density

3.1.2 Subgrade Properties versus FWD-Based Test Results

The entire IMS dataset of cross-referenced subgrade soil properties, where both traditional and FWD-based test results were available in the IMS, were then analyzed without including the granular base data. A correlation matrix was developed for all available test results, though with the FWD results load-normalized to a standard plate pressure of 400 kPa (58 psi). The resulting correlation matrix is shown in Table 8, which includes all correlation factors from all subgrade data where there are all of the following data: deflections, traditional subgrade soil tests, general soil properties, and resilient modulus tests.

The strongest modulus relationships in Table 8 are those between the coarse fraction (GT 2 mm) of the gradation and the composite surface moduli from the 3rd and 4th sensors, and some of the other offset sensors as well. Other strong relationships are also indicated, for example the AF^2 versus % silt ($R^2 = 0.91$). However, the data represent only 12 sections, seven from Arizona (three from SPS-1 and four from SPS-2), four from Colorado's SPS-2, and one from Louisiana's SPS-1, because these are the only SPS sections where laboratory resilient modulus data exists in the Level E database. The dataset is therefore too small to draw any definite conclusions.

Table 8. Correlation Matrix for Subgrade Soils with Lab M_r Tests

		In-situ Density	In-situ MC	Max Den	Opt MC	Rel.Den %	% GT 2 mm	% Cr. Sand	% Fine Sand	% SILT	% CLAY	LL	PL	PI	Lab Mr
Normalized Deflection and deflection Parameters	D1	-0.28	0.53	-0.20	0.35	-0.28	-0.28	-0.07	0.57	-0.21	0.53	0.14	-0.07	0.34	-0.27
	D2	-0.21	0.46	-0.16	0.25	-0.21	-0.21	-0.05	0.459	-0.18	0.44	0.13	-0.01	0.25	-0.22
	D3	-0.19	0.37	-0.14	0.15	-0.18	-0.19	-0.08	0.35	-0.10	0.32	0.12	0.06	0.14	-0.20
	D4	-0.32	0.42	-0.27	0.17	-0.27	-0.27	-0.25	0.26	0.07	0.36	0.30	0.27	0.23	-0.24
	D5	-0.41	0.44	-0.36	0.18	-0.34	-0.31	-0.38	0.16	0.21	0.36	0.42	0.40	0.29	-0.26
	D6	-0.48	0.42	-0.42	0.17	-0.39	-0.32	-0.50	0.06	0.33	0.34	0.47	0.47	0.31	-0.28
	D7	-0.48	0.35	-0.43	0.15	-0.37	-0.33	-0.55	-0.01	0.42	0.26	0.40	0.42	0.23	-0.33
	Delta2	-0.32	0.57	-0.23	0.410	-0.333	-0.320	-0.076	0.638	-0.222	0.588	0.152	-0.105	0.394	-0.307
	D2-D3	-0.23	0.54	-0.17	0.35	-0.23	-0.21	-0.03	0.55	-0.26	0.54	0.13	-0.10	0.35	-0.23
	$\frac{(D2-D4)}{D3}$	-0.02	0.49	0.05	0.34	-0.10	0.04	0.18	0.60	-0.55	0.55	-0.00	-0.37	0.43	-0.08
	Delta3	-0.27	0.53	-0.18	0.36	-0.28	-0.27	-0.04	0.59	-0.24	0.54	0.12	-0.11	0.34	-0.27
Surface Moduli, MPa	E_o1	0.11	-0.48	-0.08	-0.25	0.29	0.18	-0.19	-0.82	0.53	-0.61	-0.12	0.13	-0.36	0.28
	E_o2	0.25	-0.48	0.10	-0.29	0.34	0.38	0.09	-0.65	0.18	-0.57	-0.27	-0.10	-0.36	0.44
	E_o3	0.31	-0.31	0.18	-0.21	0.36	0.50	0.22	-0.43	-0.12	-0.36	-0.29	-0.28	-0.19	0.49
	E_o4	0.43	-0.30	0.32	-0.24	0.41	0.60	0.36	-0.29	-0.34	-0.32	-0.40	-0.44	-0.21	0.55
	E_o5	0.52	-0.34	0.40	-0.27	0.49	0.65	0.49	-0.23	-0.46	-0.35	-0.52	-0.56	-0.28	0.57
	E_o6	0.61	-0.37	0.48	-0.28	0.56	0.65	0.60	-0.17	-0.54	-0.37	-0.60	-0.63	-0.35	0.55
	AREA	-0.18	-0.30	-0.33	-0.10	0.04	-0.18	-0.49	-0.72	0.82	-0.42	0.16	0.43	-0.20	-0.09
	A.F.^2	-0.33	-0.16	-0.49	0.06	-0.05	-0.30	-0.64	-0.65	0.91	-0.28	0.24	0.42	-0.05	-0.23
	Eff. SG	-0.29	-0.19	-0.46	0.04	-0.01	-0.24	-0.60	-0.69	0.89	-0.33	0.19	0.37	-0.08	-0.16

Table 9. Correlation Values from Subgrade Data Set Without Lab M_r Tests

		In-situ Density	In-situ MC	Max Den	Opt MC	Rel.Den %	% GT 2 mm	% CR. Sand	% Fine Sand	% Silt	% Clay	LL	PL	PI
Normalized Deflection and deflection Parameters	D1	-0.43	0.47	-0.39	0.42	0.00	-0.44	-0.04	0.43	0.15	0.39	0.01	-0.11	0.18
	D2	-0.41	0.43	-0.30	0.29	-0.13	-0.46	0.09	0.49	0.09	0.45	0.09	0.00	0.20
	D3	-0.42	0.41	-0.22	0.17	-0.26	-0.47	0.12	0.47	0.08	0.46	0.15	0.11	0.18
	D4	-0.52	0.49	-0.25	0.14	-0.38	-0.52	0.03	0.42	0.17	0.49	0.21	0.18	0.20
	D5	-0.54	0.50	-0.24	0.09	-0.44	-0.48	-0.04	0.35	0.19	0.46	0.23	0.22	0.20
	D6	-0.51	0.45	-0.17	-0.01	-0.51	-0.38	-0.08	0.29	0.16	0.41	0.23	0.23	0.19
	D7	-0.52	0.44	-0.21	0.02	-0.47	-0.37	-0.14	0.23	0.21	0.29	0.09	0.10	0.06
	Delta2	-0.43	0.48	-0.44	0.49	0.09	-0.41	-0.12	0.37	0.18	0.33	-0.04	-0.17	0.16
	D2-D3	-0.35	0.40	-0.34	0.39	0.05	-0.39	0.04	0.44	0.08	0.39	0.01	-0.13	0.20
	$\frac{(D2-D4)}{D3}$	0.13	0.01	-0.15	0.32	0.47	0.10	-0.01	0.05	-0.08	-0.06	-0.20	-0.35	0.05
	Delta3	-0.39	0.44	-0.39	0.44	0.06	-0.41	-0.04	0.41	0.13	0.36	-0.02	-0.14	0.17
Surface Moduli, MPa	E_o1	0.23	-0.33	0.32	-0.42	-0.21	0.24	-0.01	-0.39	-0.01	-0.20	0.14	0.27	-0.08
	E_o2	0.52	-0.49	0.46	-0.47	0.01	0.62	0.03	-0.40	-0.29	-0.42	-0.04	0.04	-0.14
	E_o3	0.63	-0.52	0.44	-0.37	0.22	0.71	0.08	-0.35	-0.39	-0.47	-0.16	-0.15	-0.14
	E_o4	0.64	-0.52	0.39	-0.28	0.34	0.68	0.10	-0.31	-0.39	-0.44	-0.13	-0.14	-0.09
	E_o5	0.60	-0.49	0.29	-0.16	0.43	0.57	0.10	-0.30	-0.31	-0.39	-0.12	-0.14	-0.07
	E_o6	0.41	-0.33	0.05	0.10	0.54	0.30	0.01	-0.30	-0.08	-0.30	-0.13	-0.14	-0.09
	AREA	-0.28	0.09	-0.03	-0.16	-0.41	-0.24	-0.18	-0.22	0.32	0.09	0.25	0.38	0.01
	A.F.^2	-0.39	0.18	-0.18	-0.02	-0.34	-0.31	-0.33	-0.29	0.45	0.09	0.25	0.35	0.04
	Eff. SG	-0.31	0.11	-0.12	-0.07	-0.31	-0.23	-0.32	-0.34	0.41	0.03	0.23	0.34	0.02

With the omission of subgrade modulus tests, it was possible to construct a correlation matrix that represents 36 SPS sections instead of only 12. Table 9 is the resulting set of correlations where the subgrade data included all other lab tests, field densities and FWD deflections. Although not a complete set of all LTPP SPS-1, -2 and -8 sections, there are now enough sections to identify the traditional material properties such as gradation, Atterberg limits, densities, and moisture contents that relate to deflections measured directly on the subgrade during construction.

A confounding factor with the field deflection measurements and the material properties with fine-grained soils is that some of the SPS soils did receive some sort of stabilization, such as lime, cement or fly ash, in the upper 150- to 300-mm of subgrade. It was not clear exactly what the status of the stabilization was at the time of deflection testing, or if in fact stabilization was used at all. The effect of the stabilization would be most pronounced near the load plate and less pronounced away from the FWD load plate. This is possibly one of the reasons that the correlations are evidently better for deflections or composite moduli from offset sensors rather than the sensors near the load plate.

As shown in Table 9, the deflection-based values that show the strongest relationship with the material property data, in general, are the composite surface moduli E_o3 and E_o4. Based on these correlations, the overall observations that can be made regarding the resulting correlations are listed in Table 10. Instead of mentioning these many conclusions in the text, this table has been prepared for better clarity, as kind of an "executive summary".

Based on all of the above, when considering the concept of using some means of measuring soil stiffness in the field during construction as a method of quality control during pavement construction, the LTPP data certainly shows promise. Even though none of the data elements show a strong relationship to deflections or deflection-derived values, they generally show the expected trend.

To further investigate potential relationships between subgrade densities and FWD test results, a plot of the normalized center deflections and the in-situ densities for the 36 sections where both types of data existed (along with laboratory Proctor densities) is plotted in Figure 19. The trend line in Figure 19 is based on *all* of the data points shown, some of which are obvious outliers. The three large open square plot points are from Louisiana's SPS-1 site, while the large open circle point is from Section 08-0221 (Colorado) and the other smaller open circle plot points are from the other Colorado SPS-2 sections in the dataset. A closer investigation has revealed that the subgrade of Louisiana's SPS-1 site was in fact stabilized, which is the overriding reason for the relatively low deflection measurements. The lone point in the upper-right of the plot is from Arizona's SPS-2 Section 04-0213. It can be seen that this section had significantly higher deflections, which were uncharacteristic of the density of the soil and appreciably different from the three other Arizona SPS-2 sections in the dataset that had approximately the same in-situ densities.

Figure 20 shows a strong relationship between the normalized center sensor deflection and in-situ moisture content for most of the data points. The plot points with the different symbols are the same as shown in Figure 19. In both cases, it is quite obvious that without the inclusion of stabilized subgrades and some of the other anomalies noted, the R-squared values would have been much higher, and also the slope of the best fit line would have been somewhat altered.

Table 10. General Conclusions Concerning Correlations of Subgrade Parameters

In-situ density:	The surface modulus increases as the in-situ density increases, as expected
In-situ moisture content:	The surface modulus decreases as the in-situ moisture content increases, as expected.
Maximum density:	The surface modulus increases as the maximum density increases, as expected.
Optimum moisture content:	The surface modulus decreases as the optimum moisture content increases, as expected.
Relative density:	The surface modulus increases as the relative density increases, as expected.
Percent greater than 2 mm:	The surface modulus increases as the percentage of particles over 2 mm increases, as expected.
Percent coarse sand:	The surface modulus has very little correlation with the percentage coarse sand in the soil. We would expect the modulus to increase as the amount of sand increases, but this data set may be dominated by the particle size fraction over 2 mm.
Percent fine sand:	The surface modulus decreases as the percentage of fine sand in the soil increases. Depending on the sample set, this may go the other way, particularly if the soil samples are mostly fine-grained.
Percent silt:	The surface modulus decreases as the percentage of silt increases. This is generally as expected. If the soil was allowed to dry out, the opposite trend might be observed.
Percent clay:	The surface modulus decreases as the percentage of clay increases, as expected.
Liquid Limit:	The surface modulus decreases as the liquid limit increases, as expected.
Plastic Limit:	The surface modulus decreases as the plastic limit increases, as expected.
Plasticity Index:	The surface modulus decreases as the plasticity index increases, as expected.

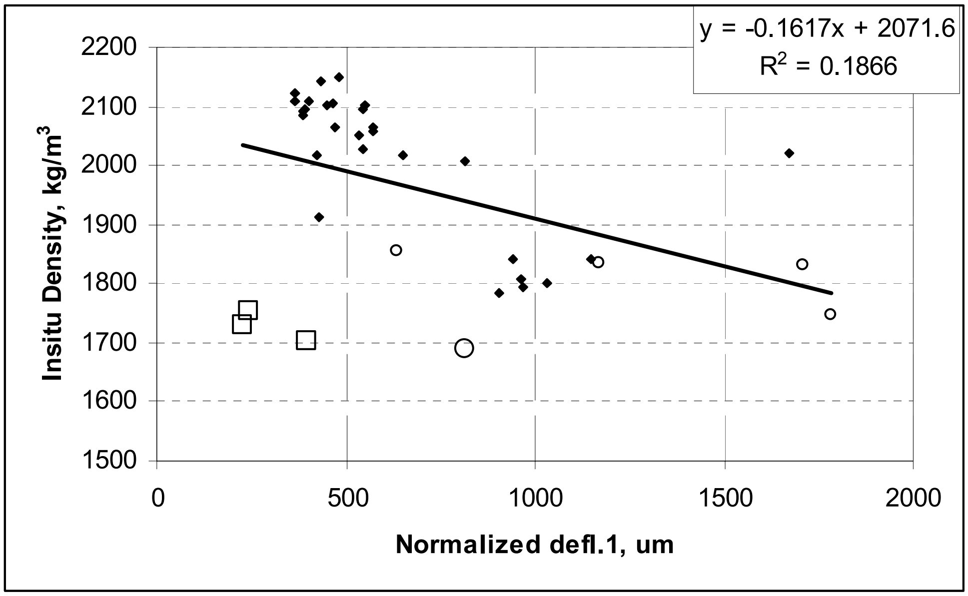

Figure 19. Normalized Center Deflection versus In-situ Density for Subgrades

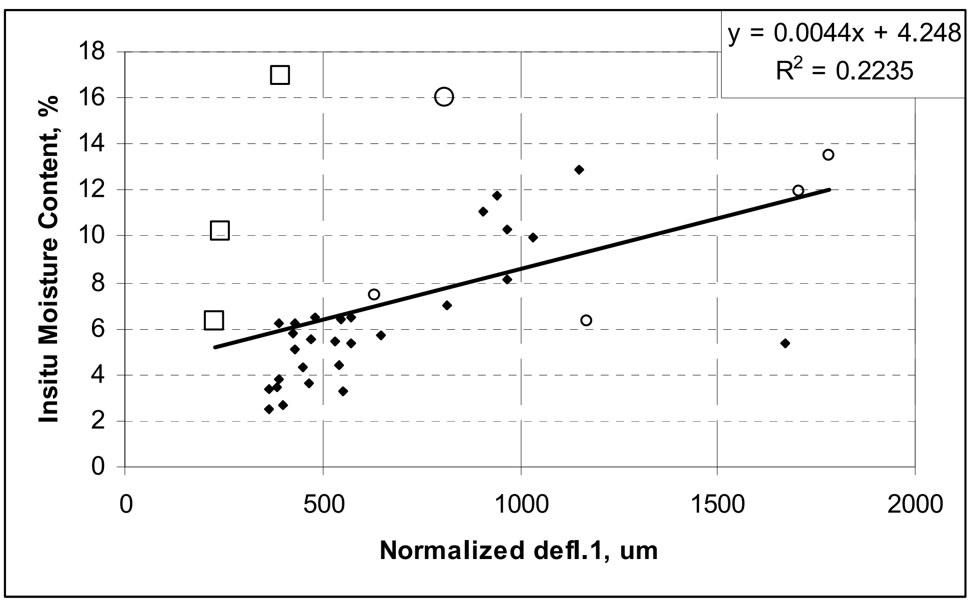

Figure 20. Normalized Center Deflection versus In-situ Moisture for Subgrades

Figure 21 is a comparison of Sensor 4-derived composite modulus from on-grade FWD tests to one particular soil gradation characteristic, i.e. percent retained on the 2-mm sieve. Stronger soil types generally tend to have more coarse material (as defined by the 2-mm sieve size) than softer, weaker soils. This is consistent with the general expectation that coarser grained materials have higher stiffnesses. A more commonly used parameter to describe soils is called the “P200” (percent passing the #200 or 0.074-mm sieve). It is generally considered that the higher the P200 value, the weaker the soil, and Figure 22 bears this out, but shows a lot of scatter. The plotted points in Figure 22 might infer that there could be families of relationships for this parameter.

Analysis of the data set with multiple regression techniques did not provide much improvement over the individual relationships. Table 11 shows one such regression. In this case, the in-situ density, two gradation factors and, to a lesser extent, the inverse of the moisture content can be used to predict the surface moduli with a standard error of estimate of 43 MPa (~6,000 psi), which is roughly one-sixth of the range in section-by-section averages of E_o3.

The plot in Figure 23 compares the $E_0 3$-calculated from the deflections and the $E_0 3$-predicted from the regression coefficients shown in Table 11. This plot shows that the predicted E_o3 values do not span the range that the deflection-based values do. The coefficients tend to over-predict subgrade stiffness for softer soils and under-predict subgrade stiffness for stiffer soil types. One inference is that there are other relevant factors that are not reflected in the subgrade datasets that were created for use in this study.

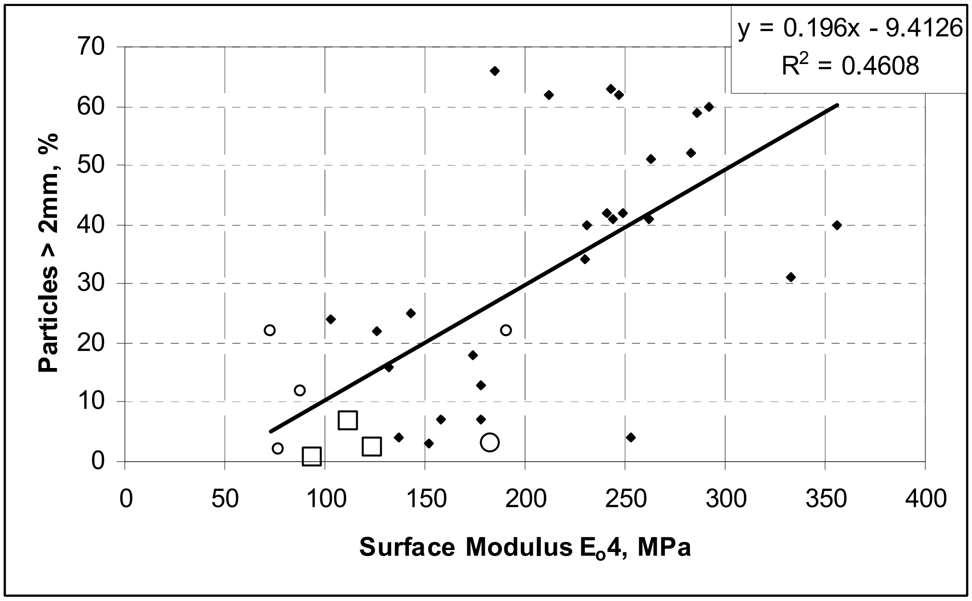

Figure 21. Composite Subgrade Modulus from Sensor 4 versus % Retained on 2-mm Sieve

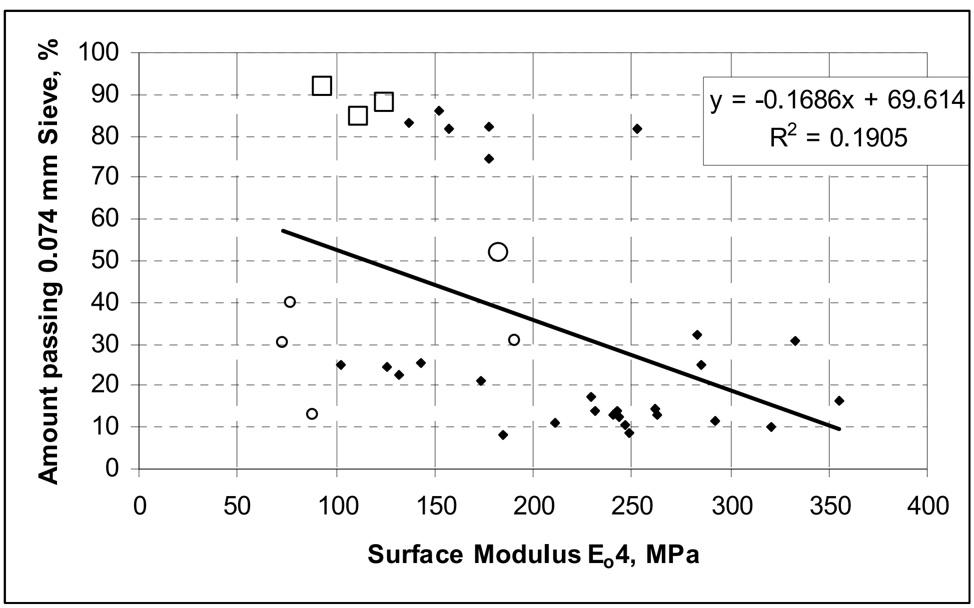

Figure 22. Composite Subgrade Modulus from Sensor 4 versus % Passing 0.074-mm Sieve

Table 11. Regression Statistics of Subgrade Soil Properties versus Composite Moduli

Regression Statistics	
Multiple R	0.7716
R Sqr.	0.5954
Adj. R Sqr.	0.5262
Std. Err.	42.5540
Observations	36

ANOVA

	df	SS	MS	F	Signif. F
Regression	4	85267.8	21317.0	11.7718	6.25046E-06
Residual	32	57947.1	1810.8		
Total	36	143214.9			

	Coef.	Std. Err.	t Stat	P-value	Lower 95%	Upper 95%
Intercept	0	#N/A	#N/A	#N/A	#N/A	#N/A
In-situ Density	0.063	0.010	6.352	0.000	0.043	0.083
Inv. In-situ MC	236.199	119.286	1.980	0.056	-6.778	479.176
Percent >2mm	1.524	0.386	3.945	0.000	0.737	2.311
Percent Coarse Sand	-2.012	0.835	-2.409	0.022	-3.714	-0.311

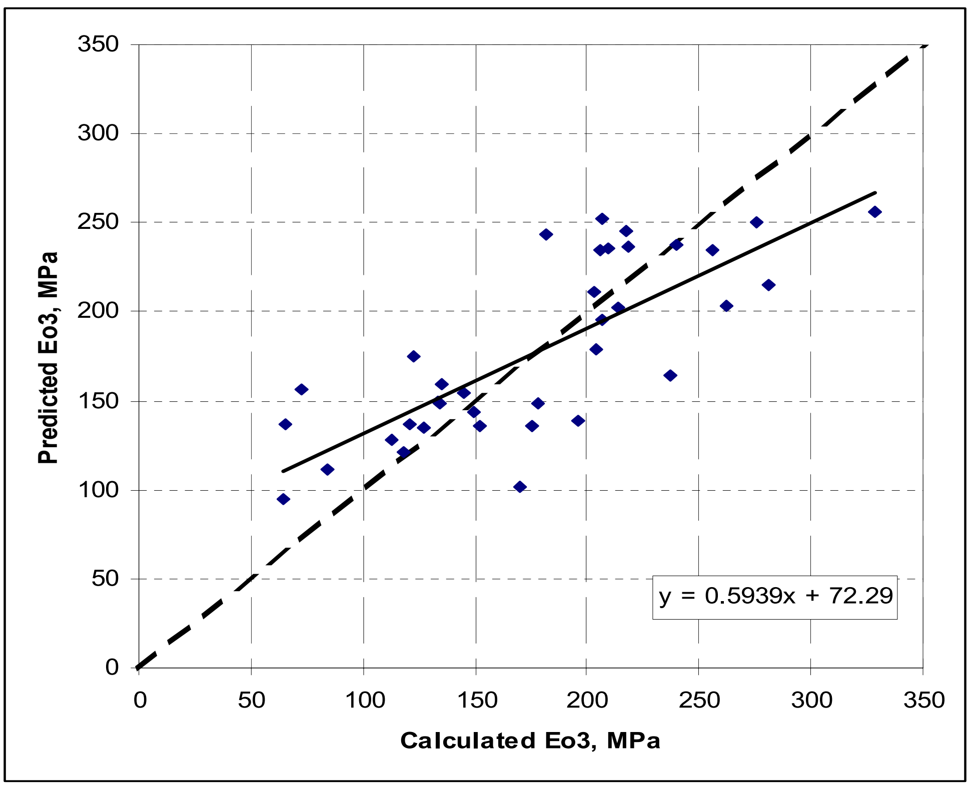

Figure 23. Predicted versus Observed Values for Regression Analysis Shown in Table 11

In addition to the various subgrade soil properties and test results outlined in the foregoing discussion, graphs and tables, laboratory resilient modulus tests were also conducted on unbound materials following LTPP's P46 test method [see FHWA Test Protocol, Reference (9)]. There were 12 sections that had laboratory moduli results and FWD tests, seven in Arizona, five in Colorado, and one in Louisiana. The sections with both sets of data are shown in Table 12.

The data are admittedly quite limited at this time, but the results are both promising and revealing nonetheless. The trend line and corresponding regression equation shown in Figure 24 are for the data from Arizona's SPS-1 and SPS-2 sections. The P46 results are compared to test results, and composite moduli calculations from FWD tests conducted directly on the subgrade. Two of the Colorado sections and the Louisiana section also seem to follow the general trend set by the Arizona sections, while two of the Colorado sections (08-0214 and 08-0221) do not. The two Colorado outliers appear much stiffer in the field than the laboratory results would indicate (see the top two square data points in Figure 24). Section 08-0214 is listed in the IMS as having a coarse grained soil (clayey sand with gravel) and Section 08-0221 is listed as having a fine-grained soil (sandy lean clay). The laboratory tests show both sections having soils that are very dependent on the deviator stress and not very dependent on the bulk stress, which is consistent with typical behavior of fine-grained soils. The deflection basins indicate a much stiffer material at depth, and since the relationship shown is with the surface modulus calculated from subgrade tests and the Sensor 5 deflections, the lower layers have an influence on the resulting moduli.

The laboratory moduli are based on a loading condition of 40 psi (275 kPa) and a confining pressure of 15 psi (10 kPa). Based on the best possible correlation to the P46 test results, the so-called "universal" model equation was changed slightly, to the following form:

$$Log(\frac{M_r}{P_a}) = k_1 + k_2 Log(\frac{\theta}{P_a}) + k_3 Log(\frac{\tau_{oct}}{P_a}) + k_4 (Log(\frac{\tau_{oct}}{P_a}))^2$$

This universal model equation is the same as that recommended by Uzan [see Reference (10)] for modeling unbound materials, except for the second order term of the Log-octahedral shear stress. This term was included because there is a reasonably strong trend for the P46 test results to be somewhat non-linear for the Log-octahedral shear stress term. Many of the materials move from a stress softening to a stress stiffening behavior (or vice-versa) as the deviator stresses increase. Non-linear behavior of the Log-bulk stress is not as common, but can occur in some cases.

The k_1 to k_4 coefficients were determined from the 15-psi stress states used in the P46 test. The loading stress of 40-psi and confining stress of 15-psi were then used to calculate the moduli shown in Table 12. The resulting relationship shown in Figure 24 is excellent, all things considered. As is common when field-derived values are compared to laboratory values, the tendency is for the field values to be somewhat higher than the lab values; in this case, the field-derived moduli from FWD Sensor 5 on the subgrade are about 50% greater than the corresponding Test Method P46-derived laboratory values.

Table 12. Resilient Modulus Results for Test Protocol P46

Section	Layer No.	In-situ Density kg/m^3	In-situ Moisture %	E_O5 MPa	%Rel. Den	Lab.M_r MPa
04-0113	1	2151	6.5	258	107%	188
04-0115	1	2101	3.3	134	99%	115
04-0123	1	2110	2.7	233	98%	158
04-0213	1	2020	5.4	152	99%	99
04-0216	1	2057	5.4	153	99%	100
04-0217	1	2006	7.0	150	98%	108
04-0223	1	1911	5.1	177	92%	130
08-0221	1	1688	16.1	188	91%	47
08-0224	1	1745	13.5	87	92%	95
08-0214	1	1855	7.4	200	93%	85
08-0219	1	1830	11.9	75	92%	98
22-0119	2	1731	6.4	99	95%	75

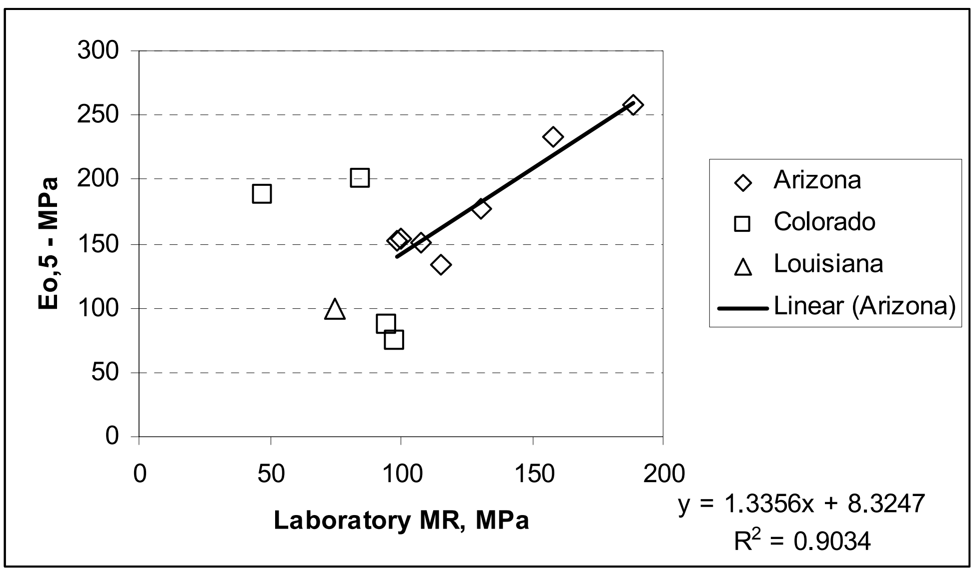

Figure 24. Subgrade Laboratory M_r versus Composite Modulus from FWD Sensor 5

3.2 Comparison of Bound Material Data with Deflection-Associated Data

3.2.1 Bound Base Properties versus FWD-Based Test Results

Although there are some sections in the IMS that contain FWD test results conducted on two types of bound bases (LCB and PATB), there are very few corresponding datasets of other types of QC/QA test results available taken at the same sections. In total, there were only 8 test sections where FWD tests were conducted directly on Lean Concrete Base and 16 sections where FWD tests were conducted on Permeable Asphalt Treated Base (see Table 5). None of the AC sections and only a few of the PCC sections had any conventional QC/QA data such that comparisons could be made with the FWD-derived test results.

Nevertheless, two SPS sites (with 4 test sections in each) were used as examples to show how the FWD can be used to forward-calculate the effective stiffnesses of these two materials (and probably any other bound bases, for that matter). These examples were shown in *Section 2.1.5 – Examples of Bound Layer Stiffnesses Derived from Equation (5),* see Figure 10 and Figure 11 for LCB and PATB, respectively. As can be seen, these forward calculated stiffnesses appear to be quite normal, as would be expected for these two materials.

The fact that there are no comparisons between FWD and other QC/QA data does not at all imply that the FWD is unsuited for QC/QA on bound bases. In fact, based on the forward-calculated values obtained at the few sites where FWD testing did take place, it appears that the FWD may in fact be quite well suited for QC/QA, both in terms of section averages and variability. Since the forward-calculated values are relative, or at best approximate, it may in fact be at least as revealing (in terms of quality control) to delineate spatial variability using the FWD, which can be done relatively quickly and without a major disruption to the construction process. Traditional density QC/QA tests are difficult to apply to PATB, which makes the potential of using a device such as the FWD even more attractive.

3.2.2 Bound AC Surface Course Properties versus FWD-Based Test Results

Similar to the lack of bound layer data for base course materials, there was not an abundance of normal QC/QA or other data collected and stored in the IMS during the flexible pavement type SPS-1 and SPS-8 construction process. For example, very few backcalculated AC layer moduli exist in the computed parameters present in the Level E IMS at this time that pertain specifically to the database in question.

In the case of AC pavements, there were 41 test sections (39 SPS-1 and 2 SPS-8) where laboratory modulus of elasticity existed from LTPP Test Protocol P07 [see FHWA Test Protocol, Reference (11)]. The lab moduli, which were evaluated at three test temperatures, were linearly adjusted to the temperature of the asphalt pavement at the time of FWD test, in order to properly compare the values obtained. The results of this comparison are shown in Figure 25.

The FWD-based forward calculated data are section averages using the so-called “trimmed mean”. In general, there are some 22 FWD test points for each test section, which were generally tested within a few months after construction. All test points were used when forward-calculating the stiffnesses; however, the trimmed mean excludes the highest and lowest value, and takes the mean of the remaining values. This is essentially the same process used in Test Protocol P07, except in this case there are only 6 test results, not 22. Therefore, in the case of the FWD test results, most points in Figure 25 represent an average of 20 forward-calculated values, while with the P07 laboratory moduli, the values represent an average of two trimmed mean samples using 4 of 6 results from each sample, for each of three test temperatures. As previously mentioned, the lab results were adjusted to the field test temperature using the modulus-temperature relationship developed in the laboratory, based on the P07 test results.

The data shown in Figure 25 indicate that there is a fair to good correlation between the FWD-based (forward calculated) values using Equation (5) and the corresponding laboratory-based values from Test Procedure P07. The R-squared of the entire regression, with no data excluded, was 0.59. However, the best-fitting linear relation is not very close to the equation, y = x, especially for the higher values of modulus. Of course, it is not known whether the laboratory procedure has a tendency to produce results that are more uniform than the corresponding field conditions, or whether the forward calculation procedure suggested in this report by Equation (5) is overly sensitive to section-by-section differences in stiffness. Therefore, one test method cannot be preferred over the other based on the regression shown in Figure 25.

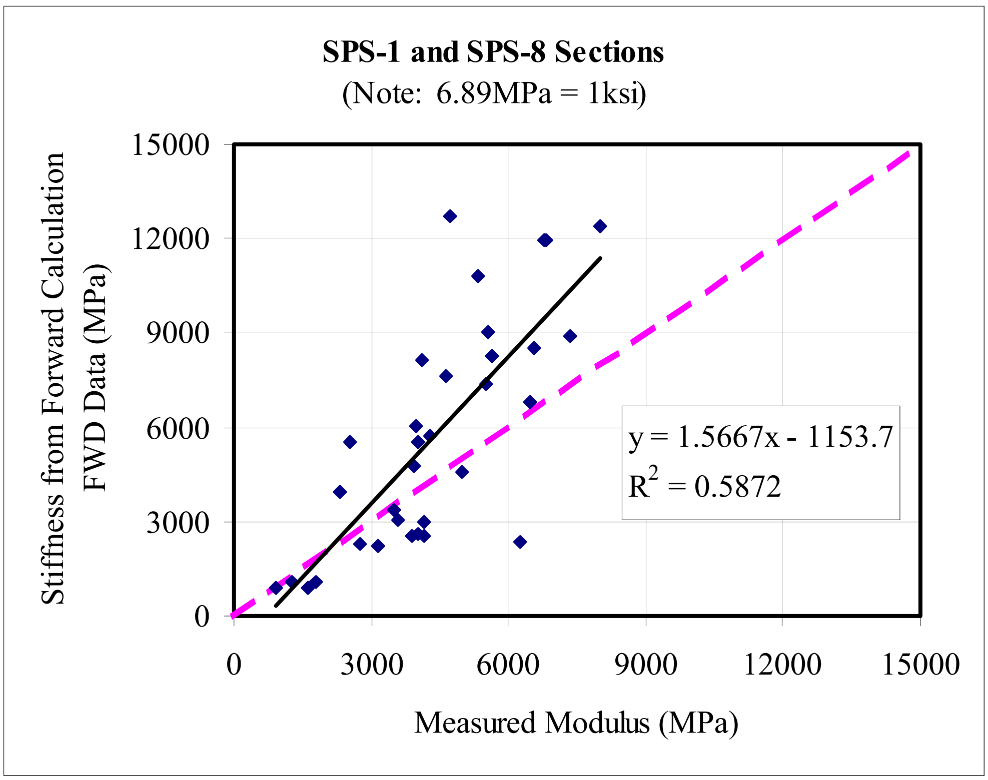

Figure 25. FWD-Based Forward Calculated Stiffnesses versus Lab Moduli for AC Surfaces

Comparison of the laboratory asphalt moduli with the moduli from forward calculations of FWD data on a site-by-site basis shows more encouragement. There are three SPS-1 sites: in Arizona, Louisiana and Montana, each of which had laboratory data for enough sections to look at relationships within these individual SPS sites. The results from the three sites varied, both in goodness of fit and regression coefficients. The Arizona site had the poorest correlation, in large part due to one outlier. The Louisiana results are exceptional, providing an R-squared of 0.92. Repeat tests in the laboratory would be hard-pressed to provide a better relationship.

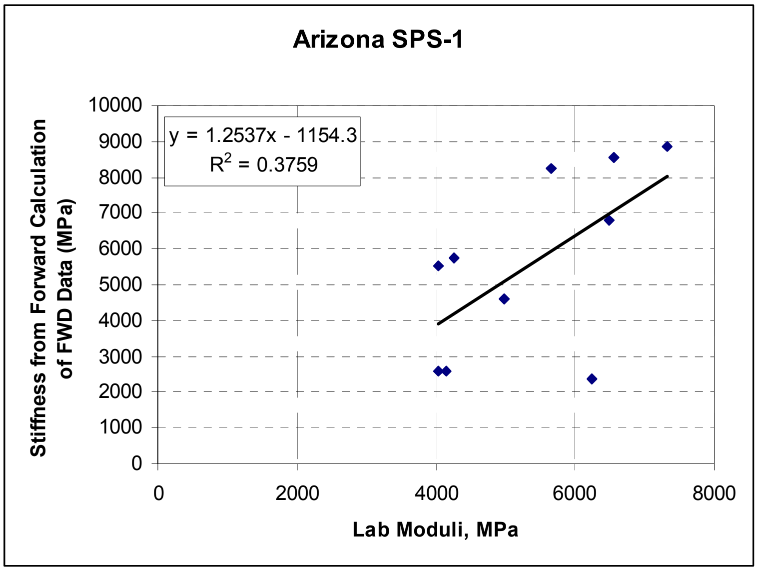

Figure 26. Comparison of Lab and FWD Moduli for the Arizona SPS-1 Site

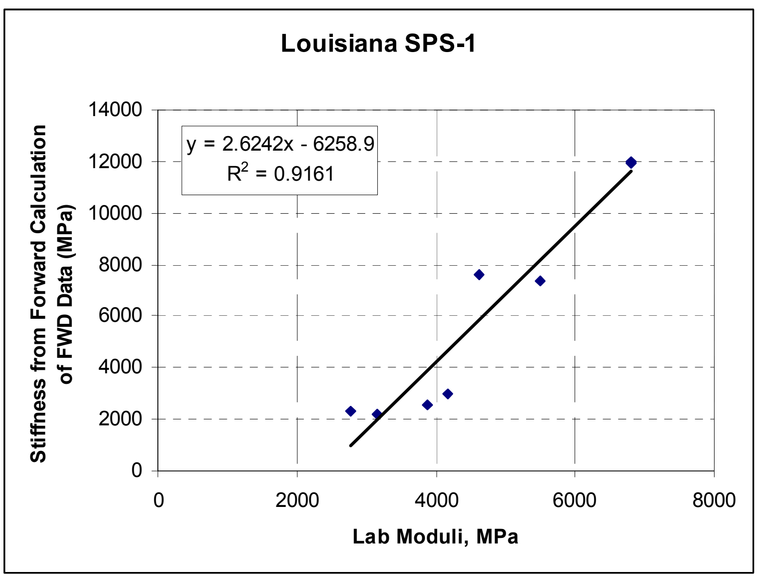

Figure 27. Comparison of Lab and FWD Moduli for the Louisiana SPS-1 Site

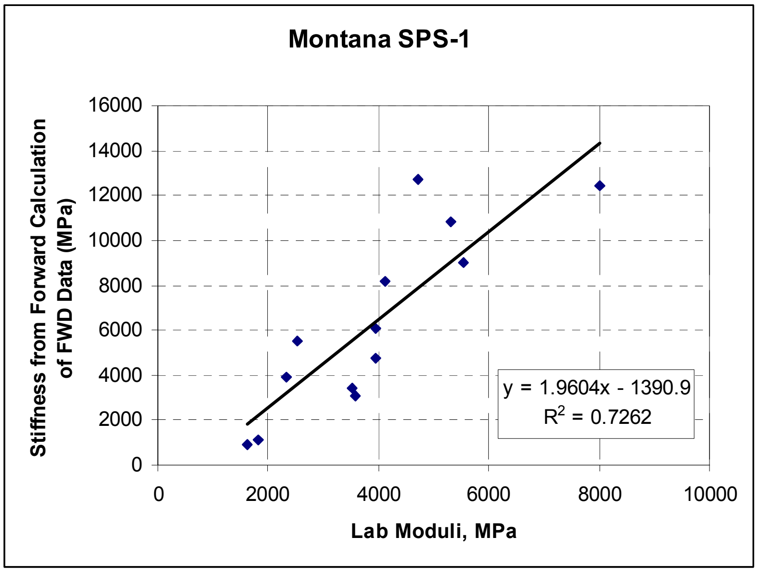

Figure 28. Comparison of Lab and FWD Moduli for the Montana SPS-1 Site

The results from Montana are typical of what were expected for this type of study. When considering the physical nature of the two test methods, it is quite amazing that results can be as good as they were in Louisiana. The laboratory tests are taken on four-inch (100-mm) cores taken from one end or the other of the test section, and tested in the laboratory using the P07 test method, while the FWD tests are a response to the entire pavement system. The resolution of calculated stiffnesses for any particular layer is quite dependent on a number of factors, including the stiffness of the underlying layers, layer uniformity, and stiffness uniformity of the asphalt layers themselves. With advancements in asphalt mix design processes, it might be feasible to supplement quality control for projects with stiffness measurements shortly after lay-down, when the temperature has stabilized. At this point, it is not expected that such a test would ever replace traditional density and volumetric quality control tests, which are strongly related to material durability and its resistance to weathering, raveling and moisture damage.

Based on the above discussion, and on the example data shown previously in *Section 2.2.5 – Examples of Bound Layer Stiffnesses Derived from Equation (5),* see Figure 12, it appears that the FWD could be very useful for quickly testing and calculating an effective stiffness of the AC layer using Equation (5), and also possibly other forward-based calculations (such as Delta2, AREA, etc.). Once again, the variability found as a result of the more extensive coverage afforded by the FWD may be just as important as the mean, or trimmed mean, values so obtained.

3.2.3 Bound PCC Surface Course Properties versus FWD-Based Test Results

In addition to considerable FWD data taken on PCC surfaces shortly after SPS-2 construction, in fact a great deal of other PCC strength data and other concrete properties exists in the IMS. The available data include compressive strength of test cylinders and cores from these test sections, two indicators of concrete tensile strength – modulus of rupture and the split tensile strength, and modulus of elasticity from static tests. It was thus possible to carry out many global (IMS-wide) comparisons of FWD-based data versus PCC strength properties. For the SPS-2 sections, the stiffness determined from FWD testing using drop height 4 was used for all correlation purposes.

First, it was thought that the AREA factor as previously described by Equation (3) could be related to some of the PCC strength properties. Recall that AREA is an indication of the stiffness of the PCC layer in comparison with the stiffness of the underlying material(s). Accordingly, AREA is plotted against three different PCC strength parameters: modulus of rupture (flexural strength), split tension and modulus of elasticity, in Figure 29, Figure 30 and Figure 31, respectively. All three PCC strength parameters were derived from laboratory tests performed at the approximate age when the FWD tests were conducted. The lab modulus of rupture values for only one section in Colorado (08-0224) did not show a positive gradient with time between 14 days and 28 days. Since this was a high strength concrete section, it was reasonable to assume that this section might have had an early strength gain and the 15-day value was used to compare against 28 day FWD test results.

Figure 29 shows that there is virtually no correlation between AREA and M_r-values.

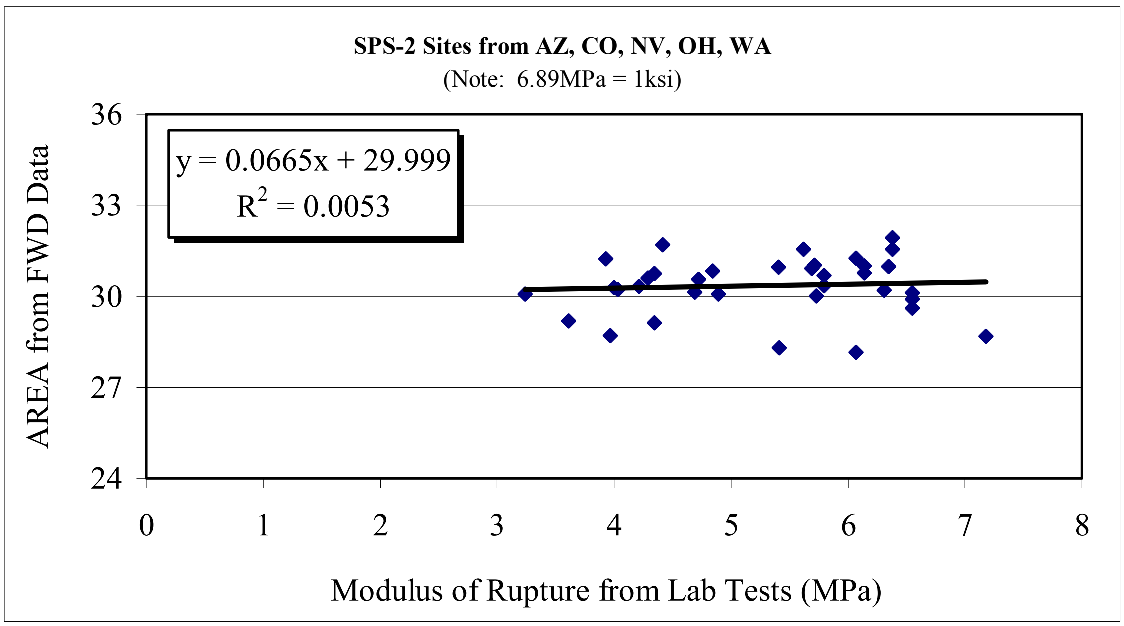

Figure 29. AREA Factor versus Lab Modulus of Rupture for PCC Sections

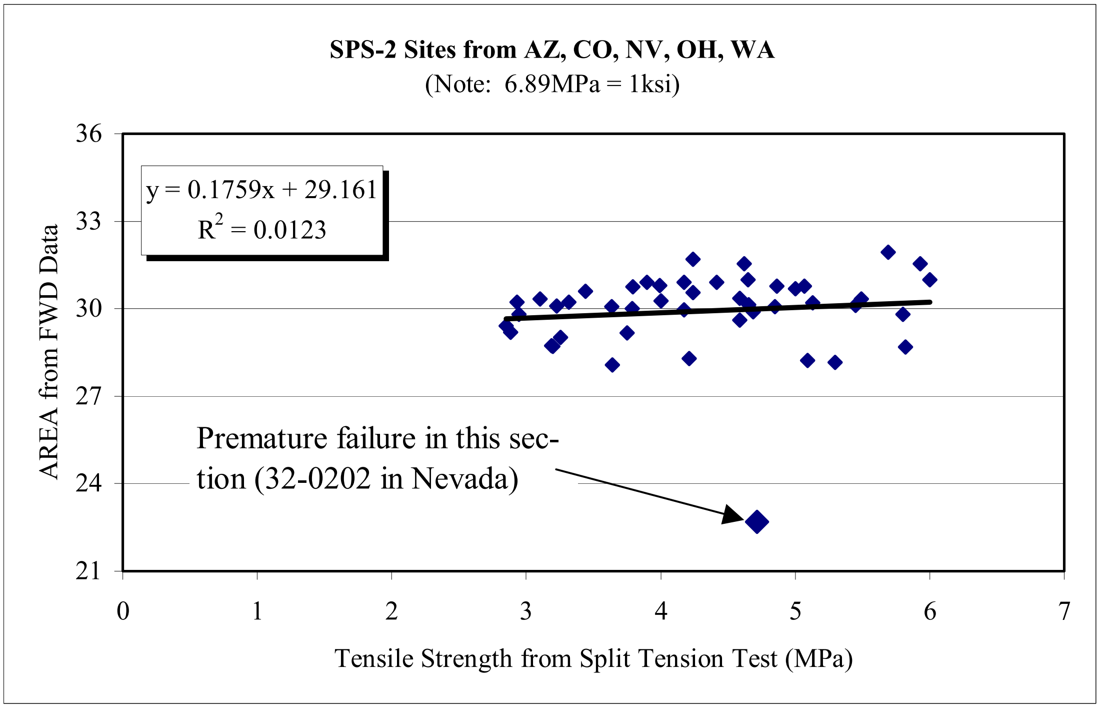

Figure 30. AREA Factor versus Tensile Strength from Split Tension for PCC Sections

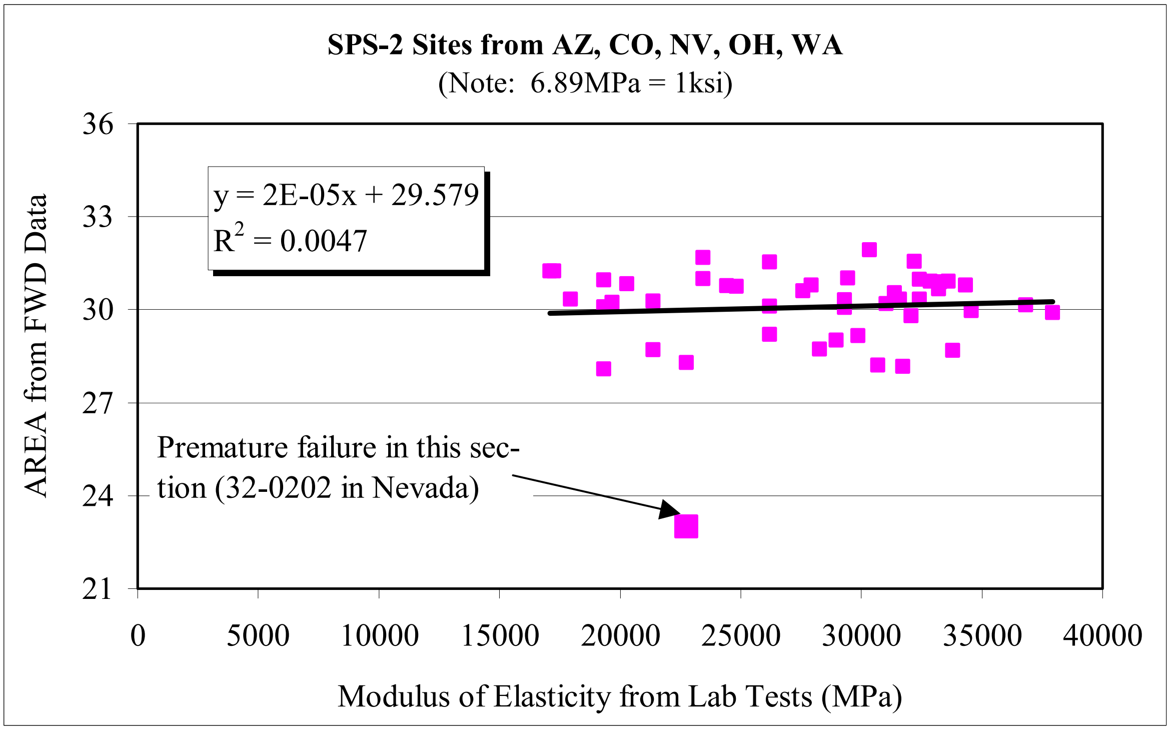

Figure 31. AREA Factor versus Lab Modulus of Elasticity for PCC Sections

At first glance, the situation appears to be the same with respect to split tension and lab modulus tests, as shown in Figure 30 and Figure 31. However, there is one salient test point present in these two figures that was not included in the dataset for modulus of rupture. This is associated with Section 32-0202 (one of Nevada's high-strength SPS-2 sections), as pointed to in Figure 30 and Figure 31.

Rather than just throwing this test point out as an "anomaly", the reasons for such results were investigated. Almost all of the 10 or 11 FWD test points from this test section showed a very low AREA (and forwarded-calculated stiffness), so as far as the FWD test results were concerned, this was clearly ***not*** an anomaly. From the Resident Engineer at the time, a copy of the construction report from LTPP was obtained. It turns out that it was very difficult to achieve the desired 900-psi flexural strengths in Northern Nevada during the mix design phase of the SPS-2 project, while 550-psi strengths were not a problem. Due to the poor aggregate available in this part of the state, no matter what W/C ratio was in the mix design, or what was added, 900-psi flexural strengths were not achievable. Therefore, an additive was found that could, at least in the laboratory, bring the flexural strengths up to approximately 800-psi. However, when the time came to do the actual fieldwork, the accelerator in the concrete was extremely fast-acting, so what left the plant as a 4-inch slump arrived at the job, less than a half-hour later, as a 1-inch slump. The first section poured was Section 32-0202, and it was impossible to finish the concrete with proper vibration, etc., before it had set up. Within days, this section was cracked and was soon taken out of service. Similar problems occurred with some of the other high-strength PCC sections in Nevada as well; however these were not available for the paired database used in the analysis conducted, probably due to the same fast-set problems.

The FWD test results also indicated that the interior slab tests were conducted on cracked PCC. In some instances, the first two or three deflection values were "flat", followed by d3 or d4 with a sharp decline, etc. This means that there were, literally, just pieces of concrete slab, and the FWD was testing these pieces. In other words, there was a kind-of joint behavior everywhere, even at the J1 position.

Based on the above discussion, and on the apparent outlier in Figure 30 and Figure 31, it appears that the FWD picked up a problem with the concrete that the other conventional tests were unable to discern, probably because the test specimens were cast in the field but cured in the laboratory. On the other hand, it seems as if there are a great deal of more subtle differences in the laboratory test results that are not picked up by the use of the AREA factor with the FWD in the field.

To see if the above relationships could be improved in lieu of the AREA factor, the forward calculated stiffnesses from Equation (5) were investigated. First, these stiffnesses are plotted against the tensile strength from split tension tests conducted in the laboratory (see Figure 32). In this case, the correlation is very poor ($R^2 = 0.025$). Once again, the "outlier" from Section 32-0202 is shown, and it can be seen that the FWD detected this imminent failure, while the split tension test did not. The same approach was used to compare forward-calculated stiffnesses [again, from Equation (5)] with the laboratory-determined modulus of elasticity. As can be seen, the slope of the regression line is in the correct direction; however the average values obtained are not equal and the correlation is still poor, although slightly better with $R^2 = 0.05$. Once again, the lab tests were unable to discern any significant problem in a soon-to-fail PCC section, while the FWD-based calculations clearly showed an extremely low effective stiffness in what turned out to be a prematurely cracked concrete pavement.

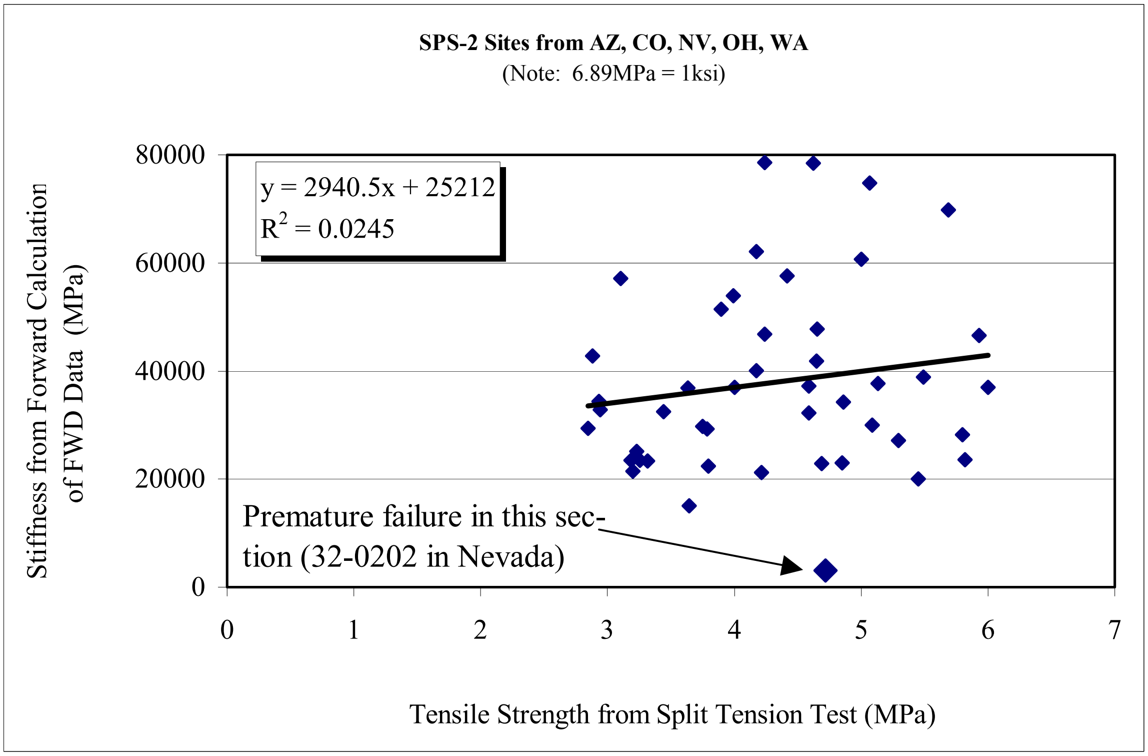

Figure 32. PCC Effective Stiffness versus Tensile Strength from Split Tension Tests

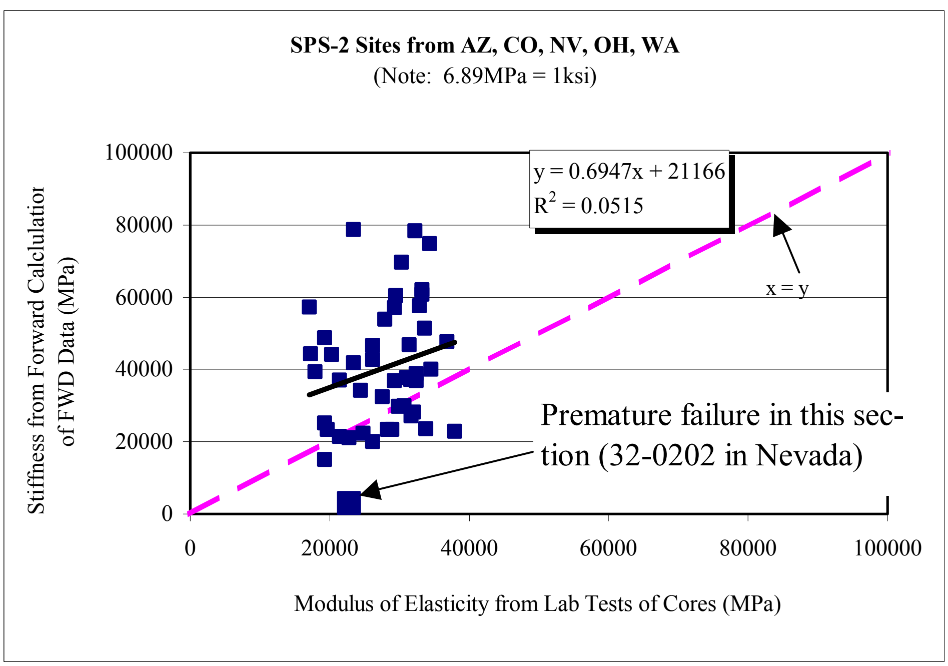

Figure 33. PCC Effective Stiffness versus Lab Modulus of Elasticity from Cores

It is important to note here, from both an engineering and practical point of view, that several factors tend to significantly affect the moduli derived from FWD tests. These factors include the base strength and deformation properties (i.e., the ability of the base to "give"), the bond characteristics at the interface with the underlying base layer, seasonal variations and time of testing that can cause slab curling and warping, and dynamic load application in FWD testing, etc. However, these factors do not in any way affect concrete strength properties from lab tests and hence can contribute partially to the poor correlations observed. The data points used in these correlations included sections from varied climatic zones of the country and different base materials, etc. These correlations are solely geared toward demonstrating the potential of using forward-calculated stiffnesses from FWD data as but one indicator of pavement construction quality.

The next step in the investigation of PCC surfaces was to see whether the laboratory moduli from cores are any better related to the backcalculated moduli in the IMS computed parameter tables. Figure 34 depicts this relationship.

As can be seen in Figure 34, there is also very little relationship between backcalculated and laboratory moduli. In fact, the R-squared value of the best-fit line is less than that shown for the forward calculated stiffnesses, i.e. $R^2 = 0.03$ versus $R^2 = 0.05$, respectively. Not surprisingly, the IMS database contains no backcalculated data for the spurious Nevada Section 32-0202. As a result, this data point is not included in Figure 34.

It was also thought that there may be a better relationship between back- and forward-calculated moduli, or stiffnesses, for the PCC surfaced SPS-2 test sections. The resulting correlation is shown in Figure 35. As can be seen, there is still little or no relationship between the two methods of calculation, whether a dense liquid or elastic solid foundation is assumed in the backcalculation technique employed. It can also be seen that the dense liquid approach results in numbers that are closer to the forward calculated values from Equation (5), and that the forward calculation method appears to be more sensitive to subtle differences in the deflection readings.

It should be noted that ***all*** the FWD data were used in the forward calculation technique, where a "trimmed mean" was used to define the overall effective stiffness of the PCC for a given section, whereas in the backcalculation procedure, only those basins resulting in "reasonable" values were used. Therefore, the backcalculated values represent a smaller sample size, per section, than the forward calculations do. This may explain the primary difference between the two methods of modulus calculation.

Finally, we investigated the differences (if any) between the back- and forward-calculated PCC moduli versus the design (or nominal target) strength of the PCC. In the SPS-2 design matrix, half of the test sections were supposed to be designed for a nominal 28-day flexural strength of 550-psi, while the other half were supposed to be designed for (nominally) 900-psi. In each of the SPS-2 sites, there were up to six "matching pairs" of sections, where the only difference between the two test sections was supposed to be the nominal PCC strength.

The results of these analyses are shown in Figure 36. If indeed the nominal 900-psi PCC sections, in terms of design flexural strength, are "stronger" than the companion nominal 550-psi sections, than it is likely that the forward- or back-calculated moduli will also be higher. In other words, most of the test points plotted in Figure 36 should be ***above*** the 45^O line.

In the case of forward-calculated section averages, seven points are below the 45^{O} line (implying that the nominal 550-psi concrete had a higher modulus, on average, than the nominal 900-psi PCC), while 20 points are on or above the line, as they should be. Please note, too, that one of these seven points is the Nevada Section 32-0202, where the nominal 900-psi concrete cracked almost immediately; therefore, it is not at all surprising that this section had a lower stiffness than the nominal 550-psi companion section. If that case is omitted from a sign-test analysis, there are twenty of twenty-six cases remaining where the forward calculated values are appropriately greater. These results are strongly statistically significant, i.e. that the forward calculated values increase as expected (P-value = 0.0047 for a one-sided sign test). The Wilcoxon signed rank test produces a similar strength of significance (P-value = 0.008), and the t-test is also significant, with a P-value of 0.032. The t-test is less significant because of the two positive and one negative extreme outliers in the paired differences.

No such statistically significant results can be shown for backcalculated moduli. In the case of the backcalculated values, the plot indicates that about half are above the 45^{O} line and half are below (recall that Section 32-0202 is not present in the IMS tables, so this section pair is not plotted). These results indicate that the forward-calculated moduli behave more predictably than the backcalculated moduli.

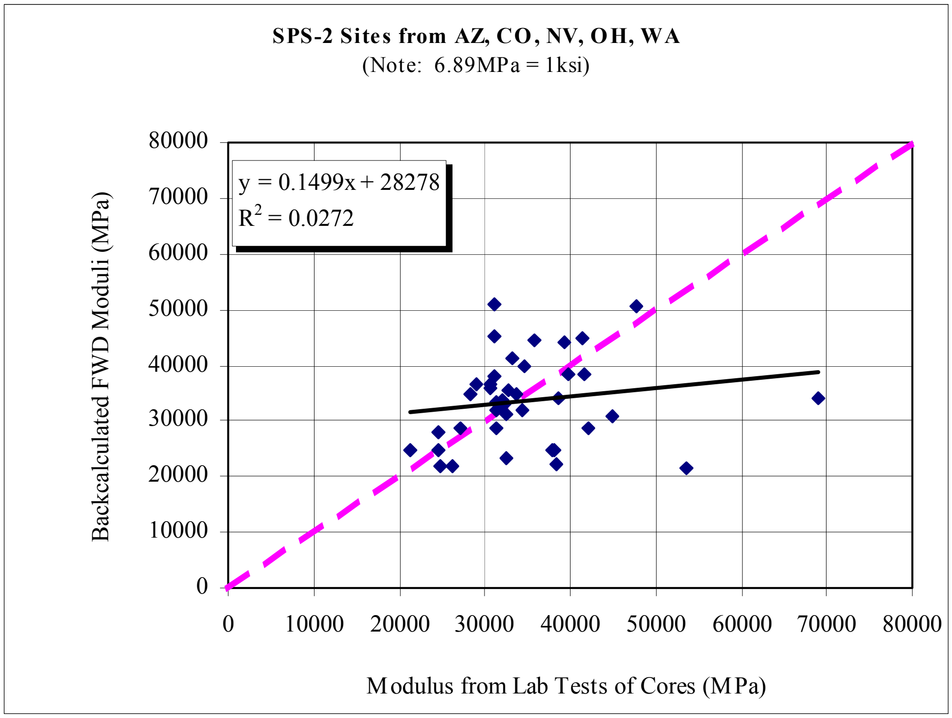

Figure 34. PCC Backcalculated Moduli versus Lab Moduli from Cores

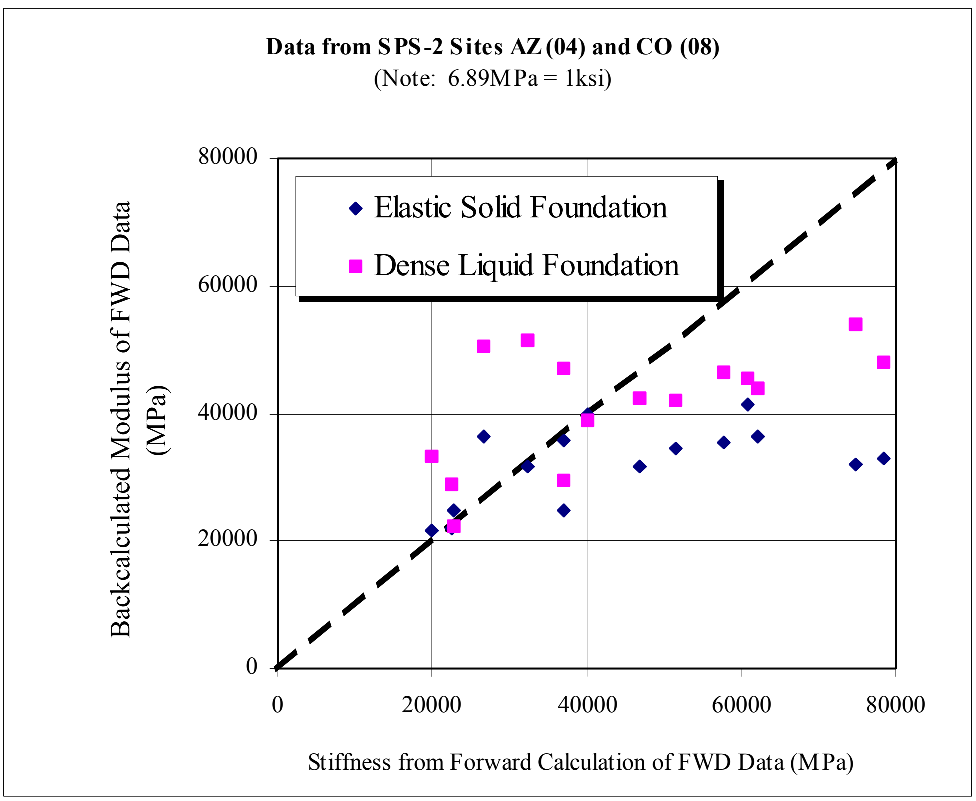

Figure 35. PCC Backcalculated versus Forward Calculated Moduli from FWD Tests

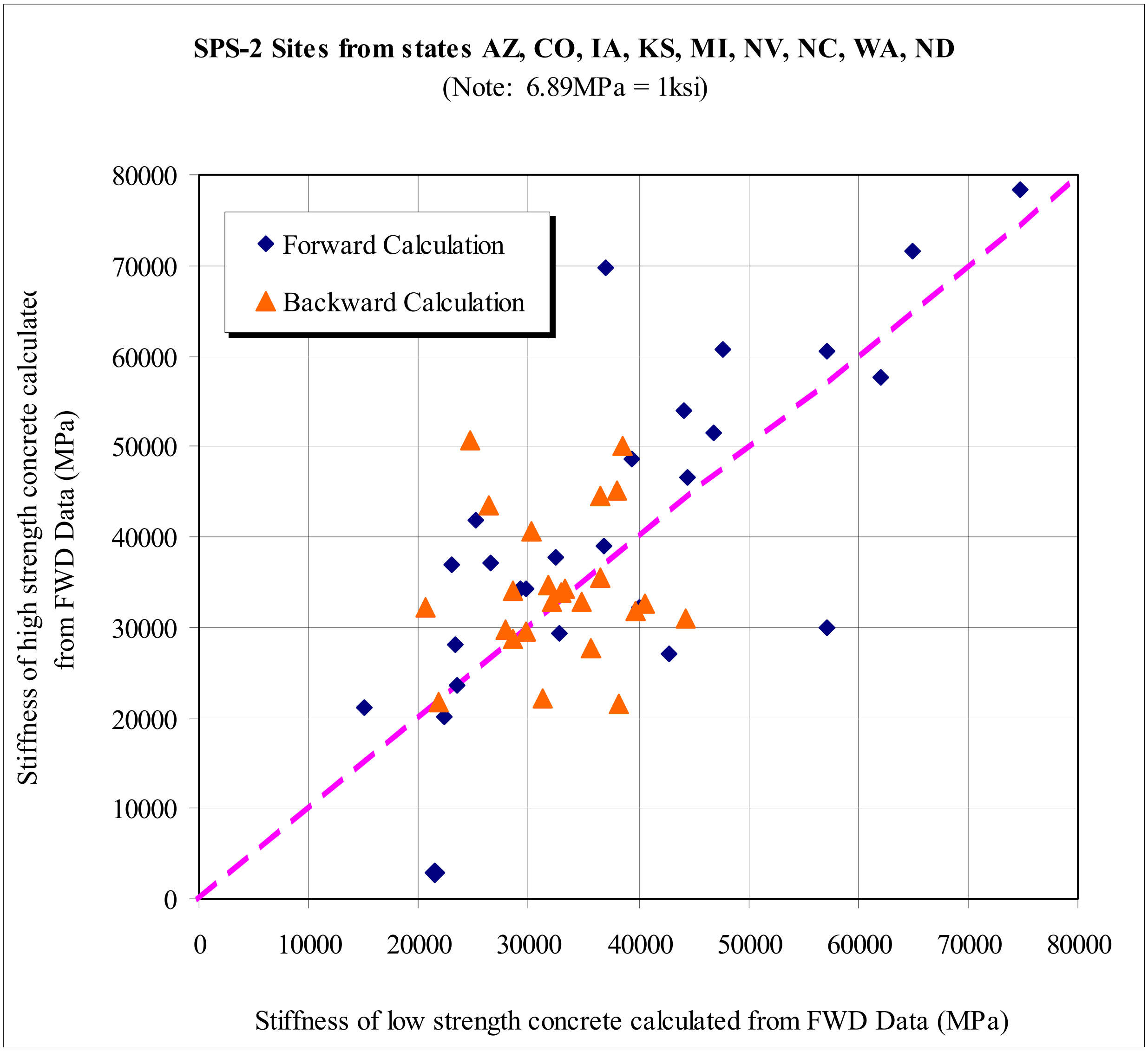

Figure 36. PCC Stiffnesses as a Function of 550- and 900-psi Nominal Flexural Strengths

3.3 Summary of Conventional versus FWD-Based Test Results

From the outset it appeared that the most promising use of the Falling Weight Deflectometer for QC/QA during pavement construction would be in connection with tests on unbound materials, based on the fact that traditional (e.g., density) tests are time consuming and oftentimes difficult to interpret. This could be due to the small sample sizes employed on highly variable materials, and from the difficulty of associating the correct laboratory maximum density and moisture content (Proctors) to the field tests carried out in-situ. It also appeared likely that the FWD could be used to test bound bases as well (e.g., LCB or PATB), since the traditional QC/QA procedures for these materials are also time consuming and only sporadically applied.

Asphalt concrete and (especially) portland cement concrete surfaces are, presently, better defined and controlled during the construction process. They also tend to be more uniform, based on the high quality of contemporary construction equipment and (for the most part) sound construction practices. As long as the proper mix design, surface preparation and application temperature, etc. are adhered to, a reasonable product generally results. However, in Chapters 2 and 3 it was clearly shown how the FWD (even from the deflections alone) detected two separate and significant problems, one with the AC surfacing for Sections 39-0101 and -0105 and the other with the PCC surfacing for Section 32-0202. It may have been possible to detect such problems through other means or tests, but it is clear that there in fact were problems after these surface courses were constructed. Subsequent investigations have revealed that these sections failed prematurely and were removed from the SPS study after a short period of time.

Nevertheless, the promising results when FWD test results were compared to other conventional unbound material properties and tests means that the FWD could, in fact, provide a useful extra tool that could be used in the QC/QA process during pavement construction. This is not to suggest that other tests should be replaced or precluded; only that the FWD can (especially) tell the engineers and/or contractors a lot more about the variations in material quality along a new pavement section, even before these variations and/or input design mean values lead to premature pavement failures.

CHAPTER 4—CONCLUSIONS AND RECOMMENDATIONS

4.1 General Conclusions

The results presented in this report indicate that the Falling Weight Deflectometer (FWD) can be effectively used to assist in the characterization of pavement quality for new or reconstructed pavements during construction. "Quality" can be directly equated to stiffness or modulus, and/or to other deflection-based parameters, in the expectation that these parameters are more directly related to pavement "quality" and, by extension, pavement performance. It is also vital to understand that the spatial variations in these "direct" parameters (e.g., stiffness) are at least as important to pavement quality and performance as the absolute values, or averages. None of the other pavement quality parameters currently used, or readily available, come even close to the sampling rate and, by extension, the ability of a given QC/QA test procedure to assess both the general level of "quality" of a pavement section and the variations in pavement quality along the section. Also, none of the conventional QC/QA parameters also measures stiffness or modulus directly and in-situ, while mechanistic design methods *do* consider pavement layer stiffnesses or moduli. The conclusion to these issues is: The FWD is indeed very effective, if properly used, to characterize pavement construction quality.

FWD results for the subgrade and base materials are, quite possibly, more useful than for the bound surface courses. Even so, there is evidence that the FWD may be useful, even shortly after construction has been completed, based on the forward calculation techniques and analyses performed on AC and PCC surface courses in the LTPP database.

It is doubtful, however, that the FWD alone should be used to assess the *total* quality of any given pavement layer. High quality subgrade and base course materials and good AC and PCC mix designs, for example, along with stringent construction specifications and methods are—and will probably remain—vital to achieving the goal of building high quality and long-lasting pavements. As such, deflection-based methods are recommended as an additional QC/QA procedure, not a replacement for existing methods.

The advantage of using the FWD is even more than its speed of operation. Modulus (or stiffness) is a direct, as opposed to an indirect, indicator of bearing capacity. Clearly, moisture content, density, plasticity index, etc. are all indirect measures of pavement quality. Nevertheless, conventional QC/QA parameters can and certainly do aid the contractor, through standard QC procedures, in locating good materials and achieving adequate compaction and material strengths. Good strength generally will, by and large, result in a higher stiffness and an improved bearing capacity, even without using the FWD to more directly measure pavement strength through modulus of elasticity calculations (whether these are forward- or back-calculated).

In conclusion, the primary findings from this research can be summarized as follows:

1. The research literature, outside of LTPP, contains much information on the use of the FWD for "checking pavement construction quality", or quality control and quality assurance, during pavement construction. However, very little of this literature reached any definite conclusions on exactly *how* this is to be accomplished. An exception to this is the case of the county of Portugal, where the FWD (or HWD) is already used for QC/QA work during pavement construction. The extent of the data available in LTPP,

and the quality of this data, has now made it possible to recommend the development of a deflection-based test method on a more widespread basis.

2. The FWD sampling rate used in LTPP (normally 50-foot test spacings along two test lines) can reveal much more information about the *variability* of the materials along a design segment of new pavement than traditional tests are able to accomplish. For example, one of the most widely used traditional test methods for unbound materials involves moisture-density measurements. In LTPP, these were normally conducted at a rate of three tests per 500-foot test section per layer. In the case of the FWD, generally 22 tests were conducted along the same design segment for each pavement layer. This additional data allows for a more certain result, both in terms of averages and spatial variations, in the material properties encountered.
3. Several simple methods are available for determining pavement layer stiffness, or modulus, that do not require complex and non-unique backcalculation techniques. When these methods are employed, reasonable results (i.e., within expected ranges) are obtained, and the results are *unique* in that they use forward-calculation techniques.
4. Even though an "ideal" test protocol was probably not employed during FWD testing on unbound materials during LTPP pavement construction, the results obtained were very reasonable, both in terms of drop-to-drop variations and deflection magnitudes. The only drawback is the potential overloading of these materials compared to the effect of traffic loadings, which may have biased the results somewhat (but probably not the variations).
5. FWD test results obtained on bound materials, whether bound bases or surface courses, showed very small drop-to-drop variations and several different load levels from which to choose. This is because the FWD sensors and the load plate are always better “seated” on bound (versus unbound) layers.
6. FWD test results “track” (are parallel to one another) reasonably well from layer to layer, with each succeeding layer tests showing somewhat less (but still parallel) variations. In comparing one layer to the next, FWD Sensors 3 or 4 tracked the best on unbound materials while Sensors 6 or 7 tracked best on bound layers. The decreasing deflections (or apparent increasing stiffnesses) found, from layer to layer towards the surface course, were as expected, due to the increasing strength of and confining pressures within each layer as construction proceeds. The one exception to this conclusion was the test results on PCC pavement surfaces, which did not track well with the test results taken on the unbound layers below.
7. The correlations between the FWD-derived unbound material parameters and many of the traditional unbound material parameters were fair to good. This is not surprising, however, in light of the fact that two or more essentially unrelated test methods are being compared. Therefore, none of the traditional parameters or the proposed FWD test method can be invalidated or criticized, one way or the other, based on correlations between these unrelated test methods (e.g., density and stiffness).
8. The correspondence between the FWD-derived bound layer parameters and some of the other available bound layer parameters was also good; an asphalt layer problem from two SPS-1 sections and a concrete layer problem from one SPS-2 test section detected by the FWD during this study were subsequently found to have failed prematurely. In these cases, the pavement surfaces had to be reconstructed and the original sections were taken out of LTPP’s SPS study matrix.

9. The available data set was easily sufficient to detect, using a paired comparison, a statistically significant difference in stiffness, or modulus, in the available SPS-2 sites, corresponding to the difference in the concrete mix design between nominal 550-psi flexural strength and nominal 900-psi flexural strength PCC layers. This difference was detectable using a forward-calculation technique developed during the course of this research project. Available backcalculated parameters in the IMS, however, were unable to detect this difference in apparent stiffness, or modulus.
10. The analyses presented clearly indicate that the FWD can be used effectively to delineate certain important aspects of the quality of new pavement construction. For example, testing just after construction immediately identified that the AC layer of Section 39-0101 produced very large deflections and was not performing as expected. In a real situation, a more detailed study of this situation could have been carried out at the time to determine the cause of the high deflections and very low AC stiffnesses. Accordingly, the pavement areas represented by high deflections would have been repaired or replaced. Similarly, FWD testing can certainly delineate between well- and poorly-compacted base, subbase and subgrade materials, and the spatial variability in compaction and/or other material properties.

4.2 Recommendations

Since only one method or protocol of testing during new construction with the FWD was evaluated, it is difficult to recommend a specific FWD test procedure based on the LTPP database alone. As such, specific recommendations should incorporate additional data and experience from other projects and/or sources of information. Suffice it to say the LTPP protocol was basically a good one, even though some improvements thereto could undoubtedly be recommended. On the other hand, based on available data and the literature study conducted as part of this research, it is by no means certain that any other specific protocol would yield appreciably better, or more accurate, results.

It can only be speculated that a generally lower stress level on unbound materials would be advisable, as long as the specific FWD model (the Dynatest 8000) used to generate the LTPP database was selected for further studies or test protocol (test method) development. Also, depending on the length of the specific project, closer FWD test spacings would also be desirable, since the FWD is indeed very fast and a lot of "territory" can be covered within a short period of time. On the other hand, 22 FWD test points in a 500-foot section of pavement is easily adequate to delineate both reliable averages and standard deviations. Whatever may be chosen as a specific test protocol, based on additional data and further research as mentioned below, the forward calculation techniques that result in elastic stiffnesses or composite (in-situ) moduli, as described in this report, are recommended for general use in the process of developing a specific test method. Again, the optimal or ideal test method(s) may or may not be identical to the LTPP-specified FWD test protocols.

Based on the results of the feasibility study conducted, it is highly recommended that further steps be taken to develop an optimal FWD test method, or methods, that can be used during pavement construction to assist in the QC/QA process, and/or for any other type of pavement quality characterization need. Such methods may or may not be the same as was used during LTPP. In particular, it may be advisable to utilize a larger FWD load plate size when testing unbound materials. More research needs to be carried out to determine the best possible FWD configuration and test spacings.

It is also possible that a lighter-load or more portable FWD would be better suited to testing of unbound materials. Some of these units are already on the market. Many years ago, Dynatest also developed a prototype Light Weight Deflectometer (LWD) that showed great promise. However the market never developed at the time, so the manufacture of this product was suspended. Now, at least two other viable LWDs are available on the market. One of these is an LWD device called the "PRIMA" or "Viatest" and the other is called the "LOADMAN." Other LWDs may also be commercially available.

The FWD is a vehicle-drawn or -mounted device that requires a pavement surface suitable to drive on. This is not always possible during pavement construction. Further, an FWD is expensive and may not always be immediately available. For unbound materials, a lighter-load, more portable, and less expensive device may do just as good a job as the FWD, possibly even better. For surface course tests, however, an FWD appears to be the device of choice, at least considering today's available technology.

4.3 Further Research Needs

The three following research needs could easily be carried out and implemented without further ado, and at a reasonable cost:

1. In this report, it was pointed out that spatial ***variability*** could be at least as important as absolute values, or averages, when using the FWD for quality control or quality assurance during pavement construction. This aspect of the research was not fully explored, mainly because the other traditional test methods did not have the necessary sampling frequency with which to compare spatial variability from FWD test results. All available SPS-1, -2 and -8 FWD data should be evaluated and reported with respect to section-by-section spatial variability, because this very factor may be better related to long-term pavement performance than the averages or trimmed mean values reported in the foregoing.
2. It is very important to gather all available (mostly non-LTPP) data together where the 450-mm (~18") FWD load plate was used on unbound materials, in lieu of the normal 300-mm (~12") load plate. These data need to be processed in a similar manner to that outlined in detail in this report, in Chapters 2 and 3 on unbound materials. It is hoped that the deflections taken closer to, or in the center of, the FWD load plate with a larger plate size, will show more promise and better correlations with parallel QC/QA data. In LTPP, only one SPS site exists where the 450-mm plate was utilized (the SPS-1 site in Kansas). However, it was also utilized at MnROAD (on some GPS sections), in Vermont (where a great deal of new data is available), and in a number of other studies.
3. At the same time, other methods of QC/QA are available and have been utilized during many of the studies mentioned under #1, above. One of these is the Dynamic Cone Penetrometer (DCP), which may also show promise, both as an additional QC/QA device and in correlation with FWD test results on unbound materials. At the same time the 450-mm plate size FWD data is collected, DCP and/or other unbound material test data can be assembled as well. During this research, the variability aspect of both FWD-

associated parameters and other conventional test methods, if possible, should also be investigated.

4. Based on the results of this research, and on the short-term research needs #2 and #3 above, preliminary or trial test protocols can be devised and tested on new construction projects across North America. These tentative protocols can be utilized on a quality control trial basis, without actually using the data to accept or reject a given construction, or layer, during each trial project.

Subsequent to the above-outlined short-term research needs, it is also recommended that the following long-term research goals are funded and carried out:

1. Further research will be needed, in line with the development of test protocols or methods mentioned under short-term research need #3, above, in order to both develop the method (AASHTO or ASTM) and to establish the precision and bias any proposed deflection-based test method.
2. A deflection device more suited to testing of unbound materials should be developed, and existing devices should be investigated. This device may be hand-held, much as the existing LWDs are, or it may be an "intermediate" device, i.e. something between a FWD and a LWD, for example one that can be mounted on a piece of normal construction equipment or a 4-wheel drive ATV, for example. Such a device, if properly designed and developed, may be able to replace the FWD, which presently is fairly expensive and not particularly mobile, for carrying out normal QC/QA work on unbound- or lightly-bound materials.
3. After the above two long-term research projects have been carried out, the FWD or LWD (or other similar device) can be used as part of a design-build performance based specification for pavement construction, most likely on any conceivable pavement material type, with the possible exception of portland cement concrete.

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