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LTPP Data Analysis: Variations in Pavement Design Inputs

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SUMMARY OF FINDINGS

This report presents the results of the entire research effort. The major findings are as follows:

- A review of the available literature yielded some useful information. However this literature did not reveal the variability of all of the important pavement design input parameters. Even where variability was discussed and reported in the literature, the databases used to arrive at the reported variability was generally limited.
- For the five categories of design inputs addressed in this study, the LTPP database yielded very useful information on variably. All important pavement design input parameters are well represented in the database with the possible exception of sufficiently accurate back-calculated moduli and the long-term variability of load transfer efficiency of jointed concrete pavements.
- Variability in design inputs is generally measured in terms of standard deviation or coefficient of variation. More importantly, the use of various "certainty" or confidence levels for pavement design inputs is also reported. Typical values for each major pavement design input category has been developed and are reported in various summary tables, mainly in Chapters 2 and 4.

The impetus behind this report is to encourage the pavement designer to consider pavement *variability* of input design parameters, in addition to the "typical" or average input values normally used. It is oftentimes precisely this variability that can be the ultimate cause of observed pavement distresses, for example roughness or fatigue cracking. This is oftentimes *not* because the pavement is under-designed on an overall basis.

Using typical or “default” pavement design values, most of the pavement section exhibiting early distress may in fact be structurally sound and thus result in an adequate or better-than-planned design life. But the areas where the pavement is weakest are precisely those areas that will exhibit early distresses such as roughness, cracking or rutting. Having an understanding of and quantifying the typical variability associated with pavement design inputs can thus lead to more accurate and a better understanding of mechanistic pavement design input values.

This report quantifies the variability of these key pavement design parameters.

CHAPTER 1 — INTRODUCTION AND RESEARCH APPROACH

1.1 INTRODUCTION

The goals of this research, as stated in the Research Problem Statement and the Project's Objective in the RFP for this project, are reiterated as follows:

RESEARCH PROBLEM STATEMENT

Because of the effects of traffic loads and pavement material properties on pavement performance, understanding the changes in traffic loadings and material properties and the variations in pavement layer thickness is an essential part of the pavement design process. However, the variability of these factors within projects and the changes that occur over long time periods are not well documented. Without this information, selecting appropriate designs for new and rehabilitated pavement structures is a difficult task. The data available from the Long-Term Pavement Performance studies are expected to provide such information.

Research is needed to determine the variability of design inputs within projects as well as the changes that occur over long periods of time. The findings of this research will provide guidance for the design of new and rehabilitated pavement structures.

OBJECTIVE

The objective of this research is to analyze, based on the data available from the LTPP studies, the changes in traffic loadings and material properties and the variations in pavement layer thickness. The research is concerned with the variability within projects and the changes that occur over long time periods; it will not address daily and seasonal changes. The research shall be limited to using the data available in the LTPP Information Management System (IMS) database classified as "Level E".

<u>Pavement Input and Output Design Parameters</u>

Any tenable study of variability in pavement design inputs must also involve a comparison of as-specified (‘input’) and as-built (‘output’) design values wherever possible. Pavement engineers typically base design values on the assumption that the specified or desired values will be obtained during the construction process and that the traffic loading estimates used are accurate, or more accurately it is assumed that these design input values are conservative or on the "safe" side. However in most cases, there has been no effective feedback process to ascertain whether or not the specified/expected design values were in fact achieved, or whether they are reasonable in terms of typical pavement variability as a function of location and/or time.

Mechanistic pavement design procedures require the use of several key input quantities. At a minimum, the stiffness or modulus of each structural layer (including the subgrade), the traffic volume and load spectrum expected over the design life, the environmental effects that may influence the structural design, and the layer thickness are required. Accordingly, the following variables were identified during Phase I of this project as being the most important:

- Pavement layer stiffness (or moduli) and/or other physical properties (e.g., flexural strength) as they vary, both with time and by location, along any given design segment of pavement. These include values of layer modulus, etc. that are generally used in pavement rehabilitation design.
- Traffic loads passing across a design segment of pavement as they change with time.
- The long-term effect of both environmental and load-associated factors, as these factors may affect pavement performance over the lifetime of a given structural pavement section.
- Pavement layer thicknesses and their variations along a given design segment of pavement, and the relationship between the design and as-constructed thicknesses.

Data from the LTPP program have provided an excellent basis upon which to study the variability of these key pavement design input parameters. Both the General Pavement Studies (GPS) and Specific Pavement Studies (SPS) experiments within LTPP contain very useful data available that were used in this research.

1.2 RESEARCH APPROACH USING THE LTPP DATABASE

This section introduces and discusses the Long-Term Pavement Performance (LTPP) program, initiated as part of the Strategic Highway Research Program (SHRP), which now consists of a wealth of data that can be used to ascertain and quantify variations in pavement design input parameters, or variables. This section also discusses the extent of variables present in the LTPP database, and why the existing data is useful, and its potential limitations. Finally, the key design input parameters selected for detailed analysis of their variability characteristics will be introduced and the appropriate LTPP and other generic data element acronyms and respective data tables are identified. In the body of this report (Chapter 2 onwards), these easily recognizable names of the data elements and tables will be used without further clarification, for ease of reading and understanding of this report.

The LTPP data is very useful for purposes of defining the variability of pavement design input parameters because it is a very broad-based study. All 50 states in the United States are represented in the pavement test section database, along with a handful of U.S. Possessions and Canadian provinces. The data thus encompasses virtually every conceivable climatic zone possible. Furthermore, the database is very extensive, with adequate and non-localized data with which to ascertain the variability of the four broad categories of pavement design factors listed in Section 1.1 (Introduction).

It is common in the pavement evaluation sector, whether from the Long-Term Pavement Performance (LTPP) program or otherwise, to utilize acronyms or abbreviations instead of “spelling out” each and every pavement-related name or designation. The following subsection consists of a list of acronyms used in this report. In the next subsection, a list of common pavement-related abbreviations is presented.

1.2.1 Long-Term Pavement Performance Program Acronyms

- FHWA = Federal Highway Administration
- SHRP = Strategic Highway Research Program, initiated during the Reagan administration and after five years turned over to FHWA. The pavement performance portion of this research project was simply called the "LTPP Study" henceforth.
- LTPP = Long-Term Pavement Performance study, which began as part of the SHRP program.
- GPS Section = General Pavement Studies from the LTPP program, an existing pavement section, usually 500 feet (~150 m) in length nominated by a given State or Province for inclusion into a GPS design "matrix" of pavement sections.
- SPS Section = Specific Pavement Studies from the LTPP program, a newly constructed pavement section, or new rehabilitation of an existing pavement section, usually 500 feet (~150 m) in length and constructed during the LTPP program as part of an SPS design "matrix" of pavement sections.
- SMP Section = LTPP's Seasonal Monitoring Program, sections that were also from the GPS or SPS program, tested with the FWD multiple times per year over one or more years.
- Lane 1 = LTPP Testing conducted along a line between the wheel paths.
- Lane 3 = LTPP Testing conducted along a line approximately in the right-hand wheel path.
- Lane F3 = Flexible pavement section tests conducted along Lane 3.
- Lane J1 = Jointed rigid pavement section tests conducted along Lane 1 at an interior slab position.
- Lane J4 = Jointed rigid pavement section FWD tests conducted along Lane 1 at an approach joint.
- Lane J5 = Jointed rigid pavement sections FWD tests conducted along Lane 1 at a leave joint.
- Lane C1 = Continuously reinforced rigid pavement section tests conducted along Lane 1 at an interior 'slab' position.
- Lane C5 = Continuously reinforced rigid section FWD tests conducted along Lane 1 at a leave crack.
- 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).

  Note: Six characters are used to designate sections. The left two characters are the state code. If the third character from the left is a "0" or a letter, the section belongs to an SPS site, designated by the fourth character from the left or third character from the right. If the section is part of an SPS study, the right two characters indicate which design cell the section belongs to.

- 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).
- Level E Data = Data included in the LTPP database that has been screened for quality and conformity to LTPP's QA/QC standards for Level E data.

- LTPP IMS = LTPP's Information Management System, the database that contains all Level E data.
- DataPave = A data CD set that contains most of the Level E data, in database format (Access) and which FHWA has made available to the public. Current version used in this report = DataPave 2.0. [Other Level E data that became available after DataPave 2.0 was released was also requested from the IMS and used in this research.]
- Layer 1, 2, 3 ... = Layer 1 under LTPP's definition is the *bottom* layer (the subgrade), followed by the subbase = Layer 2, the base = Layer 3, etc. In other words, each distinct structural layer, or lift (including the subgrade), numbered consecutively upwards from the bottom to the top of the pavement's structural cross-section. [In a number of sections, the subgrade consists of more than one layer, so Layer 2 is not always the subbase or base.]
- AVC (Traffic Data) = Automated Vehicle Classification traffic data (in terms of vehicle loads and axle configurations) from the LTPP IMS.

1.2.2 Common Pavement-Related Abbreviations

Other common abbreviations and acronyms, not limited to the LTPP program, that are also used in this report include:

- FWD = Falling Weight Deflectometer, load-deflection measuring device.
- AC = Asphalt Concrete (surface course).
- SFC = Surface Friction Course = an open-graded asphalt-bound material, when used considered part of the AC layer in the LTPP program.
- 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).
- AB = Aggregate Base, similar or identical to DGAB.
- AS = Aggregate Subbase, a lesser quality material than DGAB, but still a select material as opposed to natural subgrades or embankments.
- TB = Treated Base, either treated with cement, lime or asphalt in relatively small quantities.
- TS = Treated Subbase, same as TB but generally a lesser quality material.
- JPCP = Jointed Plain Concrete Pavement.
- JRCP = Jointed Reinforced Concrete Pavement.
- CRCP = Continuously Reinforced Concrete Pavement.
- JPC = Jointed Portland Cement pavement.
- JPC-PJ = Jointed Portland Cement pavement with Plain Joints (aggregate interlock).
- JPC-DJ = Jointed Portland Cement pavement with Doweled Joints.
- LTE = Load Transfer Efficiency at JCB joints or corners.

Statistical abbreviations that are used throughout this report include:

- Mean = Same as Average (total divided by number of observations).
- Std.Dev. = Standard Deviation about the Mean.

- CV = Coefficient of Variation, i.e. the Standard Deviation divided by the Mean, usually expressed in percent.

The LTPP database, as presented and made available by the FHWA in DataPave 2.0, has incorporated into it a variety of measurement units, both SI and U.S. Customary. Accordingly, the work conducted during this study was generally carried out in the measurement units presented in DataPave2, which are inconsistent.

In terms of the *variations* in material and other input design quantities, however, the units used should not make any significant difference. Therefore, no attempt was made to change the existing database into a consistent system of units. In the text, dual units are used wherever feasible or if deemed necessary.

Statistical Approach

The most commonly used measure of variability is the standard deviation and/or coefficient of variation, or CV (CV = the standard deviation divided by the mean). Although there are other measures of variability—notably the interquartile range and the mean absolute deviation—the standard deviation is most familiar to pavement engineers and the most readily available in typical software programs. The range is also sometimes used, but it is heavily influenced by unusual data; thus it may not be as useful. However, for two-point data sets, which comprise much of the data from the GPS database, the range and standard deviation are equivalent measures. In such cases, the standard deviation will be equal to 0.707 times the range.

For this project, it is felt that one not only wants to measure the standard deviation of pavement design variables, but also to think of this measure of variability as a quantity or property whose variability itself should be understood. For example, the factors that affect the average variability of a pavement design input quantity, as well as the variability of the standard deviation of individual variables, apparently have not previously been studied. This is also an important part of this study of pavement design input variability.

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CHAPTER 2 — VARIABILITY IN PAVEMENT DESIGN INPUTS

2.1 VARIABILITY OF LAYER THICKNESSES

This section summarizes the research findings of the pavement thickness information in the Long-Term Pavement Performance (LTPP) database. The main source of information on layer thickness variability (spatial) is from the "grid" elevation measurements taken with rod and level, at each new layer interface, during construction of Specific Pavement Studies (SPS) projects. The SPS projects studied represent, respectively, new flexible pavements (SPS-1), new rigid pavements (SPS-2), new asphalt-bound overlays above existing flexible pavements (SPS-5), and new asphalt-bound overlays over existing rigid pavements (SPS-6).

There is a possibility that some of the conclusions drawn from analyzing the substantial database from the SPS projects studied may be biased or incorrect. Two possible sources of bias were identified:

1. *A bias (or difference) due to measurement methods, or the timing thereof.* The SPS data on layer thicknesses contain two types of measured thicknesses. First, there are the “grid data” for all of the 55 (or occasionally fewer) measures made at each section; these were taken at eleven stations from zero to five hundred feet, separated by fifty feet, and at each of these eleven stations, measurements were made at five lateral offsets. In addition, there are two average measures of bound layer thicknesses, primarily from cores taken just outside the ends of each test section. A natural question to ask of the SPS data is: “Are the averages of the grid layer thicknesses significantly different from the averages of the data from the two end-point measurements of the bound layers?” If the answer to this question is “yes,” then other questions that should be answered are: "Does this difference only apply to particular types of layers, or layer sequences? Why is there such a difference? Should the SPS grid data be adjusted to reflect this difference?” The answer to this latter question involves judgment of the reason for the difference, and which of the two averages a pavement expert believes is more nearly correct.
2. *The so-called "Hawthorne Effect".* The Hawthorne Effect is the name for the potential difference between the performance and the results that arise when personnel are aware of careful observation. The SPS studies have this potential difference from the GPS studies, and our analyses measure the extent to which this potential is realized. GPS construction was generally carried out in the states and provinces prior to the advent of the LTPP program, and as such can be regarded as constructed under reasonably normal construction conditions and more typical quality control levels. SPS construction procedures, on the other hand, were carried out under greater scrutiny and over a shorter length of uniform pavement. In addition to the shorter average project length (approx. 500 feet / 236 m for each SPS section), it is well documented in statistical literature that there is a possible effect on any process when the participants in that process are aware of being carefully observed and/or monitored. Good statistical technique requires that the Hawthorne Effect be checked whenever possible.

2.1.1 Layer Thickness Bias Based on Measurement Method

The summary table presented below (Table 1), together with the more complete tables shown in the Appendix A, show that there is a clear pattern of statistically significant differences between the SPS end-point and grid thickness data, *though only in cases where bound layers are built over "weak" or unbound subbases*. This pattern is strong evidence that there is either a bias between the different methods of measuring the thicknesses of these layers, or an actual difference due to construction procedures or techniques.

In fact, there are plausible explanations that there is a natural bias for both asphalt concrete (AC) and Portland cement concrete (PCC) core or test pit measurements to appear thicker, when compared to the corresponding grid layer thicknesses from the same LTPP section. These explanations will be offered towards the end of this section, after the data have been presented.

Before presenting the results of our analyses, some technical notes follow about the use of the term "statistically significant". All statistical comparisons of layer thicknesses were made using t-tests for paired data. This method requires viewing the data as a single sample of differences. The difference between end-point averages (2-point data) and the corresponding grid averages (core average - grid average) was computed for each SPS section in the database. The null hypothesis for this test is that these data come from a population of differences whose mean is zero, i.e.:

$$H_0\text{: } \mu_{\text{core average – grid average}} = 0 \text{ versus } H_{alt}\text{: } \mu_{\text{core average – grid average}} \neq 0$$

When these tests were performed on data that came from AC or PCC layers over *weak* subbases, *all* of these tests showed statistically significant differences from zero. On the other hand, when the same statistical tests were performed on the population of AC or PCC layers over *strong* subbases, none of the tests showed statistically significant differences from zero.

The statistical difference between weak and strong subbases is illustrated in Table 1, where the cutoff (maximum) P-value for statistically significant differences is the normally recommended α-value of 0.05.

Some of the data sets, before combination, (for example, 4 inch over AC over weak subbase) had too small sample sizes and did not necessarily show the statistical significance indicated in Table 1 (see complete data tables in Appendix A). However, *all* of the data from layers over "weak" or unbound subbases showed the same magnitude and direction of the bias as observed in Table 1. When the 4" and 7" (100 mm and 180 mm) AC layers or the 4", 8" and 12" (100, 200 and 300 mm) ATB layers are combined, both the trend and the corresponding statistics are very clear. It was not necessary to do the same with the PCC layers (i.e., combine the 8" and 11" data), but when this is done the same results are obtained: *There is in fact a statistically significant difference in the measured bound layer thicknesses over weak or unbound subbases (including PATB).*

Table 1. Study of Average Layer Thickness Biases for SPS-1 and -2 Sections

Description of Layer	Grid Average Thickness (inches)	Core Average Thickness (inches)	Difference (Core-Grid Thickness) (inches)	Value of T-statistic	Number of Sections	P-value for two-tailed test	Significant at level 0.05?
Combined 4 and 7 inch AC over weak subbase (1)	5.486	5.626	0.140	2.254	29	0.032	Yes
Combined 4, 8 and 12 inch ATB over weak subbase	8.090	8.259	0.169	2.134	39	0.039	Yes
8 inch PCC over weak subbase	8.339	8.592	0.253	2.318	25	0.029	Yes
11 inch PCC over weak subbase	11.193	11.363	0.172	2.487	30	0.019	Yes
Combined 4 and 7 inch AC over strong subbase	5.492	5.506	0.014	0.274	42	0.786	No
8 inch PCC over strong subbase	8.200	8.137	-0.063	-0.819	15	0.427	No
11 inch PCC over strong subbase	11.212	11.106	-0.106	-1.111	17	0.283	No
4 inch PATB (2)	3.966	3.904	-0.062	-0.651	13	0.843	No
6 inch LCB	6.258	6.284	0.026	0.333	22	0.743	No

Notes:

1) Excludes Station 5+00 from nominal 4" Section 350102 where an extreme outlier exists (~7"). Evidently, the transition zone to a thicker section begins at grid Sta.5, which was also much thicker than the rest of the section (~5.5").
2) Excludes Sections 350108 and 350112 where there was either no core data from one end of the test section (350112), or one set of end-point core data appeared to be in a transition zone (350108).

The magnitude of the observed differences is between about 0.15" (4 mm) and 0.25" (6 mm). As can also be seen, there is no consistently clear difference in the case of PATB or LCB over any subbase type (always "weak"), nor is there a difference with either AC or PCC over "strong" or bound subbases (i.e., ATB or LCB). The reason PATB fell into the "weak" subbase group is that permeable asphalt treated base is not compacted because it is designed as a porous drainage layer. Its stiffness or modulus (based on another NCHRP study of LTPP data presently underway) is similar to the stiffness of compacted granular base (DGAB).

Whenever many data sets are subjected to repeat and multiple tests of significance, it is not unusual to obtain some statistically significant results that are spurious. However in the above cases, the pattern that we observe in all of four of the independent tests conducted on reasonably large data sets is conclusive evidence that the observed differences are real. However, the reason, or reasons, for these differences may be different for the cases of AC and PCC surfaced pavements.

The probable reasons for these differences were identified during the course of this project, although these reasons are beyond the project's scope. Nevertheless, it is still worthwhile to speculate about the reasons for the differences encountered in the measured AC and PCC thicknesses because they may be important when interpreting the layer thickness data.

With respect to the observed differences in AC (or ATB) thickness over weak base, discussions with some of the personnel involved in conducting these measurements, from two of the four LTPP regions, were initiated. Based on these discussions and "expert" opinions of the personnel interviewed, it appears most likely that the difference in the observed AC and ATB thicknesses is primarily due to post-compaction of the unbound layer(s) below, during construction of the ATB and/or AC layer(s). In these cases, the cores and test pit thicknesses were the larger of the two by an average of approx. 0.15” (~4 mm). Other factors are possible, for example the adherence or “sticking” of the AC layer to the unbound layer it is placed above due to the occasional use of tack coats or for other reasons. However, this explanation appears unlikely due to the way the AC cores were handled and measured in the laboratory, a rather thorough process that considered these potential shortcomings.

Therefore, the average core and test pit layer thicknesses of AC and ATB are probably the more nearly correct (from a structural strength point of view) of the two measures.

With respect to the observed differences in PCC thickness over weak subbase, the two-tailed test result has a statistically significant P-value, $\alpha < 0.05$ in both instances tested. On the other hand, there is no significant difference in PCC layer thicknesses over strong subbase, or for lean concrete base over weak subbase. Therefore, there is a statistically significant difference between thicknesses measured by cores and/or test pits and grid elevations during PCC construction.

It is proposed that the PCC thickness differences observed are most likely due to the adherence or “infiltration” of the grout in the (wet) concrete mix to the unbound materials below. Thus when cores are extracted, this causes an average additional measured thickness of some 0.2” (~5 mm), with the thicker of the two sets of values being from the cores and test pits. This hypothesis was *not* checked with LTPP regional personnel and is thus the collective opinion of the research team. If so, this additional thickness measurement is probably not structurally equivalent to the grid layer Portland cement concrete thicknesses reported.

Therefore, the average grid layer thicknesses of PCC layers are probably the more nearly correct (from a structural strength point of view) of the two measures.

Conversely, there is no statistically significant difference between the measured thicknesses of *any* PCC or AC layer constructed over “strong” or bound subbases, nor is there any observable difference between the thicknesses measured of PATB or LCB over weak subbases. It is not entirely clear why this latter phenomenon was observed, other than the theory that LCB does not as readily adhere to or infiltrate the unbound materials below due to the leanness of the mix, as does ordinary PCC.

2.1.2 The Hawthorne Effect on SPS Construction Techniques

The end-point standard deviations in the GPS pavement data tables of AC and PCC thicknesses were used to compare with similar values in the SPS data tables, using the following five available and comparable data sets:

1) GPS nom. 3"- 5" AC vs. SPS-1 nom. 4" AC over weak subbases
2) GPS nom. 7"- 9" AC vs. SPS-1 nom. 8" AC over weak subbases
3) GPS nom. 10"- 12" AC vs. SPS-1 nom. 12" AC over weak subbases
4) GPS nom. 7"- 9" PCC vs. SPS-2 nom. 8" PCC
5) GPS nom. 10"- 12" PCC vs. SPS-2 nom. 11" PCC

   Note: 1" = 25.4 mm

The flexible SPS data used to study the Hawthorne effect included all AC layers over weak or unbound subbases only, i.e., where a corresponding data set was available in the GPS end-point database. All rigid SPS data for PCC layers was also included, as the GPS data tables did include similarly bound subbases.

A summary of the statistical analyses conducted to study the Hawthorne Effect of the SPS vs. GPS construction techniques is shown in Table 2. Unlike the study above, which could compare core-data versus grid data from the same section, this study presented no possibility of matched pairs or paired comparison study. These tests view the standard deviations of the two core-data measurements from each section as independent samples from two populations, the GPS core standard deviations and the SPS core standard deviations. Then the hypothesis that the two averages of the standard deviation data from each of these populations is tested: H_0: $\mu_{\text{Std dev. from GPS cores}} = \mu_{\text{Std dev. from SPS cores}}$, with a two-sided alternative hypothesis. While sample standard deviations based on only two points will tend to vary erratically, and not be normally distributed, the two-sample t-test with the sample sizes below are not sensitive to the assumption of normally distributed data.

Note: The sample size of 11 in one case shown in Table 2 is too small to assume this analysis is accurate; however, the general result of this analysis is the weak conclusion that there is no consistent evidence of a Hawthorne Effect. Thus this particular case is not significant.

Table 2. Statistical Analysis of the Hawthorne Effect on GPS vs. SPS Construction

t-Test: Two-Sample Assuming Unequal Variances:		
~4" AC Layers	*GPS nom. 3-5"*	*SPS nom. 4"*
Mean of All 2-point Standard Deviations	0.223054826	0.276043609
Number of Observations	123	52
Degrees of freedom	84	
t Stat	-1.162148658	
P(T>=|t|) two-tail [not significant]	0.24846642	
~8" AC Layers	*GPS nom. 7-9"*	*SPS nom. 8"*
Mean of All 2-point Standard Deviations	0.300278213	0.198873782
Number of Observations	73	32
Degrees of freedom	89	
t Stat	2.294200192	
P(T>=|t|) two-tail [significant]	0.024132465	
~12" AC Layers	*GPS nom. 10-14"*	*SPS nom. 12"*
Mean of All 2-point Standard Deviations	0.429314789	0.507831234
Number of Observations	56	11
Degrees of freedom	12	
t Stat	-0.519493706	
P(T>=|t|) two-tail [not significant]	0.612864344	
~8" PCC Layers	*GPS nom. 7-9"*	*SPS nom. 8"*
Mean of All 2-point Standard Deviations	0.162189527	0.32350135
Number of Observations	143	40
Degrees of freedom	44	
t Stat	-2.675543	
P(T>=|t|) two-tail [significant]	0.01043812	
~11" PCC Layers	*GPS nom. 10-12"*	*SPS nom. 11"*
Mean of All 2-point Standard Deviations	0.227393332	0.22868134
Number of Observations	139	47
Degrees of freedom	103	
t Stat	-0.02896488	
P(T>=|t|) two-tail [not significant]	0.97694865	
NOTE: A few AC sections were not used due to differences in the two tables of Construction Numbers (1 vs. 2).		

The Hawthorne Effect could only be studied on bound surface course data, due to potential non-uniform sections and layer thickness identification problems in the test pits and borings from the GPS experimental test sections (see also *Section 2.1.3,* under the subheading GPS Layer Thickness Variability Based on End-point Data).

The above statistical analyses show that there is no strong indication of a true “Hawthorne Effect” based on the potentially more tightly controlled, uniform SPS sections vs. the state nominated GPS sections (the latter of which in turn were generally much longer). Out of five data sets that could be analyzed and compared, three of these indicate a two-tailed P-value appreciably greater than the typical maximum cut-off value of $\alpha = 0.05$ (i.e., P-values = 0.25, 0.61 and 0.98, respectively). In the two remaining cases, a Hawthorne Effect indeed does appear possible. However, in one case (the nominal 8" AC data sets), the Hawthorne Effect showed results in one direction, with SPS construction showing a more tightly controlled variation in layer thickness than the GPS counterparts. In the other case (the nominal 8" PCC data sets), exactly the opposite occurred, with GPS construction showing a more tightly controlled variation in layer thickness than SPS. Arguments can be made either way, for one type of construction being "better" or more tightly controlled than the other, whether due to worker observation on the length of the construction project where a "uniform section" was constructed. Regardless of the actual reasons, however, it is evident that there is no clear Hawthorne Effect overall, based on the relatively large LTPP database analyzed to study this phenomenon.

The result that there is no Hawthorne Effect is reassuring; the grid thickness data are a better source of information about variations in layer thicknesses, since in most instances they are based on 55 discrete measurements of layer thickness with this input design parameter varying both longitudinally and laterally. The 2-point or end-station data are based on two, somewhat more accurate or “averaged” measurements of layer thickness. However, since the end-point data are already averaged over a relatively small surface area (approx. 3' x 3' or 1m x 1m), the true spatial variation in layer thickness cannot be reliably quantified using these data. The same is true for the SPS layer thickness data, all of which are based on two-point "averaged" data.

The following section, *Section 2.1.3 (Spatial Variations in Layer Thicknesses)*, is based on an analysis of the variation in layer thicknesses using SPS-1, -2, -5 and -6 new construction grid thicknesses.

2.1.3 Spatial Variations in Layer Thicknesses

SPS-1: New Flexible Pavement Construction

An analysis of all grid layer thicknesses in the DataPave 2.0 flexible pavement new construction database resulted in the summary statistics shown in Table 3. "Offset" in Table 3 and in other tables in this chapter refers to the lateral or transverse distance from the right-hand edge of the traffic lane, i.e. approximately the lateral position of the right edge stripe.

Table 3. Statistical Analysis of Grid Layer Thicknesses for SPS-1 Sections

Averages and Standard Deviations for all SPS-1 Sections [see Notes 1a, 1b and 1c]								
Planned Thickness (inches)	Offset 0"-6" Grid (inches)	Offset 36" Grid (inches)	Offset 72" Grid (inches)	Offset 108" Grid (inches)	Offset 138"-144" Grid (inches)	Number of Sections	**Overall Average Grid (inches)**	**Notes**
4" DGAB	3.93 ± 0.43	3.98 ± 0.43	3.95 ± 0.42	3.96 ± 0.43	3.91 ± 0.47	35	**3.95 ± 0.44**	**2)**
8" DGAB	7.93 ± 0.44	7.90 ± 0.42	7.92 ± 0.41	7.95 ± 0.45	7.91 ± 0.52	24	**7.93 ± 0.45**	
12" DGAB	11.97 ± 0.47	11.98 ± 0.44	11.98 ± 0.45	11.95 ± 0.47	11.88 ± 0.54	23	**11.95 ± 0.48**	**3)**
4" PATB	3.95 ± 0.31	3.99 ± 0.30	4.02 ± 0.30	3.99 ± 0.29	3.86 ± 0.32	70	**3.96 ± 0.31**	
4" ATB	4.09 ± 0.28	4.02 ± 0.28	3.98 ± 0.30	4.01 ± 0.28	4.09 ± 0.29	22	**4.04 ± 0.28**	
8" ATB	7.97 ± 0.35	8.03 ± 0.35	8.00 ± 0.31	7.98 ± 0.31	8.02 ± 0.34	36	**8.00 ± 0.33**	**4)**
12" ATB	11.76 ± 0.39	11.81 ± 0.47	11.84 ± 0.34	11.88 ± 0.35	11.91 ± 0.34	24	**11.84 ± 0.38**	**5)**
4" AC+SFC	4.18 ± 0.29	4.13 ± 0.27	4.14 ± 0.26	4.13 ± 0.28	4.14 ± 0.29	72	**4.15 ± 0.28**	**6), 7)**
7" AC+SFC	7.07 ± 0.28	6.99 ± 0.27	6.95 ± 0.27	6.95 ± 0.28	7.00 ± 0.28	71	**6.99 ± 0.28**	**8)**

Notes:

1a) Figures shown represent the overall SPS-1 average of each individual section average and standard deviation.

1b) All statistics exclude any section with fewer than 5 thickness measurements per test line.

1c) "Offset" refers to the transverse distance from the right-hand edge of the traffic lane, or approximately the edge stripe.

2) Includes Section 350106 w/<3" DGAB & 220117 w/>5' DGAB.

3) 12" DGAB statistics exclude Section 050114 (non-protocol ~5" DGAB thicknesses appear to have been built).

4) Includes Section 350103 with < 7" grid thicknesses (cores ~7.5" average).

5) Includes Section 220116 with < 11" grid thicknesses (no cores).

6) 4" nom. AC surfaces slightly skewed, mostly due to ~5" Sections 100107, 220113, *350103* and 400120.

7) A handful of other 4" AC sections appear to be slightly over-designed, possibly in part due to SFCs (esp. Section 350103 with > 5" grid thicknesses & cores ~4.4" average).

8) Includes Section 220114 with >9" grid thicknesses (no cores).

Table 4. Nationwide Statistical Variations for Use in Flexible Pavement Design

Nominal Layer Thickness & Type	Standard Deviation (± inches)	Adjustment for 80th percentile design certainty level (+inches)	Adjustment for 85th percentile design certainty level (+inches)	Adjustment for 90th percentile design certainty level (+inches)	Adjustment for 95th percentile design certainty level (+inches)	Coefficient of Variation (± percent)
4" DGAB	0.44"	0.37"	0.46"	0.57"	0.74"	11.1%
8" DGAB	0.45"	0.38"	0.47"	0.58"	0.75"	5.7%
12" DGAB	0.48"	0.41"	0.50"	0.62"	0.80"	4.0%
4" PATB	0.31"	0.26"	0.32"	0.40"	0.52"	7.8%
4" ATB	0.28"	0.24"	0.29"	0.36"	0.47"	6.9%
8" ATB	0.33"	0.28"	0.35"	0.43"	0.55"	4.1%
12" ATB	0.38"	0.32"	0.40"	0.49"	0.64"	3.2%
4" AC+SFC	0.28"	0.24"	0.29"	0.36"	0.47"	6.7%
7" AC+SFC	0.28"	0.24"	0.29"	0.36"	0.47"	4.0%

An analysis of the effect of offset was performed on the data shown in Table 3. It was found that there is no consistent offset or lateral bias for the various SPS-1 structural layers. Accordingly, it was possible to analyze layer thicknesses as a simple study in variations consisting (primarily) of 55 points per test section, with only one unknown parameter and therefore 54 degrees of freedom in the calculation of standard deviations. The data were also analyzed to insure that they were not significantly different from a normal distribution, and passed these tests for normality.

The useful figures in Table 3 are shown under the column heading "Overall Average Grid (inches)", where the average layer thicknesses, together with the corresponding average standard deviations, are listed. Please keep in mind that these are *averages* of each individual section mean and standard deviation, and as such represent a nationwide average condition, in terms of overall averages and standard deviations, of new pavement construction. Please note as well that the mean AC thicknesses listed are in all likelihood on the low side, assuming that the conclusion of bias in measurement method is correct. This conclusion is: an average of 0.15 inches (4 mm) of post-compaction in the subbase occurs when AC or ATB is constructed directly above weak subbase (see also *Section 2.1.1).* Thus the average grid thicknesses for AC materials shown in Table 3 are slightly on the low side, while the standard deviations are still correct.

It can be seen in Table 3 that the overall nationwide averages were very close to the design or planned thicknesses, for all layers studied. The interesting and more relevant aspects of Table 3 are the listed *variations* in layer thickness, which must ultimately be considered and accounted for in a proper structural pavement design.

There are several ways in which these variations may be characterized. The most common of these is the Coefficient of Variation. However, since the standard deviations found were not directly proportional to the nominal or as-constructed layer thickness, a more relevant measure of construction variability is the standard deviation, or alternatively the use of confidence levels. Through the use of appropriate confidence levels, one could in theory design layer thicknesses on the "high" side in order to insure that the actual pavement thickness encountered at any given point along the roadway (both longitudinally and transversely) is statistically certain to be as designed, or better.

The relevant flexible pavement (SPS-1) statistical variation data that may be used for new flexible pavement design are shown in Table 4. Here, the nominal layer thickness and type is shown, followed by the standard deviation (from Table 3), the 80th, 85th, 90th and 95th percentiles or confidence levels, and finally the coefficient of variation as a percentage of the as-constructed or average layer thickness. It should be kept in mind that the figures in Table 4 represent nationwide averages, which may or may not apply to a given state's or contractor's "track record" in achieving an overall average level of quality control in terms of layer thicknesses. In some instances, the contractor may achieve something better than, in others something not as good as, the values listed in Table 4. This aspect of variability is further dealt with in *Chapter 3 — How Variability Can Effect Pavement Design*.

SPS-2: New Rigid Pavement Construction

The results of the calculations of the summary statistics for all grid layer thicknesses in the DataPave 2.0 rigid pavement new construction database are shown in Table 5.

Table 5. Statistical Analysis of Grid Layer Thicknesses for SPS-2 Sections

Averages and Standard Deviations for all SPS-2 Sections [see Notes 1a, 1b and 1c]									
Planned Thickness (inches)	Offset 0-6" Grid (inches)	Offset 24"-36" Grid (inches)	Offset 60"-72" Grid (inches)	Offset 96"-108" Grid (inches)	Offset 132-144" Grid (inches)	Offset 162"-168" Grid (inches)	Number of Sections	**Overall Avg. Grid (inches)**	**Notes**
4" DGAB	4.13 ± 0.52	4.10 ± 0.36	4.12 ± 0.39	4.05 ± 0.40	4.00 ± 0.41	3.94 ± 0.41	40	**4.07 ± 0.45**	
6" DGAB	6.03 ± 0.40	5.98 ± 0.33	6.01 ± 0.33	6.09 ± 0.33	6.07 ± 0.36	6.11 ± 0.37	36	**6.04 ± 0.38**	**2)**
4" PATB	4.15 ± 0.31	4.17 ± 0.29	4.11 ± 0.31	4.10 ± 0.31	4.14 ± 0.31	4.03 ± 0.33	43	**4.13 ± 0.34**	**3)**
6" LCB	6.26 ± 0.36	6.20 ± 0.31	6.22 ± 0.30	6.22 ± 0.30	6.22 ± 0.32	6.21 ± 0.31	44	**6.22 ± 0.35**	**4)**
8" PCC	8.33 ± 0.35	8.14 ± 0.28	8.22 ± 0.29	8.19 ± 0.28	8.18 ± 0.30	8.36 ± 0.30	64	**8.23 ± 0.34**	**5)**
11" PCC	11.15 ± 0.31	11.15 ± 0.29	11.20 ± 0.31	11.18 ± 0.32	11.17 ± 0.31	11.17 ± 0.31	66	**11.17 ± 0.33**	**6)**

Notes:

1a) Figures shown represent the overall SPS-2 average of each individual section average and standard deviation.

1b) All statistics exclude any section with fewer than 5 thickness measurements per test line.

1c) "Offset" refers to the transverse distance from the right-hand edge of the traffic lane, or approximately the edge stripe.

2) Section 190216 excluded from DGAB analysis (only 3 base thicknesses per test line).

3) 4" Nominal Sections 370209 & 390210 had larger average PATB thicknesses, approx. 5.5" (skewed average results slightly).

4) 6" Nominal Section 050220 had a larger average LCB thickness, approx. 7.5" (skewed average results slightly).

5) 8" Nominal Section 370202 had a larger average PCC thickness, approx. 10" (skewed average results slightly).

6) 11" Nominal Sections 100212 & 190223 had larger average PCC thicknesses, approx. 12.5" (skewed average results slightly).

Table 6. Nationwide Statistical Variations for Use in Rigid Pavement Design

Nominal Layer Thickness & Type	Standard Deviation (± inches)	Adjustment for 80th percentile design certainty level (+inches)	Adjustment for 85th percentile design certainty level (+inches)	Adjustment for 90th percentile design certainty level (+inches)	Adjustment for 95th percentile design certainty level (+inches)	Coefficient of Variation (± percent)
4" DGAB	0.45"	0.38"	0.47"	0.58"	0.75"	11.1%
6" DGAB	0.38"	0.32"	0.40"	0.49"	0.64"	6.3%
4" PATB	0.34"	0.29"	0.36"	0.44"	0.57"	8.2%
6" LCB	0.35"	0.30"	0.37"	0.45"	0.59"	5.6%
8" PCC	0.34"	0.29"	0.36"	0.44"	0.57"	4.1%
11" PCC	0.33"	0.28"	0.35"	0.43"	0.55"	3.0%

As was done for the SPS-1 data, these data were examined for an offset effect, and for departures from the assumption of normality. The conclusions were the same, i.e. that there is no offset effect and the data are not significantly different from a normal distribution, and the standard deviation has 54 degrees of freedom.

The relevant figures in Table 5 are shown under the column heading "Overall Avg. Grid (inches)", where the average thicknesses together with the corresponding average standard deviations are listed. Please keep in mind that these are averages of each individual section mean and standard deviation, and as such represent a nationwide average condition of new pavement construction. Please note as well that in this case the PCC average thicknesses listed may be slightly less than those measured through cores or test pits. In these cases, approximately 0.2 inches (5 mm) of additional (but probably non-structural) thickness is evident when PCC is constructed on weak subbase (see also *Section 2.1.1)* and measured after construction has been completed.

It can be seen in Table 5 that the overall nationwide averages were very close to the design or planned thicknesses, for all layers studied, and that each average thickness is slightly greater than the nominal or design thickness. The more relevant aspects of Table 5 are the listed *variations* in layer thickness, which must ultimately be considered in a proper structural pavement design.

There are several ways in which these variations can be characterized. The most common of these is the use of the coefficient of variation. However, since the standard deviations found were not directly proportional to layer thickness, a more relevant measure of construction variability is the standard deviation, or alternatively the use of confidence levels. Through the use of appropriate confidence levels, one can design layer thicknesses on the "high" side in order to insure that the actual pavement thickness encountered at any given point along the roadway (both longitudinally and transversely) is statistically certain to be as designed, or better.

The relevant rigid pavement (SPS-2) statistical variation data that may be used for new rigid pavement design are shown in Table 6. Here, the nominal layer thickness and type is shown, followed by the standard deviation (from Table 5), the 80^{th}, 85^{th}, 90^{th} and 95^{th} percentiles or confidence levels, and finally the coefficient of variation as a percentage of the intended or as-constructed average layer thickness. It should be kept in mind that the figures in Table 6 represent nationwide averages, which may or may not apply to a given state's or contractor's "track record" in achieving this overall average level of quality control in terms of layer thicknesses. In some instances, the contractor may achieve something better than, in others something not as good as, the values listed. This aspect of variability is further dealt with in *Chapter 3 — How Variability Can Effect Pavement Design*.

SPS-5 and -6: AC Overlay Construction on Existing Flexible and Rigid Pavements

An analysis of all grid layer thicknesses in the DataPave 2.0 asphalt-bound overlay construction database resulted in the summary statistics shown in Table 7.

Table 7. Statistical Analysis of Grid Layer Thicknesses for SPS-5 and -6 Sections

Averages and Standard Deviations for all SPS-5&6 Sections [see Notes 1a, 1b and 1c]								
Planned Thickness (inches)	Offset 0" Grid (inches)	Offset 24-40" Grid (inches)	Offset 60-82" Grid (inches)	Offset 96-118" Grid (inches)	Offset 132"-151" Grid (inches)	Number of Sections	**Overall Average Overlay (inches)**	**Notes**
SPS-5:								
2" AC Overlay	2.07 ± 0.27	2.33 ± 0.25	2.21 ± 0.25	2.43 ± 0.23	2.23 ± 0.25	41	**2.25 ± 0.26**	**2)**
5" AC Overlay	4.62 ± 0.34	4.93 ± 0.32	4.82 ± 0.27	5.02 ± 0.26	4.81 ± 0.27	40	**4.84 ± 0.30**	**2)**
SPS-6:								
4" AC Overlay	4.12 ± 0.38	4.18 ± 0.35	4.16 ± 0.34	4.21 ± 0.34	4.18 ± 0.31	20	**4.17 ± 0.38**	**3)**
8" AC Overlay	7.86 ± 0.34	8.08 ± 0.49	8.22 ± 0.37	8.37 ± 0.40	8.34 ± 0.39	5	**8.17 ± 0.49**	**3)**

Notes:

1a) Figures shown represent the overall SPS-5 and -6 average of each individual section average and standard deviation.

1b) All statistics include at least 9 thickness measurements per test line (offset); offset distances varied between sections.

1c) "Offset" refers to the transverse distance from the right-hand edge of the traffic lane, or approximately the edge stripe.

2) Overall grid standard deviation is pooled using all five offsets (adjusted to exclude lateral variations from rutting).

3) Grid standard deviation not pooled (utilized entire grid thicknesses to calculate standard deviation and percentiles).

Table 8. Average Nationwide Statistical Variations for Use in AC Overlay Design

Nominal Layer Thickness & Type	Standard Deviation (± inches)	Adjustment for 80th percentile design certainty level (+inches)	Adjustment for 85th percentile design certainty level (+inches)	Adjustment for 90th percentile design certainty level (+inches)	Adjustment for 95th percentile design certainty level (+inches)	Coefficient of Variation (± percent)
SPS-5:						
2" Overlay	0.26"	0.22"	0.27"	0.33"	0.43"	12.4%
5" Overlay	0.30"	0.25"	0.31"	0.39"	0.50"	6.3%
SPS-6:						
4" Overlay	0.38"	0.32"	0.40"	0.50"	0.64"	9.1%
8" Overlay	0.49"	0.42"	0.51"	0.64"	0.82"	6.1%

As was done in the case of SPS-1 and -2 projects, an analysis of the effect of offset was performed on the data shown in Table 7. It was found that there is no consistent offset bias for the two SPS-6 nominal structural layers analyzed. It was therefore possible to analyze the layer thicknesses for SPS-6 overlays as a simple study in variations consisting (primarily) of 55 points per test section, as before. However in the case of the SPS-5 overlays, there ***is*** an offset effect in overlay thickness, with the greatest average thicknesses occurring at offsets ~36" (~1m) and ~108" (~3m), respectively. This is not surprising in that some of the overlays occurred over rutted flexible pavements, and these two offsets usually fall within the wheel paths. Therefore, it was necessary to adjust the variability of the overlay thickness due to rutting out of the normal variability in overlay thickness. Accordingly, SPS-5 variability is presented herein as a function of stationing only, not stationing and offset combined. The data were also analyzed to insure that they were not statistically significantly different from a normal distribution. Once again, no statistically significant difference was found.

The relevant figures in Table 7 are shown under the column heading "Overall Average Overlay (inches)", where the average AC overlay thicknesses, together with the corresponding average standard deviations, are listed. Please keep in mind that these are averages of each individual section mean and standard deviation (adjusted for offset in the case of SPS-5), and as such represent a nationwide average condition of new AC overlay construction.

It can be seen in Table 7 that the overall nationwide mean overlay thicknesses were reasonably close to the design or planned thicknesses, for all four overlay situations studied. The interesting and more relevant aspects of Table 7 are the listed ***variations*** in overlay thickness, which must ultimately be considered in a proper structural overlay design. If there is rutting prior to overlaying, this variation must also be considered as a separate (but equally important) issue when calculating requisite material quantities and layer thicknesses.

There are several ways in which these variations can be characterized. The most common of these is the use of the coefficient of variation. However, since the standard deviations found were not directly proportional to overlay thickness, a more relevant measure of construction variability is the standard deviation, or alternatively the use of confidence levels. Through the use of appropriate confidence levels, one can then design AC overlay thicknesses on the "high" side in order to insure that the actual overlay thickness encountered at any given point along the roadway (longitudinally only, in the case of SPS-5) is reasonably certain to be as-designed, or better.

The relevant overlay (SPS-5 and -6) statistical variation data that may be used for new asphalt-bound overlay design are shown in Table 8. Here, the nominal layer thickness and type is shown, followed by the standard deviation (from Table 7), the 80^{th}, 85^{th}, 90^{th} and 95^{th} percentiles or confidence levels, and finally the coefficient of variation as a percentage of the intended (average) overlay thickness.

It should be kept in mind that the figures in Table 8 represent nationwide averages, which may or may not apply to a given state's or contractor's "track record" in achieving an overall average level of quality control in terms of asphalt-bound overlay thicknesses. In some instances, the contractor may achieve something better than, in others something not so good as, the values listed. Again, the variation in overlay thicknesses as a result of any rutting present should also be considered. These aspects of variability are further dealt with in *Chapter 3 — How Variability Can Effect Pavement Design*.

GPS Layer Thickness Variability Based on End-point Data

An analysis of all end-point (or 2-point) layer thicknesses in the DataPave 2.0 GPS database resulted in the summary statistics shown in Table 9 and Table 10. Only nominal layer thicknesses similar to those encountered in the corresponding SPS databases are listed, for purposes of comparison. Please refer to Appendix A for a complete tabular listing of all GPS data analyzed.

It should be noted that there were a number of outliers in the database utilized to arrive at the values listed in Table 9 and Table 10. In the cases of AC and PCC layers, no outliers were found. However in the cases of unbound and bound bases or subbases, a total of 50 outliers out of 905 sections analyzed (~6%) were identified and omitted from the statistical database for further analyses. These outliers were generally due to unreasonably large differences in the base and/or subbase layer thicknesses measured at each end of a given SPS section, or to differences at each end in the material identification associated with the various layers encountered. In some of these cases, the section in question may either have been non-uniform or there was an unclear boundary between the base, subbase and/or subgrade layers. It is also possible that the state-nominated GPS section was in fact a combination of two or more design thicknesses, due to local variations in materials, cuts or fills, etc. In each case, the various bound layers were combined to arrive at an overall AC or PCC thickness. Similarly, the unbound base and subbase layers identified as such in the GPS database were also combined, as were the bound base layers (when there was more than one layer listed), to arrive at an overall GB+GS or TB+TS layer thickness shown in Table 9 and Table 10.

In terms of the variation in layer thicknesses observed in GPS vs. SPS construction and the different measurement techniques employed in LTPP, it can be seen in Table 9 and Table 10 that the bound layer thickness variations were similar to those encountered in the SPS database. This was especially true when the 2-point GPS data was compared to the corresponding 2-point SPS data in lieu of the 55-point grid data. On the other hand, the variation in unbound base and subbase (GB & GS) layer thickness is decidedly higher in the case of the GPS measurements when compared to SPS data — whether end-point or grid data. This may however be due to material identification problems or non-uniform sections present in GPS, as mentioned above.

It is therefore suggested that the layer thicknesses measured by the SPS grid layer measurements are exclusively used to quantify layer thickness variations of unbound layers for purposes of new pavement design inputs. It can also be seen in Table 9 and Table 10 that bound base layers reveal considerably smaller thickness variations than unbound base layers. On the other hand, it is probably not as difficult to identify a bound base layer in the field, in the case of GPS construction. Bound bases in the case of GPS construction show similar levels of layer thickness variation to the PATB and LCB material types associated with SPS construction. It is therefore suggested that the grid layer thickness variation data for these two types of SPS layers may also be expected for other types of bound bases or subbases.

Table 9. Summary of Relevant Two-point Variations for Flexible GPS Sections

Standard Deviation	Section Count	Nominal or Design Thickness (inches)
AC Thicknesses (GPS Flexible):		
0.22"	123	Nominal 3"- 5" incl.
0.30"	73	Nominal 7"- 9" incl.
0.43"	56	Nominal 10"- 14" incl.
0.28"	411	Overall AC
GB & GS Thicknesses (GPS Flexible):		
0.74"	29	Nominal 3"- 5" incl.
1.22"	55	Nominal 7"- 9" incl.
0.77"	90	Nominal 10"- 14" incl.
1.25"	307	Overall GB & GS
TB & TS Thicknesses (GPS Flexible):		
0.26"	20	Nominal 3"- 5" incl.
0.50"	29	Nominal 7"- 9" incl.
0.63"	21	Nominal 10"- 14" incl.
0.46"	134	Overall TB & TS

Table 10. Summary of Relevant Two-point Variations for Rigid GPS Sections

Standard Deviation	Section Count	Nominal or Design Thickness (inches)
PCC Thicknesses (GPS Rigid):		
0.18"	227	Nominal 7" - 9" incl.
0.21"	98	Nominal 10" - 12" incl.
0.20"	360	Overall PCC
GB & GS Thicknesses (GPS Rigid):		
0.72"	74	Nominal 3"- 5" incl.
1.13"	69	Nominal 5"- 7" excl.
0.95"	220	Overall GB & GS
TB & TS Thicknesses (GPS Rigid):		
0.24"	102	Nominal 3"- 5" incl.
0.33"	48	Nominal 5"- 7" excl.
0.35"	194	Overall GB & GS

2.2 VARIABILITY OF PCC-RELATED DESIGN PARAMETERS

The following categories of PCC-related parameters have been studied to determine the characteristics related to the variability of rigid pavement design inputs:

1. Concrete Material Parameters (Laboratory)
    a. Compressive strength
    b. Flexural strength
    c. Split-tensile strength
    d. Modulus of elasticity
2. Joint and Crack Load Transfer Efficiency (LTE)
    a. Joint LTE for jointed concrete pavements (JCP)
    b. Crack LTE for continuously reinforced concrete pavements (CRCP)
3. Back-calculated Moduli
    a. Modulus of subgrade reaction
    b. Concrete modulus of elasticity (based on liquid foundation and elastic solid foundation approach)
    c. Subgrade modulus of elasticity (based on elastic solid foundation approach)

The discussion of the variability characteristics of each of the above design parameter is presented in the following subsections.

2.2.1 Variability of Concrete Materials Data

The following are the key concrete materials–related laboratory data for use as input data for concrete pavement design:

1. Compressive strength
2. Flexural strength
3. Split-tensile strength
4. Modulus of elasticity

<u>Data Source</u>

The concrete material data were extracted from the appropriate LTPP IMS tables in DataPave 2.0, released September 1999, as follows:

1. *PCC Compressive Strengths from Inventory Records* (Table INV_PCC_STRENGTH) – This table contains data from construction time testing and includes all of the four concrete materials data types. These data are available only for GPS test sections, as furnished by the various state and provincial DOTs. The data for each test section include number of test samples, mean value and standard deviation for each test type. Strength data are primarily for tests conducted at 28 days (nominally).

2. *PCC Compressive Strengths* (Table TST_PC01) – This table contains core compressive strength data from GPS test sections and construction time cylinder compressive strength data from SPS-2 and SPS-8 test sections.
3. *PCC Split Tensile Strengths* (Table TST_PC02) – This table contains core split tensile strength data from GPS test sections and construction time cylinder split tensile strength data from SPS-2 and SPS-8 test sections.
4. *Modulus of Elasticity from PCC Cores* (Table TST_PC04) – This table contains core modulus of elasticity data from GPS test sections and construction time cylinder modulus of elasticity data from SPS-2 and SPS-8 test sections.
5. *Flexural Strengths from PCC Beams* (Table TST_PC09) – This table contains construction time beam flexural strength data from SPS-2 and SPS-8 test sections.

For data from the above-listed test tables, only the SPS-2 data were analyzed. The GPS core data were not analyzed for variability because only two core tests were available for each section.

The SPS-2 data were further subdivided as follows:

1. Test PC01 – Cylinder compressive strength data at 14, 28 and 365 days (nominally).
2. Test PC02 – Cylinder split tensile strength data at 14, 28 and 365 days (nominally).
3. Test PC04 – Cylinder modulus of elasticity data at 28 and 365 days (nominally).
4. Test PC09 – Beam flexural strength data at 14, 28 and 365 days (nominally).

Thus, within each SPS-2 project (or "sites" within each State), the following information was available for each testing age:

- Mean compressive, split tensile, and flexural strengths.
- Standard deviations of the mean compressive split tensile and flexural strengths.

The SPS-2 projects incorporate two concrete mix designs – one designed to produce a nominal 550-psi flexural strength and the other designed to produce a nominal 900-psi flexural strength at 14 days, based on third point beam tests. The distribution of the test sections by concrete mix type is given in Table 11. [It should be noted that the exact mix design to achieve these two levels of strength were not always adhered to by the various State DOTs.]

The first task in manipulating the SPS-2 tables was to subdivide the sections within a given SPS-2 project into the two strength categories – 550- and 900-psi concrete. Within each strength category, the test data were further subdivided by age at time of test, typically 14-day, 28 days and 1 year (nominally). However, it should be noted that the actual test ages did not always correspond to these three specified test ages. Still, an attempt was made to group the test data within the appropriate specified test age group.

Table 11. Section Numbers for Two PCC Strengths Associated with SPS-2 Projects

550-psi Sections	900-psi Sections
201	202
203	204
205	206
207	208
209	210
211	212
213	214
215	216
217	218
219	220
221	222
223	224

Compressive Strength Data Analysis – GPS Sections

Compressive strength data were available from 58 GPS test sections. Most of these data (46 sections) are for tests conducted at 28 days on cylinders prepared as part of construction time QA/QC, as reported by the various state and provincial DOTs. The data presumably relate to the total project, incorporating each 500-foot test section and so incorporate variability for projects, possibly several miles in length. A summary of data is given in Table 12 and details for each test section are given in Table B1 in Appendix B. The number of compressive strength tests, *per section*, ranged from only 3 to a total of 179.

The data indicate reasonably good quality control for the concrete used to construct the GPS test sections – the average coefficient of variation is about 10% and most values were under 15%.

A comparison of variability for 7 days versus 28-day results is given in Table 13 and details are given in Table B2 of Appendix B.

It is seen that although the overall mean strength and standard deviation values are higher for the 28-day test data, the coefficient of variation values are similar for the two categories of data.

A comparison of variability was performed for lower strength (< 4,000-psi) and higher strength (> 5,000-psi) concrete. The comparison is summarized in Table 14 and details are given in Table B3 of Appendix B.

Table 12. Summary of GPS Concrete Compressive Strength Data Variability (all ages)

Parameter	Mean	Minimum	Maximum	Standard Deviation	No. of Sections
Mean Section Strength, psi	4,848	3,110	6,208	749	58
Section Std. Deviation, psi	470	4	921		58
Section Coeff. of Var., %	10	0	24		58
Sample Size within Section	35	3	179	38	58

Table 13. Comparison of GPS 7-day vs. 28-day Compressive Strength Data

Parameter	Mean	Minimum	Maximum	Standard Deviation	No. of Sections
7-Day:					
Mean Section Strength, psi	3,687	3,110	4,178	399	6
Section Std. Deviation, psi	344	80	517	193	6
Section Coeff. of Var., %	10	2	17	6	6
Sample Size within Section	12	4	26	9	6
28-Day:					
Mean Section Strength, psi	4,914	3,838	6,208	669	44
Section Std. Deviation, psi	504	4	909	187	44
Section Coeff. of Var., %	10	0	24	4	44
Sample Size within Section	40	3	179	40	44

Table 14. Comparison of GPS Low and High Compressive Strength Data Variability

Parameter	Mean	Minimum	Maximum	Standard Deviation	No. of Sections
< 4,000-psi Mean Strength for Sections (all ages):					
Mean Section Strength, psi	3,749	3,110	4,000	294	10
Section Std. Deviation, psi	506	80	909	213	10
Section Coeff. of Var., %	13	2	24	6	10
Sample Size within Section	23	4	78	24	10
> 5,000-psi Mean Strength for Section (all ages):					
Mean Section Strength, psi	5,554	5,063	6,208	369	25
Section Std. Deviation, psi	508	37	921	208	25
Section Coeff. of Var., %	9	1	16	4	25
Sample Size within Section	21	3	99	22	25

Overall, the higher strength concrete data exhibit less variability than the lower strength concrete data, possibly indicating better control exercised to produce higher strength concrete. The overall coefficient of variation is still considered fairly good for both categories of concrete.

Variability was evaluated within each state. States considered were those that had at least five GPS-3, GPS-4 or GPS-5 concrete sections. These data are summarized in Table B4 in Appendix B. Surprisingly, the data show reasonable consistency with respect to variability of the compressive strength data. The average coefficient of variation for the five states available in the analysis ranged from 8% to 12% for average compressive strengths within the states ranging from about 4,700 to 5,600 psi.

The above-discussed compressive strength data indicate that on well-controlled construction projects, it would not be considered unreasonable to produce concrete that has a coefficient of variation of 10% or less for compressive strength.

Compressive Strength Data Analysis – SPS-2 Projects

Compressive strength data from SPS-2 projects were available for 7 days, 28 days and 1-year (nominal) aged cylinders, for the 550- and 900-psi 14-day design flexural strength concrete, respectively for 6 in. diameter cylinders prepared in the field and cured in the laboratory and for 4 in. diameter cores. A summary of the compressive strength data variability is given in Table 15 and details are given in Tables B5 (a) and B6(a) of Appendix B. Only data with at least three test values for any test condition were used. The summary shown is based on 3 to 10 test values for a given test condition, per section. The coefficient variation appears to be independent of specimen type (cylinders versus cores) test age and changes in concrete strength with age – an average coefficient of variation of about 10 to 12% for all three test ages irrespective of their compressive strength values.

The compressive strength data variability was compared for the 550- and 900-psi concrete. The variability data are summarized in Table 16 and detailed in Tables B5(b) and B6(b) in Appendix B. The data indicates that the 550-psi concrete is more variable than the 900-psi concrete. The variability for the 550-psi concrete is not considered good, while the average variability for the 900-psi concrete can be considered very good. However, within each category of data, there are extreme cases of variability. The SPS-2 variability data are consistent with the GPS variability data.

It should be noted that the cylinder and core strengths were also very consistent for the three test ages. This was a very interesting finding as it is typically assumed that core test results would be lower than laboratory cured specimens for the same test age.

Table 15. Compressive Strength Data Variability for Combined 550- and 900-psi PCC

Parameter	Mean	Minimum	Maximum	Standard Deviation	No. of Sections
Cylinder Data					
14-day Std. Deviation, psi	516	134	1,141	284	17
14-day Coef. of Variation, %	11.3	2.2	21.1	5.9	17
28-day Std. Deviation, psi	532	131	1,513	348	18
28-day Coef. of Variation, %	10.1	2.0	27.1	6.6	18
1-year Std. Deviation, psi	759	169	1,912	422	22
1-year Coef. of Variation, %	11.5	2.6	30.1	6.8	22
Core Data					
14-day Std. Deviation, psi	531	74	1,274	276	22
14-day Coef. of Variation, %	12.3	1.2	25.3	6.3	22
28-day Std. Deviation, psi	515	147	1,048	255	21
28-day Coef. of Variation, %	10.5	3.6	22.9	4.7	21
1-year Std. Deviation, psi	747	211	1,827	386	21
1-year Coef. of Variation, %	11.5	3.4	24.6	5.7	21

Table 16. Comparison of Variability of Compressive Strength for 550- and 900-psi PCC

a) Cylinder Data

Parameter	Mean	Minimum	Maximum	Standard Deviation	No. of Sections
550-psi Concrete:					
14-day Strength	3,608	2,568	5,030	777	8
14-day Std. Deviation, psi	414	175	710	181	8
14-day Coef. of Variation, %	12.0	4.7	21.1	5.8	8
28-day Strength	4,258	3,034	5,786	801	9
28-day Std. Deviation, psi	464	154	719	229	9
28-day Coef. of Variation, %	11.1	3.6	17.3	5.4	9
1-year Strength	5,677	4,412	7,226	922	11
1-year Std. Deviation, psi	788	288	1,912	466	11
1-year Coef. of Variation, %	13.8	4.6	30.1	7.3	11
900-psi Concrete:					
14-day Strength	5,970	4,740	7,020	624	9
14-day Std. Deviation, psi	607	134	1,141	336	9
14-day Coef. of Variation, %	10.6	2.2	19.9	6.3	9
28-day Strength	6,802	5,580	7,611	605	9
28-day Std. Deviation, psi	600	131	1,513	442	9
28-day Coef. of Variation, %	9.1	2.0	27.1	7.7	9
1-year Strength	8,217	5,167	10,859	1,600	11
1-year Std. Deviation, psi	730	169	1,495	394	11
1-year Coef. of Variation, %	9.3	2.6	20.6	5.7	11

b) Core Data

Parameter	Mean	Minimum	Maximum	Standard Deviation	No. of Sections
550-psi Concrete:					
14-day Strength	3,619	2,759	5,227	843	11
14-day Std. Deviation, psi	536	307	986	214	11
14-day Coef. of Variation, %	14.9	8.3	23.5	4.6	11
28-day Strength	3,998	3,160	5,569	754	11
28-day Std. Deviation, psi	504	215	1,017	237	11
28-day Coef. of Variation, %	12.3	6.4	22.9	4.5	11
1-year Strength	5,601	4,475	7,897	990	11
1-year Std. Deviation, psi	703	211	1,043	268	11
1-year Coef. of Variation, %	12.4	4.3	18.1	4.0	11
900-psi Concrete:					
14-day Strength	5,549	3,328	7,028	1,022	11
14-day Std. Deviation, psi	526	74	1,274	338	11
14-day Coef. of Variation, %	9.8	1.2	25.3	6.9	11
28-day Strength	6,080	4,102	7,314	985	10
28-day Std. Deviation, psi	528	147	1,048	286	10
28-day Coef. of Variation, %	8.5	3.6	14.3	4.2	10
1-year Strength	7,995	6,148	10,773	1,302	10
1-year Std. Deviation, psi	794	310	1,827	497	10
1-year Coef. of Variation, %	10.5	3.4	24.6	7.3	10

Flexural Strength Data Analysis – GPS Sections

Flexural strength data were available for 51 GPS test sections. These data were well divided between 7-, 14- and 28-day tests. Although the specimen sizes were not reported, it is assumed that these test were conducted on 6 in. by 6 in. by 18 in. beams fabricated at the site and later cured in the laboratory. Most of the data are for third point loading, but the data also include some center point loading. As indicated previously, the GPS data presumably relate to the total projects incorporating each 500-foot test section and so incorporate variability for projects, possibly several miles in length. A summary of the flexural strength data variability is given in Table 17 and details for each section are provided in Table B7.

As with the GPS compressive strength data, the flexural strength data indicate reasonably good quality control for the concrete used to construct the GPS test sections – the average coefficient of variation is about 10% and most of these values are under 15%.

A comparison of variability for 7-, 14- and 28-day data is given in Table 18 and details are provided in Table B8 of Appendix B. It should be noted that test age in Table 18 is not necessarily an indicator of comparable strength values. It is very likely that a design flexural strength value of about 650- to 700-psi may have been specified irrespective of the test age.

The coefficient of variation is very good for all three categories and is much better for the 7- and 14-day tests. This may indicate that the contractors possibly exercised better control over their concrete mixtures to ensure that the specified strengths were achieved at the lower test ages (i.e., 7- and 14-days).

A comparison of variability was performed for lower strength (< 650-psi) and higher strength (> 700-psi) concrete. The comparison is summarized in Table 19 and details are given in Table B9 of Appendix B. Overall, the lower strength concrete data exhibit slightly less variability than the higher strength concrete data, opposite to the trend for the GPS compressive strength data.

The variability in flexural strength data was evaluated within each state. States considered were those that had at least five GPS-3, GPS-4 or GPS-5 (concrete pavement) test sections. These data are summarized for the four states with adequate data in Table B10 of Appendix B. The variability in flexural data does not show consistency from state to state, opposite to what was noted for the compressive strength data. The average coefficient of variation within a state ranged from a low of 5% to a high of 14%, for average flexural strengths ranging from around 590- to 730-psi.

The above-discussed GPS flexural strength data also indicate that on well-controlled construction projects, a fairly good coefficient of variation, say around 15% or less, can be achieved for the flexural strength.

Table 17. Summary of GPS PCC Flexural Strength Data Variability for all Ages

Parameter	Mean	Minimum	Maximum	Standard Deviation	No. of Sections
Mean Section Strength, psi	683	488	910	94	51
Section Std. Deviation, psi	66	6	178	35	51
Section Coef. of Variation, %	10	1	34	5	51
Sample Size within Section	42	3	326	63	51

Table 18. Comparison of GPS 7-, 14- and 28-day Flexural Strength Data Variability

Parameter	Mean	Minimum	Maximum	Standard Deviation	No. of Sections
7-Day:					
Mean Section Strength, psi	697	613	795	60	12
Section Std. Deviation, psi	66	22	114	30	12
Section Coeff. of Var., %	9	3	14	4	12
Sample Size within Section	102	8	326	101	12
14-Day:					
Mean Section Strength, psi	695	488	910	105	19
Section Std. Deviation, psi	56	6	155	34	19
Section Coeff. of Var., %	8	1	19	4	19
Sample Size within Section	30	4	118	34	19
28-Day:					
Mean Section Strength, psi	671	524	853	106	15
Section Std. Deviation, psi	81	29	178	43	15
Section Coeff. of Var., %	12	5	34	7	15
Sample Size within Section	17	3	101	25	15

Table 19. Comparison of GPS Low and High Flexural Strength Data Variability

Parameter	Mean	Minimum	Maximum	Standard Deviation	No. of Sections
< 650 psi Mean Strength for Sections (all ages):					
Mean Section Strength, psi	604	488	649	40	24
Section Std. Deviation, psi	55	6	178	36	24
Section Coeff. of Var., %	9	1	34	7	24
Sample Size within Section	24	3	118	32	24
> 700 psi Mean Strength for Section (all ages):					
Mean Section Strength, psi	775	706	910	61	21
Section Std. Deviation, psi	83	22	155	33	21
Section Coeff. of Var., %	11	3	19	4	21
Sample Size within Section	68	4	326	86	21

Flexural Strength Data Analysis – SPS-2 Projects

Flexural strength data from SPS-2 projects were available for 7 days, 28 days and 1-year (nominal) ages, for the 550- and 900-psi concrete. These tests were typically conducted on 6 in. by 6 in. by 18 in. beams fabricated at the site and later cured in the laboratory, as per LTPP Program requirements. A summary of the flexural strength data variability is given in Table 21 and details are given in Table B11 in Appendix B. Only data with at least three test values for any test condition were used. The summary is based on 3 to 10 test values for a given test condition per project. All SPS-2 flexural strength testing was performed using the third point loading procedure.

As with the SPS-2 compressive strength data, the CV for the flexural strength appears to be independent of test age but is much better than for compressive strength – an average low value of 8 to 9%.

The flexural strength data variability was compared for the 550 and 900-psi concrete. The variability data are summarized in Table 22 and detailed in Table B12 in Appendix B.

The data indicate reasonable consistency in the flexural strength data between the 550- and 900-psi concrete and at different test ages. The average variability ranged from about 7 to about 11%. An interesting observation was that the average 14-day flexural strength for the 550-psi concrete was 570 psi, very close to the 550-psi target experimental value. However, the average 14-day flexural strength for the 900-psi concrete was 800 psi, much lower than the 900-psi target experimental value. It should also be noted, as shown in Table B12, for several SPS-2 projects, the average flexural strength at 1 year for the 550- and 900-psi concrete were very similar. The 900-psi concrete at these projects did not exhibit a significant gain in flexural strength between 14 days and 1 year.

Table 20. SPS-2 Flexural Strength Data Variability for Combined 550- and 900-psi PCC

Parameter	Mean	Minimum	Maximum	Standard Deviation	No. of Sections
14-day Std. Deviation, psi	59	10	153	40	19
14-day CV, %	8.8	1.8	24.9	5.7	19
28-day Std. Deviation, psi	71	25	266	55	19
28-day CV, %	9.3	3.4	30.1	6.2	19
1-year Std. Deviation, psi	70	13	246	51	20
1-year CV, %	8.6	1.3	29.4	6.2	20

Table 21. Comparison of Variability of Flexural Strength for the 550- and 900-psi PCC

Parameter	Mean	Minimum	Maximum	Standard Deviation	No. of Sections
550-psi Concrete:					
14-day Strength	570	467	684	74	10
14-day Std. Deviation, psi	43	10	101	24	10
14-day CV, %	7.6	1.8	15.3	3.7	10
28-day Strength	634	478	804	100	10
28-day Std. Deviation, psi	59	28	146	37	10
28-day CV, %	9.1	4.2	19.0	4.6	10
1-year Strength	742	627	904	112	10
1-year Std. Deviation, psi	63	30	153	35	10
1-year CV, %	8.5	4.3	19.2	4.5	10
900-psi Concrete:					
14-day Strength	801	614	906	88	9
14-day Std. Deviation, psi	77	35	153	45	9
14-day CV, %	10.2	4.2	24.9	7.3	9
28-day Strength	884	747	1,007	75	9
28-day Std. Deviation, psi	85	25	266	71	9
28-day CV, %	9.5	3.4	30.1	7.9	9
1-year Strength	913	808	1,036	69	10
1-year Std. Deviation, psi	76	13	246	64	10
1-year CV, %	8.6	1.3	29.4	7.8	10

Split Tensile Strength Data Analysis – GPS Sections

Only a limited amount of data for split tensile strengths were available for the GPS test sections. These data are summarized in Table B13. No conclusions related to the variability of split tensile strength data can be drawn from this table. For one state for which variability data was available, the coefficient of variation was reported to be 11.5% for a mean split tensile strength of about 460 psi.

Split Tensile Strength Data Analysis – SPS Projects

Split tensile strength data from SPS-2 projects were available for 7 days, 28 days and 1-year (nominal) ages, for the 550- and 900-psi concrete for 6 in. diameter cylinders prepared in the field and cured in the laboratory and for 4 in. diameter cores. A summary of the split tensile strength data variability is given in Table 22 and details are given in Tables B14a and B15a in Appendix B. Only data with at least three test values for any test condition were used. The summary is based on 3 to 10 test values for a given test condition per project.

Similar to the flexural and compressive strength data, the coefficient of variation for the split tensile strength appears to be independent of test age and is similar to that for the compressive strength data – an average value of 9 to 12%, with core test data slightly more variable.

The split tensile strength data variability was compared for the 550 and 900-psi concrete. The variability data are summarized in Table 23 and detailed in Tables B14(b) and B15(b) in Appendix B.

The split tensile strength data for the 900-psi concrete appear to be a more consistent than for the 500-psi concrete data, indicating possibly better quality control for the 900-psi concrete. It should be noted that the core test results tended to be slightly higher than the cylinder test results for the different test ages.

Modulus of Elasticity Data Analysis – GPS Sections

Only limited amounts of laboratory data for modulus of elasticity were available for the GPS sections. These data are summarized in Table B16 of Appendix B. No conclusions related to the variability of the modulus of elasticity data can be drawn from this table.

Modulus of Elasticity Data Analysis – SPS Projects

Modulus of elasticity data for SPS-2 projects was available for 28 days and 1-year (nominally) ages for the 550 and 900-psi concrete for 4-in. diameter cores. A summary of the modulus of elasticity data is given in Table 24 and details are given in Table B17 in Appendix B. Only data with at least three values for any test condition were used. The summary is based on 3 to 10 test values for a given test condition per project.

Similar to the strength parameters, the coefficient of variation for the modulus of elasticity appears to be independent of test age. The modulus of elasticity data exhibits fairly good consistency – an average CV value of about 12%.

The modulus of elasticity data variability was compared for the 550 and 900-psi concrete. The variability data are summarized in Table 25 details are provided in Table B18 in Appendix B.

The modulus of elasticity data for the 900-psi concrete appear to be more consistent than for the 550-psi concrete data, once again indicating possibly better quality control for the 900-psi concrete.

Table 22. SPS-2 Split Tensile Strength Data Variability (for both the 550- and 900-psi PCC)

Parameter	Mean	Minimum	Maximum	Standard Deviation	No. of Records
Cylinder Data					
14-day Std. Deviation, psi	42	18	76	17	18
14-day CV, %	9.6	3.5	16.8	4	18
28-day Std. Deviation, psi	50	10	138	33	15
28-day CV, %	10.1	2.5	26.4	7.2	15
1-year Std. Deviation, psi	59	19	102	24	16
1-year CV, %	10.4	2.3	18.9	4.3	16
Core Data					
14-day Std. Deviation, psi	50	13	109	30	21
14-day CV, %	10.6	3.0	21.8	6.4	21
28-day Std. Deviation, psi	59	18	151	34	22
28-day CV, %	11.7	3.5	26.1	5.9	22
1-year Std. Deviation, psi	82	25	173	36	18
1-year CV, %	12.6	4.5	24.8	5.4	18

Table 23 Comparison of Split Tensile Strength Variability of the 550- and 900-psi PCC

a) Cylinder Data

Parameter	Mean	Minimum	Maximum	Standard Deviation	No. of Records
550-psi Concrete:					
14-day Strength	384	332	481	44	9
14-day Std. Deviation, psi	44	25	68	13	9
14-day CV, %	11.4	6.7	16.8	3.4	9
28-day Strength	446	367	525	60	7
28-day Std. Deviation, psi	48	10	85	33	7
28-day CV, %	10.9	2.5	23.1	8.0	7
1-year Strength	515	413	619	60	9
1-year Std. Deviation, psi	53	22	88	18	9
1-year CV, %	10.3	4.7	16.2	3.5	9
900-psi Concrete:					
14-day Strength	517	474	580	42	9
14-day Std. Deviation, psi	40	18	76	20	9
14-day CV, %	7.8	3.5	15.9	4.0	9
28-day Strength	565	522	642	39	8
28-day Std. Deviation, psi	52	28	138	35	8
28-day CV, %	9.4	5.0	26.4	7.0	8
1-year Strength	682	539	857	107	7
1-year Std. Deviation, psi	67	19	102	29	7
1-year CV, %	10.4	2.3	18.9	5.4	7

b) Core Data

Parameter	Mean	Minimum	Maximum	Standard Deviation	No. of Records
550-psi Concrete:					
14-day Strength	434	320	599	80	11
14-day Std. Deviation, psi	53	13	108	31	11
14-day CV, %	12.4	3.3	21.8	6.6	11
28-day Strength	444	343	507	49	12
28-day Std. Deviation, psi	55	20	81	17	12
28-day CV, %	12.2	5.7	18.3	3.4	12
1-year Strength	593	430	696	93	8
1-year Std. Deviation, psi	85	31	173	46	8
1-year CV, %	13.9	6.4	24.8	6.0	8
900-psi Concrete:					
14-day Strength	566	455	755	89	10
14-day Std. Deviation, psi	47	16	109	29	10
14-day CV, %	8.7	3.0	20.2	5.8	10
28-day Strength	585	525	789	86	10
28-day Std. Deviation, psi	65	18	151	49	10
28-day CV, %	11.0	3.5	26.1	8.1	10
1-year Strength	698	555	901	119	10
1-year Std. Deviation, psi	79	25	127	29	10
1-year CV, %	11.5	4.5	20.9	4.9	10

Table 24. Variability of SPS-2 Modulus of Elasticity (for both the 550- and 900-psi PCC)

Parameter	Mean	Minimum	Maximum	Standard Deviation	No. of Records
28-day Std. Deviation, ksi	0.54	0.12	1.17	0.34	19
28-day CV, %	12.7	2.5	34.6	8.5	19
1-year Std. Deviation, ksi	0.59	0.13	1.37	0.37	17
1-year CV, %	11.9	2.6	27.3	6.5	17

Table 25. Comparison of Modulus of Elasticity Variability for the 550- and 900-psi PCC

Parameter	Mean	Minimum	Maximum	Standard Deviation	No. of Records
550-psi Concrete:					
28-day E, ksi	4.12	2.58	5.84	1.00	10
28-day Std. Deviation, ksi	0.49	0.18	1.07	0.30	10
28-day CV, %	13.2	4.4	34.6	10.2	10
1-year E, ksi	4.54	2.98	5.43	0.82	8
1-year Std. Deviation, ksi	0.63	0.13	1.37	0.44	8
1-year CV, %	13.2	2.6	27.3	8.0	8
900-psi Concrete:					
28-day E, ksi	4.84	3.36	8.04	1.35	9
28-day Std. Deviation, ksi	0.60	0.12	1.17	0.39	9
28-day CV, %	12.2	2.5	21.6	6.8	9
1-year E, ksi	4.98	3.52	6.12	0.81	9
1-year Std. Deviation, ksi	0.55	0.22	1.20	0.31	9
1-year CV, %	10.8	4.2	19.6	5.0	9

2.2.2 Variability of Load Transfer Efficiency Data

In this study, the load transfer efficiency (LTE) across joints (or cracks) was calculated using FWD deflection data obtained from the Long-Term Pavement Performance (LTPP) database. The FWD deflection data extracted from four different LTPP experiments were analyzed. The four different experiments were GPS-3 and SPS-2 for jointed plain concrete pavements (JPCP), GPS-4 for jointed reinforced concrete pavements (JRCP), and GPS-5 for continuously reinforced concrete pavements (CRCP).

<u>Data Source</u>

The FWD deflection data were stored in the IMS database table of FWD deflections for each LTPP experiment, and were extracted using the DataPave 2.0 (release September 1999). LTEs were determined for both jointed and CRC pavements. For the LTE testing, the FWD load was applied at one side of the joint or crack and the deflections were measured at both sides of the joint or crack. The LTE is defined as the deflection measured at the unloaded slab divided by the deflection measured at the loaded slab side, expressed as a percentage as shown in Figure 1.

In the LTPP FWD deflection database for rigid pavements, the FWD testing was conducted at three load levels (designated as Levels 2, 3 and 4) with four drops at each level, resulting in a total of twelve drops. For this study, Drop No. 2 of Load Level 2 (approximately 40 kN or 9,000 lbs. peak load) was used for the LTE variability analyses. The extracted data were examined to ensure their consistency and reasonableness. As a consequence, some data were excluded from the LTE variability analyses. Primary reasons for data rejection were:

- Data points with calculated LTE greater than 105%
- Data only available for part of the sections (primarily occurred at SMP sections)
- Wrong section designations, such as J4 for CRC pavements (should be C4), reverse of J4 and J5, etc.
- Unknown joint type for jointed pavement (doweled or non-doweled)
- Incorrect testing time records
- Duplicate data

Raw deflection data excluded from further analyses included 3,423 data points for JRCP, 7,239 data points for JPCP, and 268 data points for CRCP.

Table 26 shows the total number of test sections, total number of FWD deflection data sets (resulted from tests performed at different times for each section), and total number of raw data points for each type of pavement.

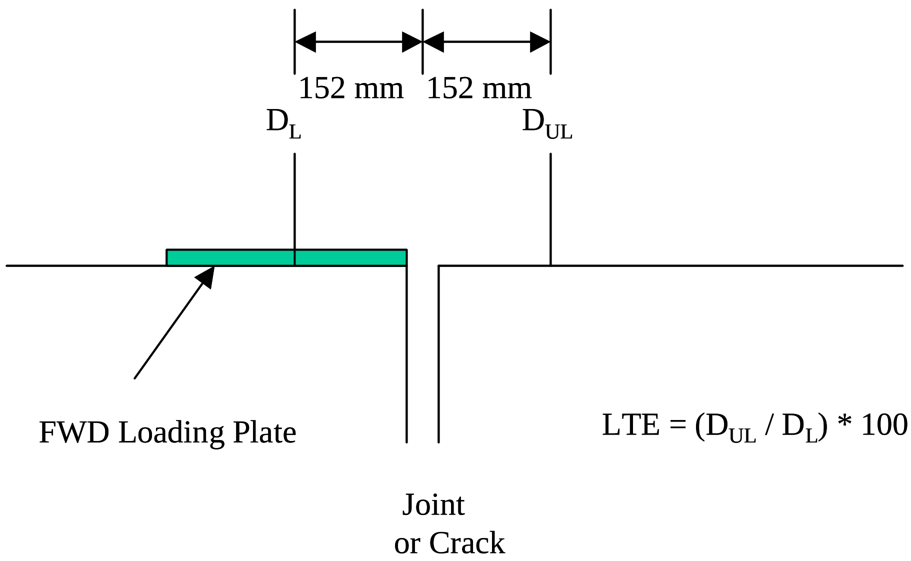


Figure 1. Definition of Load Transfer Efficiency (LTE)

Table 26. Data Availability for Analysis of LTE Variability

Pavement Type	No. of Sections	No. of FWD Data Sets	No. of Raw Data Points
JPCP with Doweled Joint	198	484	12,027
JPCP with Non-Doweled Joint	73	182	6,910
JRCP (all doweled)	62	169	5,169
CRCP	83	187	6,920

Determination of LTE and Analysis Methodology

The FWD deflection data were separated into three groups: jointed pavements with doweled joints (JPC-DJ), jointed pavements with aggregate-interlock (plain) joints (JPC-PJ), and CRCP. The GPS-3 experimental sections contain JPCP with both doweled and aggregate-interlock joints, while the SPS-2 experimental sections all have doweled joints. The JRCP (GPS-4) test sections were all doweled pavements.

For each test section, various numbers of joints or cracks were subjected to FWD tests at each time of testing. The actual number of joints or cracks tested was dependent on joint or crack spacing, with a maximum of 20 tests for each section. Further, for all types of pavement sections, two types of load transfer deflections were measured. The first type was conducted with the FWD load applied at the approach slab (designated as J4 or C4 for JPC or CRC pavements, respectively), while the second with the load placed at the leave slab (designated as J5 or C5 for JPC or CRC pavements, respectively). The average LTE for each section at each time of testing, along with its standard deviation and coefficient of variation were computed for the combined data, including both approach and leave slab tests. These values were also computed for J4 (or C4) and J5 (or C5) separately.

In this study, the coefficient of variation (CV) was used to represent the variability associated with the computed average LTE for the FWD tests conducted on each test section, at each time of testing. The CV was calculated as the standard deviation divided by the average LTE, expressed as a percentage. The general analysis processes for all three types of pavements are presented below:

- Assessment of the average LTEs and the associated CVs for loads applied on the approach slabs (J4 or C4) and the leave slabs (J5 or C5). A student t-test and an F-test were first conducted to assess if the average LTEs determined from the J4 (C4) and the J5 (C5) deflection data were from the same population. The average LTEs determined from the J4 (C4) and the J5 (C5) deflection data were considered from the same population if the null hypotheses of equal mean and equal variance could not be rejected. The distributions of the differences between the average LTEs determined from J4 and J5 (or C4 and C5) deflection data and the CVs determined from J4 and J5 (or C4 and C5) deflection data were then evaluated.

- Assessment of the CVs associated with their corresponding average LTEs for combined deflection data derived under both the J4 and J5 (or C4 and C5) test conditions. This included the use of regression techniques to evaluate the relationship between the CVs and their corresponding average LTEs, and the use of histograms to evaluate distribution of the CVs.

- Assessment of the CVs with respect to different pavement parameters, such as joint (or crack) spacing, slab stiffness, base type, shoulder type, the presence or absence of subsurface drainage, climatic variables, and age at FWD testing. Student t-tests were conducted to assess if CVs were different for pavements with different parameters. Plots were also prepared to explore if some general trends could be detected.

As stated earlier, the average LTEs and CVs were separated into three groups according to the different pavement types, i.e. jointed concrete pavements with plain joints (JPC-PJ – GPS-3 sections), jointed pavements with doweled joints (JPC-DJ – GPS-3, GPS-4 and SPS-2 sections), and CRCP (GPS-5 sections). The following sections of the report present the analyses of the variability associated with calculated average LTEs for these three types of pavement.

Analyses of Variability of LTE for JPC-PJ

Assessment of Variability of LTE Derived under J4 and J5 Loadings

In the LTPP FWD load-testing program, the FWD loads were applied at both sides of the joints, on the approach slab and on the leave slab. The load position was designated as J4 for the approach slab loadings and J5 for the leave slab loadings. The average LTEs and their associated CVs were calculated for deflection data obtained under the J4 and J5 loading conditions and for the combined deflection data. An attempt was made in this study to evaluate if the average LTEs and their associated CVs derived from the FWD deflection data obtained under the J4 and J5 loads were from the same population.

For the FWD tests conducted on each test section at each time of testing, a student t-test was conducted to compare the average LTEs computed from the J4 and the J5 deflection data, and an F-test was conducted to compare the associated variance of the J4 and J5 LTEs. The data were considered to come from the same population if, at the 95% confidence level, the null hypotheses of equal mean and equal variance could not be rejected. The analyses showed that, out of the 182 data sets, 99 (or 54%) exhibited significant differences between the LTEs determined from the J4 and J5 loading conditions.

To further analyze the differences between the average LTEs and CVs derived from data under the J4 and J5 loads, the difference between the two data were calculated for each test section tested, and at each time of test (i.e., J4-J5). Both the average LTEs and the CVs showed wide ranges of variation. The difference in average LTE ranged from -44% to +26% with an average of -6%, while the difference in CV varied between -50% and +46%, with an average value of +1%. The distributions of the differences are presented in Figures B1 and B2 in Appendix B for the average LTEs and the CVs, respectively. It can be observed from these figures that, for both the average LTE and the CV, although the ranges of the differences were wide, the majority of differences were small (within ±10%).

A comparison of the average LTEs and the CVs for the J4, J5 and combined loading conditions is shown in Table 27 for the JPC-PJ. No appreciable differences were observed on the average LTEs and CVs computed from deflections under the different FWD load positions. The analyses showed that although some differences existed between the average LTEs and the CVs determined from the deflections obtained under J4 and J5 loadings, the distribution of the differences concentrated in a narrow range (within ±10%) around zero. Also, in routine FWD deflection data analyses for pavement-related issues, the data collected from both loading positions are typically used together. The combined FWD deflection data were therefore used for further variability analyses related to LTE for pavements with plain joints.

General Assessment of the Variability of LTE

To evaluate the general trends of the variability of the LTE for the JPC-PJ, the CV was plotted as a function of the average LTE, as shown in Figure 2. As observed in this figure, the CV was inversely related to the average LTE for the sections. As the average LTE decreased, the CV increased. Also, the variations of the CVs among different sections tested at different times at lower LTE levels were much greater than those having higher LTEs. This makes intuitive sense, since JPC pavements with good average LTE (say, close to 100%) will also have low CVs, since most joints will be close to 100% LTE. Later, as the LTE falls (on average), some joints continue to perform well, while others start to loose their aggregate (or other) interlock with a corresponding drop in LTE.

The distribution of the average LTEs and the CVs are presented in Figures B3 and B4 in Appendix B, respectively. Both the average LTE and the CV showed a significant range in variability. A summary of the CVs associated with the average LTEs for the different sections tested at different times is presented in Table 28. As shown in this table, the CV ranged from 2% to 91%, with an average value of 23% (± 3% at 95% confidence level). Also, 75 percent of the CVs were found to be less than 38%.

Effect of Joint Spacing on the Variability of LTE

To evaluate the effects of joint spacing on the variability of LTE, the calculated CVs were plotted against the average joint spacing of the different test sections, as shown in Appendix B, Figure B5. As indicated in this figure, no apparent relationship between the CV and the average joint spacing could be observed.

Effects of Pavement Base Type on the Variability of LTE

Many different types of materials were used as the base layer in the pavement structures in the LTPP program. To analyze the effects of base type on the variability of LTE determined from the FWD deflection data, these different base types were combined into two groups: bound bases and unbound bases. The materials used for bound base included cement-aggregate mixture, cement treated subgrade soil, dense graded asphalt cement, lean concrete, lime treated subgrade soil, open graded asphalt cement, soil cement, and sand asphalt. The materials used for unbound base included gravel or crushed stone, limerock, and soil-aggregate mixture.

The CV data were separated according to the two base types and summary statistics were computed for each set of data. A student t-test and an F-test were also performed to compare if the two sets of CVs had an equal mean and variance. The results of these analyses are presented in Table 29. As seen from this table, the average CVs were 23% and 22% for pavement sections with bound and unbound base, respectively. At the 95% confidence level, the two sets of CVs were determined to have equal mean and equal variance, inferring that the base type did not have any statistical effect on the variability of the calculated LTE using the FWD data in the LTPP program.

Table 27. Effect of FWD Load Position on Average LTE and CV for JPC-PJ Pavements

	Load Applied at						Combined Data		
	Approach Slab (J4)			Leave Slab (J5)					
	Mean	Max.	Min.	Mean	Max.	Min.	Mean	Max.	Min.
Average LTE, %	65	99	11	71	98	12	68	97	11
Section Std. Dev., %	9	38	1	10	30	1	11	34	1
CV, %	20	104	1	20	88	1	23	91	2

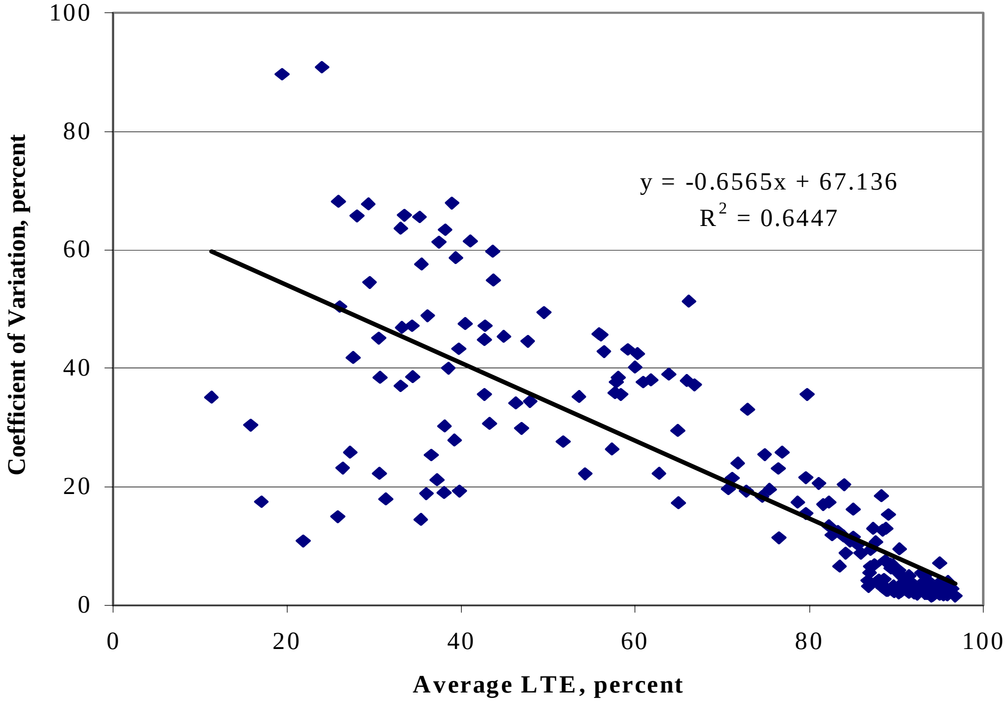


Figure 2. CV vs. Average LTE for Concrete Pavements with Plain Joints

Table 28. Summary Statistics of SD and CV Associated with Average LTE for JPC-PJ

Parameter	Mean	Maximum	Minimum	75% Point	95% CI
Average Section LTE, %	67.9	96.8	11.3	>42.8	± 3.7
Section Std. Deviation	11.2	34.0	1.5	<17.4	± 1.2
Section CV, %	22.5	90.8	1.6	<37.6	± 3.0
No. of Data Sets	182				

Effects of Outside Shoulder Type on the Variability of LTE

Two primary types of material were used as shoulder for the LTPP PCC test sections – Portland cement concrete (referred to as concrete shoulder) and asphalt (referred to as asphalt shoulder). The effect of the shoulder material used on the variability of LTE was evaluated in this study. The CV data were separated into two groups according to the two shoulder types. Summary statistics were computed and a student t-test and an F-test were performed. The results of these analyses are shown in Table 30. From this table, it appears that pavement sections with concrete shoulders had slightly smaller average CV of 21%, compared to the average CV of 24% for pavements with asphalt shoulders. However, no statistical differences were observed for both the averages and the variances of the CV data associated with pavement sections with concrete and asphalt shoulders.

A plot was also prepared to graphically assess the effects of the shoulder types on the variability of LTE. The CVs were plotted against their associated average LTE for pavements with concrete and asphalt shoulders (see Figure 3). It is clear from this figure that shoulder type had little effect on the variability of the calculated LTE.

Effects of Sub-Surface Drainage on the Variability of LTE

For the pavement structures in the LTPP program, several different types of sub-surface drainage were incorporated. In this study, the different types of sub-surface drainage were grouped into two categories, pavements with a sub-surface drainage system and pavements without a sub-surface drainage system. The CV data were grouped according to this categorization. Summary statistics were calculated for CVs for each group as shown in Table 31. Also shown in Table 31 are the results of the t-test and the F-test. The average CV was 30% for the pavements using a drainage system and was 20% for the pavements without a drainage system. The t-test also showed that, at a 95% confidence level, the CV for pavements with a drainage system was higher than that for pavements without a drainage system.

Effects of Subgrade Soil on the Variability of LTE

Using the American Association of State Highway and Transportation Official's (AASHTO) soil classification system, the PCC test section subgrade soil was classified as A-1 to A-7. For this study, the subgrade soil was grouped into granular soil (including A-1, A-2 and A-3) and fine-grained soil (including A-4, A-5, A-6 and A-7). The CV data were separated for pavements with granular soil subgrade and those with fine-grained soil subgrade. Similarly, summary statistics were calculated and a t-test and an F-test were performed to compare the CVs for the two groups of pavements (see Table 32).

The average CV was 27% for the pavements with granular soil subgrade and was 19% for the pavements with fine-grained soil subgrade. The student t-test also confirmed that, at 95% confidence level, the CVs were different for pavements with the granular and the silty-clayey soil subgrade. At first sight, this result is also somewhat counter-intuitive; however, once again the differences observed were not large enough to draw any broad conclusions.

Effects of Pavement Age on the Variability of LTE

An effort was made in this study to assess the effects of pavement age at the time of FWD testing on the variability of LTE. The CVs were first plotted as a function of pavement age at the time of FWD testing, as shown in Appendix B, Figure B10. No apparent trend can be observed in this figure. Since the CV is a function of the average LTE, as previously indicated, another effort was made to analyze the effect of pavement age with respect to the average LTE for individual pavement sections.

For individual pavement sections, the average LTE and its associated CV were plotted as a function of pavement age, as presented in Figure 4 for three typical (but quite old) LTPP pavement sections: 04-7613, 06-3017 and 06-7493. From this figure, it is apparent that the CV is closely related to the average LTE. For Sections 04-7613 and 06-3017, the average LTE remained high and constant in the early ages and the CV was low kept constant as well. Once the average LTE started deteriorating, the CV started increasing. The average LTE for Section 06-7493 remained very high and constant over the evaluation period and so did the CV.

Intuitively, this result is in accord with normal expectations. More results of this nature could not be shown, since there are very few sections in LTPP with an age of 15 or more years where significant pavement response deterioration, whether in terms of LTE or modulus of elasticity, has ensued.

Table 29. Comparison of Variability for JPC-PJ with Bound and Unbound Bases

Parameter	Bound Base				Unbound Base			
	Mean	Maximum	Minimum	95% CI	Mean	Maximum	Minimum	95% CI
Average Section LTE, %	68.5	95.8	19.4	± 4.7	66.5	96.8	11.3	± 6.1
Section Std. Deviation	11.8	34	1.5	± 1.6	10.7	26.5	1.5	± 1.8
Section CV, %	23.4	90.8	1.6	± 4.1	22.0	70.0	1.6	± 4.5
No. of Data Sets	103				76			
t-test at 95% level for CV	Equal mean for Section CV							
F-test at 95% level for CV	Equal Variance for Section CV							

Table 30. Comparison of Variability for JCP-PJ with Concrete and Asphalt Shoulders

Parameter	Concrete Shoulder				Asphalt Shoulder			
	Mean	Maximum	Minimum	95% CI	Mean	Maximum	Minimum	95% CI
Average Section LTE, %	69.5	95.9	17.0	± 5.3	66.7	96.8	11.3	± 5.1
Section Std. Deviation	10.8	26.1	1.5	± 1.6	11.6	34.0	1.5	± 1.7
Section CV, %	20.9	89.6	1.6	± 4.2	23.9	90.8	1.6	± 4.3
No. of Data Sets	83				99			
t-test at 95% level for CV	Equal mean for Section CV							
F-test at 95% level for CV	Equal Variance for Section CV							

Table 31. Comparison of Variability for JCP-PJ With and Without Drainage

Parameter	With Drainage				Without Drainage			
	Mean	Maximum	Minimum	95% CI	Mean	Maximum	Minimum	95% CI
Average Section LTE, %	59.5	96.4	26.0	± 7.8	70.6	96.8	11.3	± 4.1
Section Std. Deviation	13.5	26.1	1.8	± 2.5	10.5	34.0	1.5	± 1.3
Section CV, %	30.0	65.6	1.9	± 6.5	20.2	90.8	1.6	± 3.3
No. of Data Sets	44				138			
t-test at 95% level for CV	Unequal mean for Section CV							
F-test at 95% level for CV	Equal Variance for Section CV							

Table 32. Comparison of Variability for JCP-PJ with Different Subgrade Soils

Parameter	Granular Soil Subgrade Mean	Granular Soil Subgrade Maximum	Granular Soil Subgrade Minimum	Granular Soil Subgrade 95% CI	Fine-Grained Soil Subgrade Mean	Fine-Grained Soil Subgrade Maximum	Fine-Grained Soil Subgrade Minimum	Fine-Grained Soil Subgrade 95% CI
Average Section LTE, %	63.7	95.4	15.8	± 5.8	72.4	96.4	25.8	± 5.1
Section Std. Deviation	13.5	34.0	1.6	± 2.1	10.2	25.0	1.5	± 1.6
Section CV, %	26.8	68.0	1.7	± 4.9	19.4	67.7	1.6	± 4.1
No. of Data Sets	66				85			
t-test at 95% level for CV	Unequal mean for Section CV							
F-test at 95% level for CV	Equal Variance for Section CV							

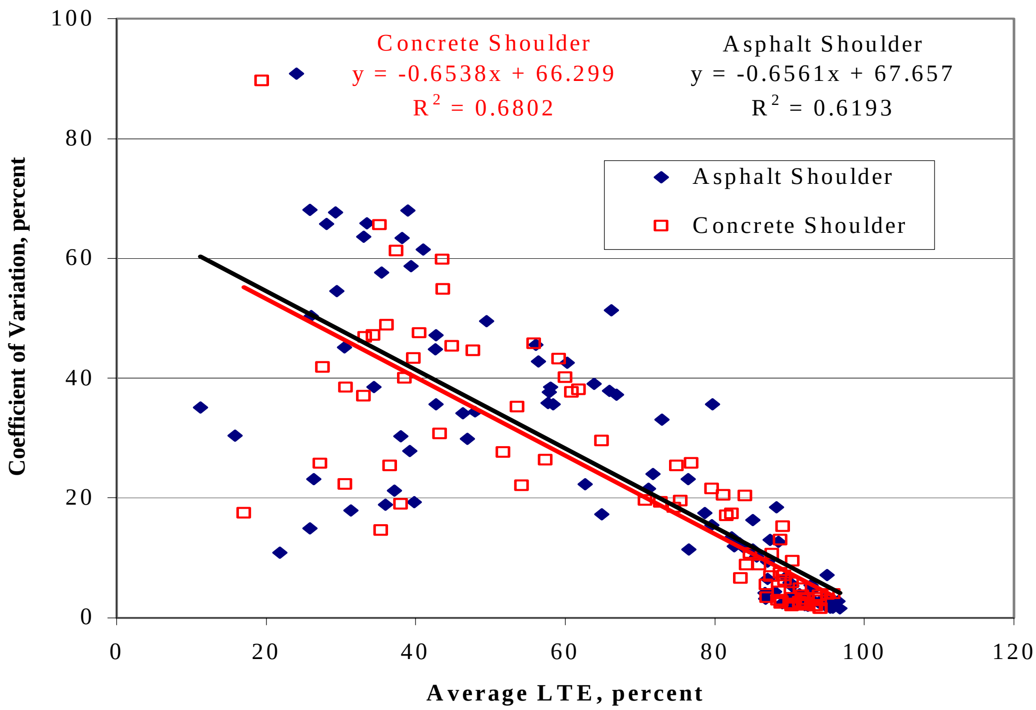


Figure 3. CV vs. Average LTE for JCP-PJ with Concrete and Asphalt Shoulders

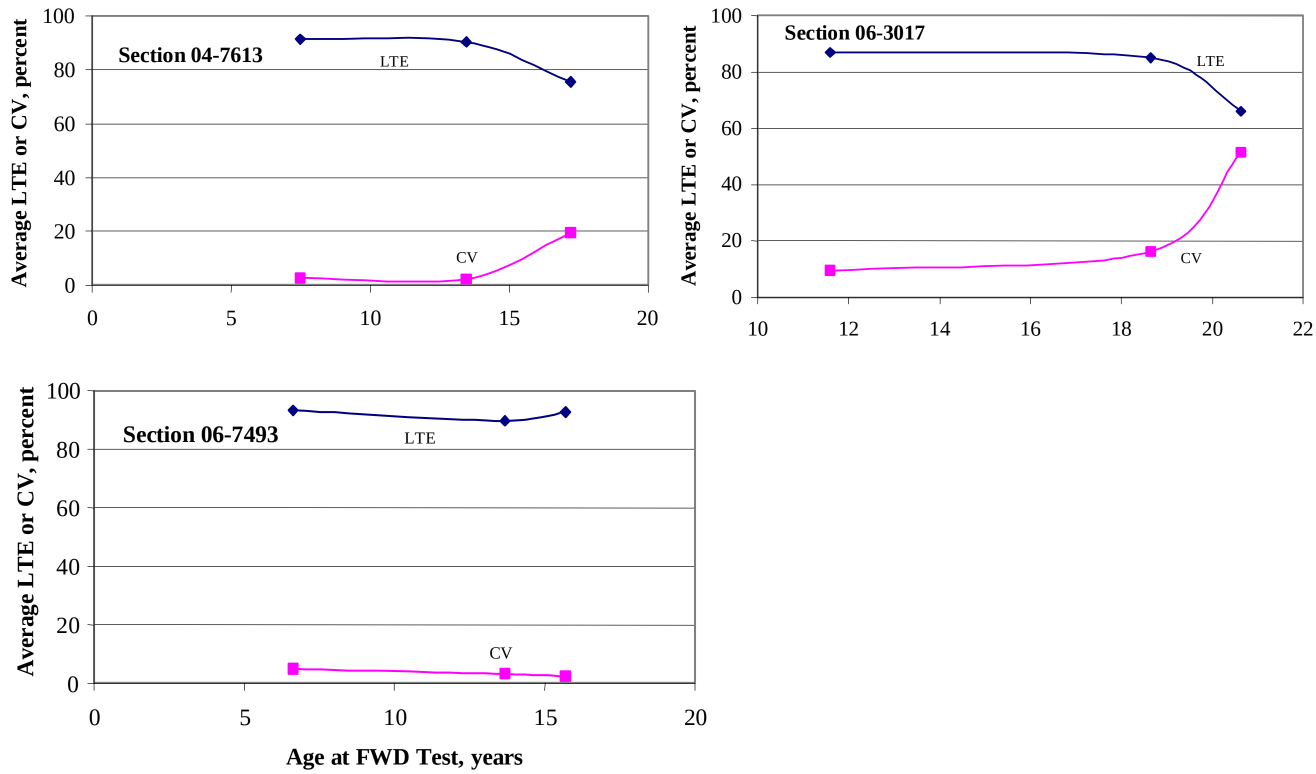


Figure 4. Long-Term Effects on CV and Average LTE for JCP-PJ

Analyses of Variability of LTE for JPC-DJ

Assessment of Variability of LTE Derived under J4 and J5 Loading

This analysis was the same as the analysis performed for JPC-DJ described earlier. For the FWD tests conducted on each test section at each time of testing, a student t-test was conducted to compare the average LTEs computed from the J4 and the J5 FWD deflection data, and an F-test was conducted to compare the associated variance of the J4 and J5 LTEs. The data were considered to come from the same population if, at the 95% confidence level, the null hypotheses of equal mean and equal variance could not be rejected. The analyses showed that, out of the 653 data sets, 202 (or 31%) exhibited significant differences between the LTEs determined from the J4 and J5 loading conditions.

To further analyze the differences between the average LTEs and CVs derived from data under the J4 and J5 FWD loads, the difference between the two average LTEs and the two CVs were calculated for each test section tested, at each testing time (J4-J5). Both the average LTEs and the CVs showed a wide range of variation. The difference in average LTE ranged from -38% to +15% with an average of -2%, while the difference in CV varied between –32% and +56% with an average value of +1%. However, from the distributions of the differences presented in Appendix B, Figures B11 and B12, for the average LTEs and the CVs, respectively, it can be observed that, for both the average LTE and the CV, the majority of the differences were very small. For both the average LTE and the CV, about 94% of the differences were within ±10%.

A comparison of the average LTEs and the CVs for the J4, J5, and combined loading conditions is shown in Table 33 for the JPC-DJ. No appreciable differences were observed on the average LTEs and CVs computed from deflections under the two different FWD loading positions. The analysis showed that the distribution of differences between the average LTEs and the CVs determined from the deflections obtained under the J4 and J5 loadings were concentrated within a narrow range (~±10%) around zero. Also, in actual FWD deflection data analyses for pavement-related issues, the data collected from both loading positions are typically used together. The combined FWD deflection data were therefore used for further variability analyses related to LTE for concrete pavements with plain joints.

General Assessment of the Variability of LTE

To evaluate the general trends of the variability of the LTE for the JPC-DJ, the CV was plotted as a function of the average LTE. Similar to that observed for plain jointed pavements, the CV was inversely related to the average LTE of the sections. As the average LTE decreased, the CV increased. Also, the variations of the CVs among different sections tested at different times at lower LTE levels were much greater than those at higher levels of LTE.

Table 33. Effect of FWD Load Position on Average LTE and CV for JPC-DJ Pavements

	Load Applied at						Combined Data		
	Approach Slab (J4)			Leave Slab (J5)					
	Mean	Max.	Min.	Mean	Max.	Min.	Mean	Max.	Min.
Average LTE, %	79	100	8	81	100	9	80	98	9
Section Std. Dev., %	8	40	1	7	41	1	8	40	1
CV, %	12	98	1	11	81	7	12	1	97

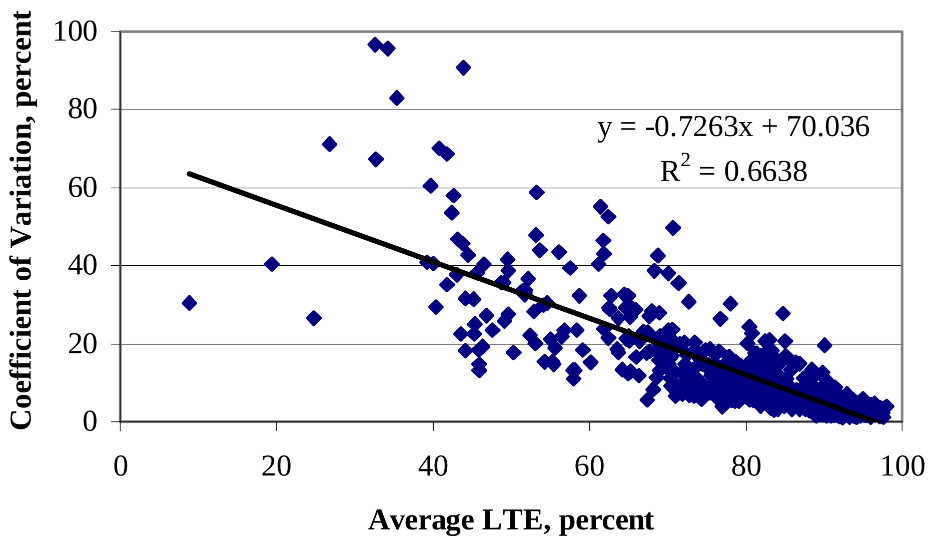


Figure 5. CV vs. Average LTE for Concrete Pavements with Doweled Joints

Table 34. Summary Statistics of CV Associated with Average LTE for JPC-DJ Pavements

Parameter	Mean	Maximum	Minimum	75% Point	95% CI
Average Section LTE, %	80.2	97.9	8.9	>73.6	± 1.1
Section Std. Deviation	7.8	39.9	1.0	<10.8	± 0.5
Section CV, %	11.7	96.6	1.0	<14.6	± 1.0
No. of Data Sets	653				

The distribution of the average LTEs and the CVs are presented in Appendix B, Figures B13 and B14 respectively. Both the average LTE and the CV showed some ranges of variations; however, the majority of the average LTEs were greater than 80% and the majority of the average CVs were less than 10%. A summary of the CVs associated with the average LTEs for the different sections tested at different times is presented in Table 34. As shown in this figure, the CV ranged from 1% to 97%, with an average value of 12% (± 1% at 95% confidence level). Also, 75 percent of the CVs were found to be less than 15%.

Effects of Joint Spacing on the Variability of LTE

To evaluate the effects of joint spacing on the variability of LTE, the calculated CVs were plotted against the average joint spacing of the different test sections, as shown in Appendix B, Figure B15. As indicated in this figure, no apparent relationship between the CV and average joint spacing could be observed.

Effects of Pavement Base Type on the Variability of LTE

In this analysis, the different types of materials used as the base layer in the pavement structures in the LTPP program were again combined into two different groups, the bound base and the unbound base. The CV data were separated according to the two base types and summary statistics were computed for each group of data. A student t-test and an F-test were also performed to compare if the two groups of CV data had equal mean and equal variance. The results of the analyses are presented in Table 35. As seen from the table, the average CVs were 12% for pavement sections with both bound and unbound base. At 95% confidence level, the two groups of CVs were determined to have equal mean, inferring that the base type did not have any effects on the variability of the calculated LTE using the FWD data in the LTPP program.

Effects of Outside Shoulder Type on the Variability of LTE

Similar to the analysis performed for JPC-PJ, pavement sections for JPC-DJ sections were separated into two groups, pavements with concrete shoulders and those with asphalt-surfaced shoulders. Summary statistics were computed and a student t-test and an F-test were performed on these two groups of CV data. The results of these analyses are shown in Table 36. The variability of the average LTE for pavements with concrete shoulders was statistically higher than that for pavements with asphalt shoulders. This difference is further substantiated in Figure 6, where the CVs were plotted as a function of average LTE for both groups of pavements.

Table 35. Comparison of Variability for JPC-PJ with Bound and Unbound Base

Parameter	Bound Base				Unbound Base			
	Mean	Maximum	Minimum	95% CI	Mean	Maximum	Minimum	95% CI
Average Section LTE, %	79.0	97.9	24.7	± 1.5	82.0	97.7	8.9	± 1.8
Section Std. Deviation	7.9	35.1	1.0	± 0.6	7.8	39.9	1.1	± 0.8
Section CV, %	11.8	96.7	1.0	± 1.2	11.7	95.6	1.2	± 1.8
No. of Data Sets	386				267			
t-test at 95% level for CV	Equal mean for Section CV							
F-test at 95% level for CV	Unequal Variance for Section CV							

Table 36. Comparison of Variability for JPC-PJ with Concrete and Asphalt Shoulders

Parameter	Concrete Shoulder				Asphalt Shoulder			
	Mean	Maximum	Minimum	95% CI	Mean	Maximum	Minimum	95% CI
Average Section LTE, %	78.5	97.9	8.9	± 2.0	81.1	97.6	24.7	± 1.6
Section Std. Deviation	9.0	39.9	1.5	± 0.9	7.3	33.8	1.0	± 0.7
Section CV, %	14.1	96.7	1.6	± 1.9	10.7	71.1	1.0	± 1.2
No. of Data Sets	261				333			
t-test at 95% level for CV	Unequal mean for Section CV							
F-test at 95% level for CV	Unequal Variance for Section CV							

Table 37. Comparison of Variability for JPC-PJ With and Without Base Drainage

Parameter	With Drainage				Without Drainage			
	Mean	Maximum	Minimum	95% CI	Mean	Maximum	Minimum	95% CI
Average Section LTE, %	79.6	97.7	8.9	± 2.1	80.2	97.9	24.7	± 1.5
Section Std. Deviation	7.3	29.3	1.2	± 0.7	8.4	39.9	1.0	± 0.7
Section CV, %	10.6	67.2	1.2	± 1.4	13.0	96.7	1.0	± 1.5
No. of Data Sets	190				404			
t-test at 95% level for CV	Unequal mean for Section CV							
F-test at 95% level for CV	Unequal Variance for Section CV							

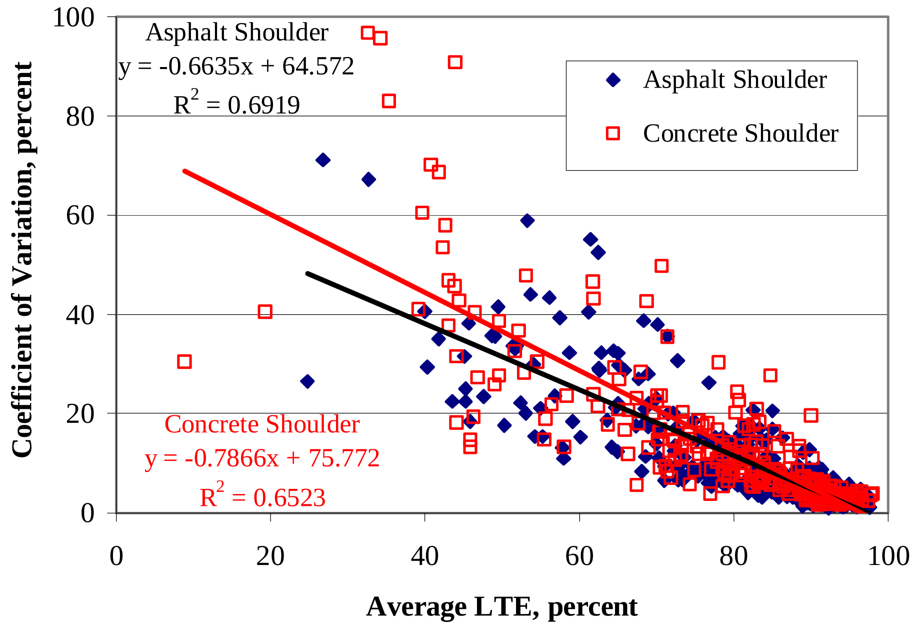


Figure 6. CV vs. Average LTE for JPC-DJ with Concrete and Asphalt Shoulders

Effects of Sub-Surface Drainage on the Variability of LTE

In this analysis, as previously, the different types of sub-surface drainage were grouped into two categories, pavements with and without a sub-surface drainage system. The CV data were grouped according to this categorization. Summary statistics were calculated for CVs of each group and are shown in Table 37. Also shown in this table are the results of the t-test and the F-test. The average CV was 11% for the pavements using a drainage system and was 13% for the pavements without a drainage system. The t-test also showed that, at 95% confidence level, the CV for pavements with a drainage system was lower than that for pavements without a drainage system.

Effects of Climatic Parameters on the Variability of LTE

Effects of four climatic parameters on the variability of LTE were analyzed. The four climatic parameters evaluated were annual precipitation, annual freezing index, the number of annual freeze-thaw cycles, and the average mean annual temperature. The analyses were performed by plotting the CVs against these four climatic parameters, as shown in Appendix B, Figures B16 to B19. It can be observed from these figures that the annual precipitation, the number of annual freeze-thaw cycles, and the average mean annual temperature did not show any effect on the CVs associated with the average LTE. However somewhat surprisingly, the CVs seemed to decrease as the annual freezing index increased. Almost all the CVs were less than 20% for pavements located at sites with an annual freezing index greater than about 600 °C-day.

Effects of Pavement Age on the Variability of LTE

The effects of pavement age at the time of FWD testing on the variability of LTE were assessed in this study. Since the CV is a function of the average LTE, an effort was made to analyze the age effects with respect to the average LTE for individual pavement sections. For these individual pavement sections, the average LTE and its associated CV were plotted as a function of the pavement age, as presented in Appendix B, Figure B20, for three LTPP pavement sections, 04-7614, 13-3007, and 01-4007. From this figure it is apparent that the CV is closely related to the average LTE. For pavement sections with consistent high average LTE, the CV remained constantly low over the years. Once the average LTE decreased, the CV increased accordingly.

Analyses of Variability of LTE for CRC Pavements

Assessment of Variability of LTE Derived under C4 and C5 Loadings

In the LTPP FWD load-deflection test program, the FWD loads were applied on both sides of the cracks, on the approach slab and on the leave slab. The load position was designated as C4 for the approach loading and C5 for the leave slab loading. The average LTEs and their associated CVs were calculated for deflection data obtained under the C4 and C5 loading conditions and for the combined deflection data. An attempt was made in this study to evaluate if the average LTEs and their associated CVs derived from the FWD deflection data obtained under the C4 and C5 loads were from the same population.

For the FWD tests conducted on each test section at each time of testing, a student t-test was conducted to compare the average LTEs computed from the C4 and C5 deflection data, and an F-test was conducted to compare the associated variance of the C4 and C5

LTEs. The data were considered to come from the same population if, at the 95% confidence level, the null hypotheses of equal mean and equal variance could not be rejected. The analyses showed that, out of the 187 data sets, 78 (or 42%) exhibited significant differences between the LTEs determined from the C4 and C5 loading conditions.

To further analyze the differences between the average LTEs and CVs derived from data under the C4 and C5 loads, the difference between the two data sets were calculated for each test section tested, at each testing time (C4-C5). The difference in average LTE ranged from -6% to +3% with an average value of -1%, while the difference in CV varied between –3% and +5%, with an average value of 0%. The distributions of these differences are presented in Appendix B, Figures B21 and B22, for the average LTEs and the CVs respectively. It can be observed from these figures that, for both the average LTE and CV, the difference between the values determined from the C4 and C5 deflection data were extremely small.

A comparison of the average LTEs and the CVs for the C4, C5 and combined loading conditions is shown in Table 38 for the CRC Pavements. Very little difference was observed in average LTE or CV, as computed from the measured FWD deflections under the two different loading positions. The analysis shows that very little difference exists between average LTE or CV determined from the deflection data obtained under the C4 and C5 loading and the distribution of the differences concentrated within a narrow range (within ± 5%) around zero. Also, in actual FWD deflection data analyses for pavement-related issues, the data collected from both loading positions will most likely be used together. The combined FWD deflection data were therefore used for further variability analyses related to LTE for continuously reinforced concrete pavements with intermittent cracks.

General Assessment of the Variability of LTE

To evaluate the general trends of the variability of the LTE for CRC pavements, the CV was plotted as a function of the average LTE, as shown in Figure 7. Unlike the general trends observed for jointed pavements, no apparent relationships between the CV and average LTE were observed. However this may be expected, since the ranges for both the CV and average LTE were very small for CRC pavements.

The distribution of average LTE and CV are presented in Appendix B, Figures B23 and B24 respectively. In general, all average LTEs were greater than 80%, with 88% of the test sections having an average LTE greater than 90%. Similarly, all CVs were less than 10%, with 92% of the test sections having a CV less than 5%. A summary of the CVs associated with the average LTEs for the different sections tested at different times is presented in Table 39. As shown in this table, the CV ranged from 1% to 13%, with an average value of 3% (± 0.2% at 95% confidence level). Also, 75 percent of the CVs were found to be less than 3.5%.

Table 38. Effect of FWD Load Position on Average LTE and CV for CRC Pavements

	Load Applied at						Combined Data		
	Approach Slab (C4)			Leave Slab (C5)					
	Mean	Max.	Min.	Mean	Max.	Min.	Mean	Max.	Min.
Average LTE, %	91	100	77	92	101	78	91	100	77
Section Std. Dev., %	3	10	1	2	10	1	3	10	1
CV, %	3	12	1	3	13	1	3	13	1

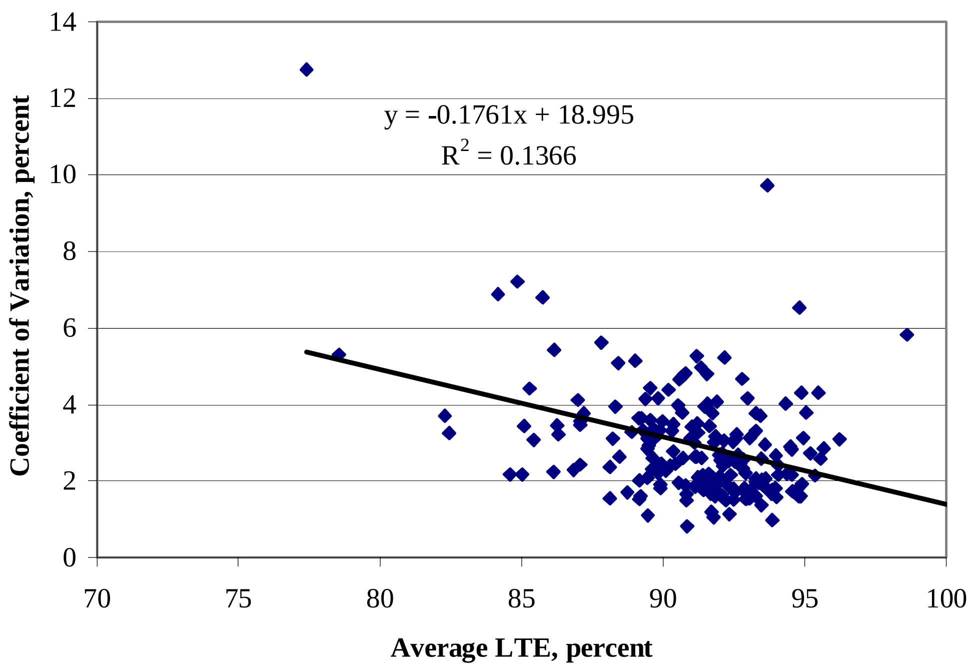


Figure 7. CV vs. Average LTE for CRC Pavements

Table 39. Summary Statistics of CV Associated with Average LTE for CRC Pavements

Parameter	Mean	Maximum	Minimum	75% Point	95% CI
Average Section LTE, %	91.1	100.4	77.4	89.6	± 0.4
Section Std. Deviation	2.7	9.9	0.7	3.2	± 0.2
Section CV, %	3.0	12.7	0.8	<3.5	± 0.2
No. of Data Sets	187				

Effects of Crack Spacing on the Variability of LTE

The average crack spacing for each CRCP section was calculated by dividing the length of the section (152 m) by the total number of cracks, as obtained by the latest manual distress surveys. To evaluate the effects of crack spacing on the variability of LTE, the calculated CVs were plotted against the average crack spacing of the different test sections, as shown in Figure 8. As indicated in this figure, no apparent relationship between the CV and the average crack spacing can be observed.

Effects of Pavement Base Type on the Variability of LTE

In this analysis, the different types of materials used as the base layer in the pavement structures in the LTPP program were once again combined into two different groups: bound and unbound bases. The CV data were separated according to these two base types and summary statistics were computed for each group of data. A student t-test and an F-test were performed to compare if the two groups of CV data had equal means and equal variances. The results of the analyses are presented in Table 40. As seen in this table, the average CVs were about 3% for pavement sections with both bound and unbound bases. At a 95% confidence level, the two groups of CVs were determined to have equal means, inferring that the base type does not have any effect on the variability of the calculated LTE using the FWD load-deflection data in the LTPP program.

Effects of Slab Stiffness on the Variability of LTE

Slab stiffness (D) is defined as:

$$D = [Eh^3]/[12\ (1-\mu^2)]$$

Where:

D = Slab stiffness, MN-m

E = Concrete modulus of elasticity, GPa

H = Slab thickness, mm

μ = Poisson's ratio, a value of 0.15 was used in this analysis.

To evaluate the effects of pavement slab stiffness on the variability of LTE, the calculated CVs were plotted against the calculated slab stiffness for the different test sections as shown in Appendix B, Figure B25. As indicated in this figure, no apparent relationship between the CV and slab stiffness could be observed.

Effects of Outside Shoulder Type on the Variability of LTE

Similar to the analysis performed for jointed concrete pavements, the CRC pavement sections were separated into two groups: pavements with concrete shoulders and those with asphalt shoulders. Summary statistics were computed and a student t-test and an F-test were performed on these two groups of CV data. The results of these analyses are shown in Table 41. The variability of the average LTE for CRC pavements with concrete shoulders was statistically equivalent to the corresponding value for CRC pavements with asphalt shoulders.

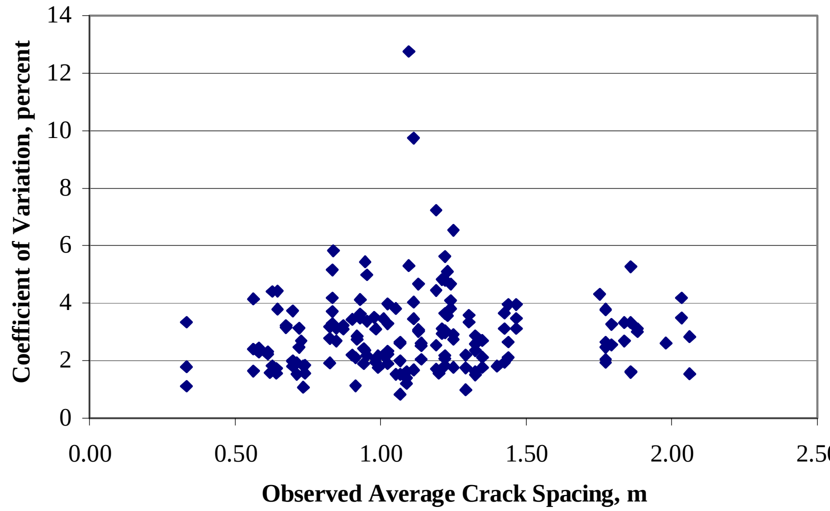


Figure 8. CV vs. Average Crack Spacing for CRC Pavements

Table 40. Comparison of Variability for CRCP with Bound and Unbound Base

Parameter	Bound Base				Unbound Base			
	Mean	Maximum	Minimum	95% CI	Mean	Maximum	Minimum	95% CI
Average Section LTE, %	90.7	100.4	77.4	± 0.5	92.6	95.7	87.1	± 0.6
Section Std. Deviation	2.7	9.9	0.7	± 0.2	2.6	9.1	0.9	± 0.5
Section CV, %	3.0	12.7	0.8	± 0.2	2.8	9.7	1.0	± 0.5
No. of Data Sets	146				41			
t-test at 95% level for CV	Equal mean for Section CV							
F-test at 95% level for CV	Equal Variance for Section CV							

Table 41. Comparison of Variability for CRCP with Concrete and Asphalt Shoulders

Parameter	Concrete Shoulder				Asphalt Shoulder			
	Mean	Maximum	Minimum	95% CI	Mean	Maximum	Minimum	95% CI
Average Section LTE, %	90.9	96.2	77.4	± 0.8	91.2	100.4	82.3	± 0.6
Section Std. Deviation	2.7	9.9	0.7	± 0.3	2.7	9.1	1.0	± 0.2
Section CV, %	3.0	12.7	0.8	± 0.4	2.9	9.7	1.1	± 0.1
No. of Data Sets	75				112			
t-test at 95% level for CV	Equal mean for Section CV							
F-test at 95% level for CV	Equal Variance for Section CV							

Effects of Climatic Parameters on the Variability of LTE

Effects of two climatic parameters on the variability of LTE were analyzed. The two climatic parameters used were the annual precipitation and the annual freezing index. This analysis was performed by plotting CV values against the two climatic parameters, as shown in Appendix B, Figures B26 and B27. No clear trends could be observed for either the annual precipitation or the annual freezing index. However, the CV seems to decrease as the annual freezing index increases.

Effects of Pavement Age on the Variability of LTE

The effects of pavement age at the time of FWD testing on the variability of LTE for CRC pavements were analyzed. The CV was plotted as a function of pavement age for 23 individual pavement sections (see Appendix B, Figure B28). All the 23 sections were subjected to FWD tests at three or more different times, covering at least five years between the first and the last tests. Because of the generally very low values of CVs for all the test sections, no clear trends can be observed in this figure.

Summary of LTE for PCC Pavements

A summary of the above data and conclusions on Load Transfer Efficiency (LTE) is presented in *Section 4.2 – Summary of Portland Cement Concrete Parameter Variability*.

2.2.3 Variability of Back-Calculated Moduli Data for PCC Pavements

As part of the LTPP monitoring program, deflection testing using the falling weight deflectometer (FWD) was (and still is) being performed periodically (every few years) at GPS and SPS test sections. At the 64 sections designated for the Seasonal Monitoring Program (SMP), FWD testing is performed more frequently, about 12 to 14 times per year. Back-calculation of the deflection data was performed to obtain the layer stiffness at each section for each testing time. Up to a maximum of 20 tests were performed within a pass at each GPS or SPS section. Additionally, at the SMP sections, multiple tests were performed during the day of testing. Only 10 locations (stations) were tested at each SMP section.

It should be noted that the back-calculation analysis process is still not a "perfect" technology and very little ground truth calibrations or validations have been carried out to ascertain the validity of back-calculated parameters. For PCC pavements, back-calculation was performed using the assumption of the Winkler (liquid) and elastic solid foundation for the subgrade developed by Khazanovich et al.(1) Back-calculation was performed for the interior loading condition (J1 pass for the jointed concrete pavements and C1 pass for continuously reinforced concrete pavements).

Data Source

Back-calculation data were obtained directly from the LTPP Customer Support Service during February 2001. These data were part of the LTPP Data Release 11.1, updated on December 22, 2000. A specific data table is present where the back-calculated moduli are listed, and the variability associated with these back-calculated moduli was derived from this Level E data table. This table contains a summary of the statistical parameters for each PCC section. The moduli parameters studied are as follows:

1. Modulus of subgrade reaction, k.
2. Concrete modulus of elasticity – liquid foundation, $E_{c\text{-}l}$
3. Subgrade modulus of elasticity – elastic solid foundation, $E_{s\text{-}es}$.
4. Concrete modulus of elasticity – elastic solid foundation, $E_{c\text{-}es}$.

Analysis Approach

For each section, information was available for the mean value and the standard deviation of the four moduli parameters, for each test pass (several test passes on a given day at seasonal sites or at different times). The mean and the standard deviation values are used to describe the spatial variability in the appropriate moduli parameter. In addition, the availability of data for different test passes (for the same day or at different times) provides an opportunity to investigate if the spatial variability remains consistent from one testing time to another.

Overall Analysis of Back-calculated Moduli Variability

A total of just over 740 records were available for analysis for JPC. Each record denotes one pass of FWD testing. Thus, three different tests in a given day at a section would result in three records. Table 42 summarizes the overall variability in the back-calculated moduli data for JPC pavements. Details for each section and test pass are given in Table B22 in Appendix B.

For CRC pavements, a total of 123 records were available. Table 43 summarizes the overall variability in the back-calculated moduli data for CRC pavements. Details for each section and test pass are given in Table B23 in Appendix B.

It is seen from Table 42 and Table 43 that, on average, the back-calculated parameters are reasonably consistent within a test section – with a mean CV of 11-15% for JPC pavements and 12-18% for CRC pavements. Also, within each section, the variability is reasonably consistent from one test pass (or visit) to another, as discussed in the following.

Analysis of Modulus of Subgrade Reaction, k, Data

The moduli of subgrade reaction data were analyzed to determine changes in the spatial variability with time (and seasonally) for a select group of LTPP pavement sections. Figure 9 and Figure 10 show the typical ranges in CV for the k-value for Sections 13-3019 and 27-4040, respectively. As seen in these figures, the range in variability of the CV for most of the data is within 15%. This is considered fairly good consistency in the spatial variability in the subgrade for a given test section. This level of variability is evidently consistent from one test time to another. Variability in the k-value over time for Sections 20-4054, 36-4018, 48-4142 and 49-3011 is shown in Appendix B, Figures B29 to B32 respectively.

Analysis of Other Back-Calculated Parameters

The variability with time in the CV for the concrete modulus of elasticity values (based on liquid foundation and elastic solid foundation) and subgrade modulus of elasticity (based on elastic foundation) were also analyzed. The trends in the variability of CV over time for these parameters were similar to that discussed above for the modulus of subgrade reaction parameters. The results of this analysis are therefore not included here.

Other analyses performed indicate a strong correlation between the variabilities in k and E_{c-l} and a weak correlation between the PCC slab thickness variability and the variability in E_{c-l}.

Summary of Back-Calculated Moduli for PCC Pavements

A summary of the above data and conclusions on back-calculated moduli is presented in *Section 4.2 – Summary of Portland Cement Concrete Parameter Variability.*

Table 42. Within Pass Variability of Back-Calculated Moduli for JPC Pavements

Parameter	Mean	Minimum	Maximum	Standard Deviation	CV, %	No. of Records
K Value:						
Section Mean, kPa/mm	59	20	213	24	40.7	744
Section SD, kPa/mm	9	1	56	7	74.4	744
Section CV, %	14.5	2.9	48.9	7.7	53.5	744
Sample Size within Section	25	3	119	18	69.0	744
Ec-l Value:						
Section Mean, MPa	44,399	17,356	69,356	9,215	20.8	744
Section SD, MPa	5,493	728	19,474	2,740	50.0	744
Section CV, %	12.6	1.9	61.9	6.6	52.4	744
Sample Size within Section	25	3	119	18	69.0	744
Es-es Value:						
Section Mean, MPa	208	72	500	69	33.0	748
Section SD, MPa	23	2	121	16	70.0	748
Section CV, %	11.00	2.00	37.10	6.10	55.7	748
Sample Size within Section	28	5	123	18	65.0	748
Ec-es Value:						
Section Mean, MPa	34,242	13,492	60,325	8,238	24.1	748
Section SD, MPa	4,992	630	19,062	2,818	56.0	748
Section CV, %	14.6	2.1	62.8	7.6	52.0	748
Sample Size within Section	28	5	123	18	65.0	748

Table 43. Within Pass Variability of Back-Calculated Moduli for CRC Pavements

Parameter	Mean	Minimum	Maximum	Standard Deviation	CV, %	No. of Records
K Value:						
Section Mean, kPa/mm	71	26	149	26	37.5	123
Section SD, kPa/mm	12	2	55	9	76.0	123
Section CV, %	16.8	4.8	50.1	9.3	55.3	123
Sample Size within Section	45	16	66	12	26	123
Ec-l Value:						
Section Mean, MPa	42,437	27,019	66,352	8,805	20.7	123
Section SD, MPa	6,698	2,537	17,996	3,015	45.0	123
Section CV, %	16.1	6.4	43.1	7.3	45.0	123
Sample Size within Section	45	16	66	12	26	123
Es-es Value:						
Section Mean, MPa	228	92	421	78	34.0	123
Section SD, MPa	28	5	95	18	64.4	123
Section CV, %	11.9	3.3	36.5	6.1	51.4	123
Sample Size within Section	49	19	66	11	22	123
Ec-es Value:						
Section Mean, MPa	31,939	19,746	55,853	7,343	23.0	123
Section SD, MPa	5,671	2,194	14,835	2,767	48.8	123
Section CV, %	17.8	7.0	39.3	7.3	41.2	123
Sample Size within Section	49	19	66	11	22	123

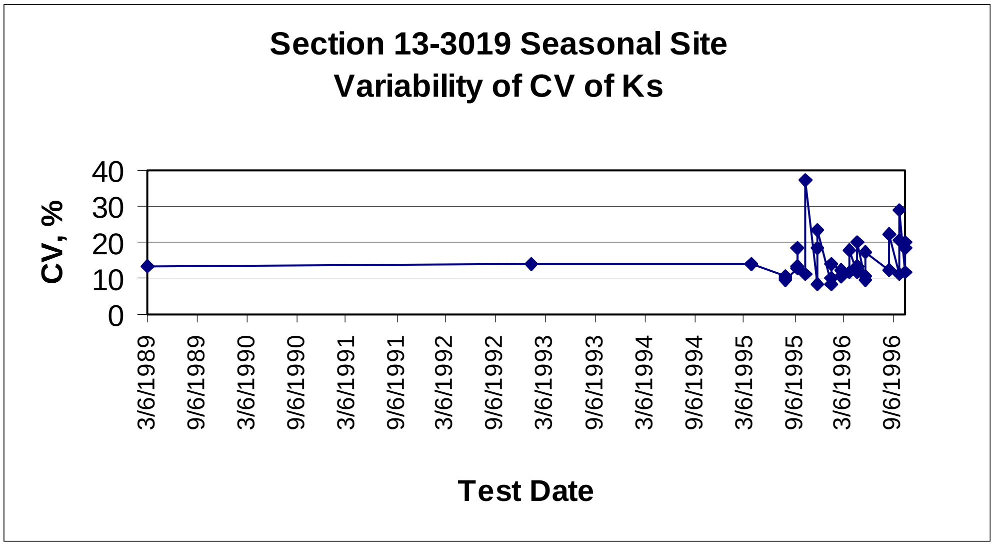


Figure 9. Variability (in CV) of the Modulus of Subgrade Reaction, Section 13-3019

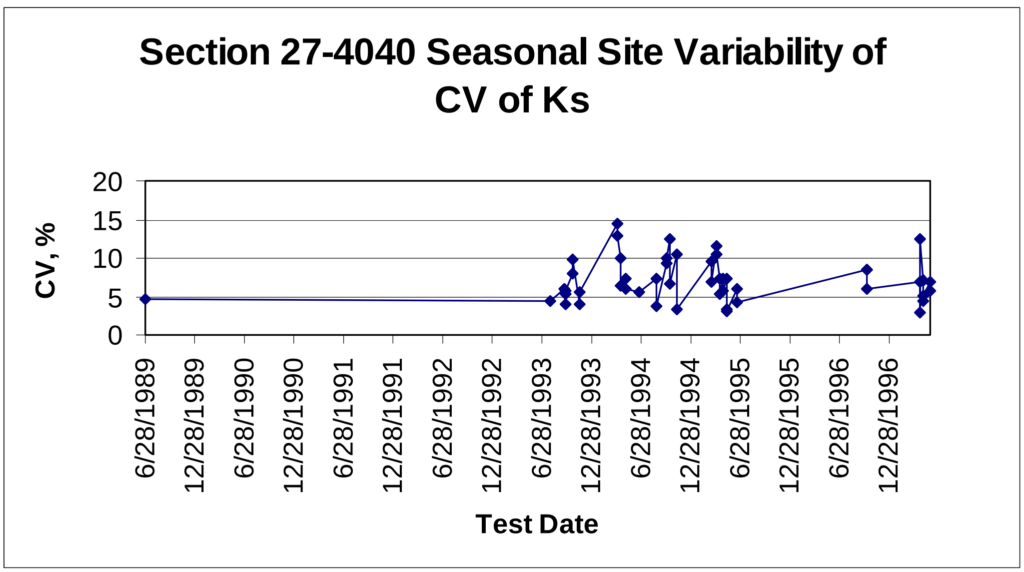


Figure 10. Variability (in CV) of the Modulus of Subgrade Reaction, Section 27-4040

2.3 VARIABILITY OF AC-RELATED DESIGN PARAMETERS

2.3.1 AC Modulus Variability Based on Back-Calculation

Deflection measurements have been made with the Falling Weight Deflectometer (FWD) on the entire set of asphalt concrete surfaced General Pavement Studies (GPS) and Specific Pavement Studies (SPS) test sections that are included in the Long-Term Pavement Performance (LTPP) program. This deflection-testing program is being conducted to obtain the load-response characteristics of the pavement structure and the subgrade.

As mentioned in the previous section on the analysis of Portland Cement Concrete (PCC) pavements, one of the more common methods used to interpret deflection data is to "back-analyze" the elastic properties of each layer in the pavement structure and foundation (subgrade). These analysis methods (referred to as back-calculation programs) provide the elastic layer moduli typically used for pavement evaluation and rehabilitation design.

The use of linear-elastic analysis programs has been only partly successful in analyzing the deflections measured at LTPP sections. For example, only about 50 percent of the flexible GPS sections were found to have error terms less than two percent per sensor, as reported by Von Quintus and Killingsworth[(2) (3)], a value that is generally considered to be an acceptable limit for back-calculation purposes.

As a result, the FHWA sponsored a study under the LTPP Data Analysis Technical Support Study (Contract No. DTFH61-96-C-00003) to calculate the load-response characteristics from the deflection data measured on all LTPP test sections and attempt to improve upon previous back-calculation analyses[(4)].

The procedure utilized to determine the back-calculated values from this study is depicted in Appendix C, Figure C1. As shown in the figure, MODCOMP4 was used to determine the back-calculated pavement layer moduli for the LTPP deflection data.

Once the back-calculation was completed, tables were generated for inclusion into the LTPP IMS. The primary IMS table used to complete the analysis for the determination of expected modulus variability for use in pavement design is called the "Table of Back-calculated Moduli." Also, other tables containing data applicable to the back-calculation analysis were also used. The terms included in these IMS tables are defined in Tables C1 to C3 of Appendix C.

Figures C2, C3 and C4 of Appendix C demonstrate that some sections may have very consistent deflection measurements throughout the entire section while others tend to have significant differences throughout, or from one end to the other. In addition, deflections can change significantly throughout the life of the pavement, either from changes in seasonal temperatures or from aging of the pavement materials and/or degradation of the pavement structure (see also Appendix C, Figure C5).

In addition to back-calculated moduli, most AC-surfaced pavements in the LTPP database have laboratory resilient moduli values determined using the standard LTPP protocol. Available data in the LTPP program include the results of the P-07 test protocol, M_r and Creep Compliance data. The P-07 tests were run on cores taken from the sampling areas associated with existing pavement sections. The sections represent a wide variety of HMA mixes and a wide variety of ages.

Objective

As with the other sections in this report, the objective of the study of back-calculated and laboratory determined layer moduli is to establish a set of recommended ranges, for example for the coefficients of variation, for asphalt-bound material properties. Along with asphalt properties, the other layers typically found in AC- and PCC-surfaced pavements also need to be determined, applicable globally, regionally, and on a project-specific basis.

Variability of Back-Calculated Moduli for AC Surfaced Pavements

The variability of the back-calculated moduli for asphalt concrete (AC) pavements was studied by utilizing data from the LTPP project IMS database extraction release date 8/25/00. The data used contained both level E and non-level E data. It was necessary to use non-level E data so the analysis would be meaningful. However, care was taken to review the non-level E data to ensure that the analysis would not be adversely affected by some data anomaly present within these data.

Two flexible pavement IMS data tables were used to complete this portion of the analysis. The first table contains the mean back-calculated moduli for each layer in each section, for each year of testing. It also contains other statistics such as the standard deviation and the number of points used (see Tables C1-C3 in Appendix C). The second table contains the layer thickness information. All back-calculation was completed using deflections normalized to the average load at each of the four different FWD drop heights called for in LTPP's FWD testing protocol where the target standard loads are 26.7, 40.0, 53.4 and 71.2 kN, or thousand Newtons (6, 9, 12 and 16 kips, or thousand pounds). However, neither the deflections nor the pavement layer moduli were transformed to a standard test temperature. It was felt that making these temperature corrections might introduce unnecessary errors into the analysis, thus making it more difficult to isolate the causes of the variability in the back-calculation process.

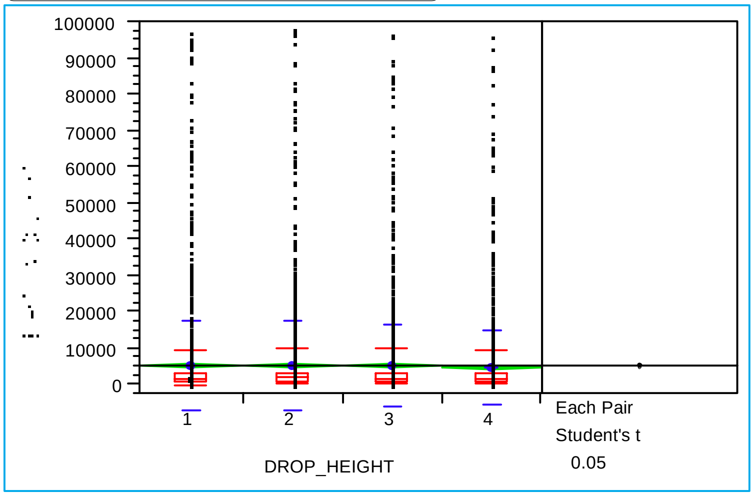
Univariate Analyses

The first analysis performed was the review of the standard deviation of the AC layer back-calculated modulus for all of the sections included in the IMS. This was divided up according to the FWD drop height. The purpose of this exercise was to determine the range and spread of back-calculated moduli variability for each AC layer. The results are summarized in Table 44.

The results show that there is great variability in the standard deviation of the back-calculated moduli for AC pavements. In addition, Table 44 also shows that the drop height does not have an effect on the standard deviation. This is verified by comparing the means of the standard deviations for all possible combinations between the drop heights. The bottom box of Table 44 contains the results of the comparisons of the means at an alpha level equal to 0.05 (where t = 1.96038). If any of the combinations were statistically different, then a positive number would result in this matrix. Since no positive numbers are shown, it can be assumed that the variability of the back-calculated AC modulus is independent of the drop height (or the imparted FWD load).

Table 44. Univariate Analysis on Standard Deviations of Back-Calculated AC Moduli

ELASTIC_MODULUS_STD By DROP_HEIGHT



Oneway Anova

Summary of Fit

RSquare	0.000347
RSquare Adj	-0.00014
Root Mean Square Error	11965.36
Mean of Response	5188.281
Observations (or Sum Wgts)	6217

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	3	308723176	1.0291e8	0.7188
Error	6213	8.89514e11	1.4317e8	Prob>F
C Total	6216	8.89823e11	1.4315e8	0.5406

Means for Oneway Anova

Level	Number	Mean	Std Error
1	1628	5271.76	296.55
2	1524	5466.19	306.50
3	1535	5164.57	305.40
4	1530	4846.42	305.90

Std Error uses a pooled estimate of error variance

Means Comparisons

Dif=Mean[i]-Mean[j]	2	1	3	4
2	0.000	194.424	301.617	619.771
1	-194.424	0.000	107.192	425.346
3	-301.617	-107.192	0.000	318.154
4	-619.771	-425.346	-318.154	0.000

Alpha= 0.05

Comparisons for each pair using Student's t

t
1.96038

Abs(Dif)-LSD	2	1	3	4
2	-849.746	-641.640	-546.605	-229.141
1	-641.640	-822.156	-727.323	-409.871
3	-546.605	-727.323	-846.695	-529.233
4	-229.141	-409.871	-529.233	-848.078

Positive values show pairs of means that are significantly different.

Correlation Analysis

Correlation between Surface and Subgrade Modulus Variability

Secondly, correlations were computed between the subgrade and surface modulus variability (i.e. the correlation between the AC layer's standard deviation and the subgrade layer's standard deviation). These were found separately for each pavement structure (AC over granular base, AC over treated base, AC over subgrade, etc.) and FWD drop height. The results of these correlations are tabulated in Table 45. The correlation values show the Pearson product-moment correlation coefficients, which summarize the strength of the linear relationships between each pair of response variables (these are the coefficients noted in Table 45 under the column heading 'Correlation'). If there is an exact linear relationship between two variables then the correlation is 1 or -1 depending on whether the variables are positively or negatively related. If there is no linear relationship the correlation tends toward zero.

The results of the analysis between the AC surface layer standard deviation and the subgrade standard deviation (as a function of the pavement structure) showed weak absolute correlations that ranged from 0.0086 to 0.2985. Therefore, it *cannot* be assumed that there is a strong linear relation between the AC modulus variability and the subgrade variability (i.e. when the AC variability is high [or conversely, low] the subgrade variability may or may not be high [or conversely, low]).

Correlation between Surface Modulus and Surface Thickness Variability

Next, an attempt was made to correlate the thickness variability to the surface modulus variability. At this time, only one SPS-1 project and one SPS-8 project have back-calculated moduli and layer thickness data in the LTPP database. Therefore, the data set used for the thickness-modulus correlation analysis was small. The results showed a weak correlation of 0.239. It is apparent that further study of the correlation between the variability of AC surface modulus and the corresponding variability of AC thickness is needed. This is an extremely important consideration in the design and overall performance of asphalt pavements, and once sufficient LTPP data is available this analysis should be continued.

Comparisons of the Mean Standard Deviations for AC Surfaced Pavements

Comparisons of the mean elastic moduli standard deviations were also conducted among different factor groups. The investigated groups were environmental zone, layer type (specifically the AC layer, but also all of the other layers present below AC surfaced pavement), and season. The LTPP environmental zones (which are based on mean annual rainfall and mean annual freezing index) are classified as follows:

- Dry-No Freeze (D-NF)
- Dry-Freeze (D-F)
- Wet-No Freeze (W-NF)
- Wet-Freeze (W-F)

To determine if back-calculated variability is a function of the environmental zone where the pavement is located, a comparison of the mean standard deviation by environmental zone was conducted. The comparison was completed for all AC layers as well as for the other pavement layers present beneath AC surface pavements. The results of the statistical comparisons are shown in Table 46.

The results show that the average standard deviations of the AC back-calculated modulus, computed from each environmental zone, are significantly different from a statistical point of view (whenever the term "significantly different" is used, this is based on statistical considerations). The lowest standard deviation is in the dry no-freeze zone, and the highest in the wet freeze zone, as one would guess. Therefore, it can reasonably be concluded that the expected variability of the AC layer *does* depend upon the environmental region in which the pavement is constructed.

Conversely, the results showed that the environment ***does not*** have an effect on the expected variability in the back-calculated moduli of the subgrade. At an alpha level equal to 0.05, the mean standard deviations of the back-calculated subgrade moduli were not statistically significantly different among the environmental zones. Therefore, the LTPP data suggests that the expected variability of the back-calculated subgrade moduli is independent of the environmental zone in which they are located.

The next investigation included an analysis of the variability of each layer included in AC-surfaced pavements in relation to each other. In other words, is one layer significantly different from the others in terms of the expected back-calculated variability? The results of this analysis are summarized in Table 47. The means shown in Table 47 are all significantly different from each other at an alpha level equal to 0.05, except for those of the treated subgrade and granular subbase.

The results also show that the granular base layer had the smallest standard deviation for the back-calculated moduli. The results also showed that the AC surface had the highest mean standard deviation. Another comparison based on the coefficient of variation (or CV, which is the ratio of the standard deviation divided by the mean expressed as a percent) was completed. These results are also shown in Table 47. The analysis shows that AC surfaces have the lowest CV for the back-calculated modulus while granular and treated subbases have the highest CV.

Therefore, it can reasonably be concluded that the expected variability in modulus or stiffness of the layers typically constructed in AC-surfaced pavements will be different from one another. This lends credence to other studies that have been completed regarding this topic and basic engineering intuition. In addition, it can also be assumed that asphalt concrete layers as well as granular base layers will typically have lower back-calculated variability in relation to the other underlying layers. This may not always be the case, because there may be certain layer configurations and areas of the country where this supposition is not true, but the pooled LTPP data suggests that this is generally the case.

The final analysis conducted included comparing the mean standard deviations by season. The usual fall, spring, summer and winter seasons were used, with the seasons defined by the standard meteorological nomenclature (Fall = September-November, Winter = December-February, Spring = March-May, and Summer = June – August). The comparison was again divided by layer type, and the results are shown in Table 48.

Table 45. Correlations between the Variability of Moduli for AC and Subgrade

Drop Height	Surface Type[1]	Correlation
1	AC/AC	0.1438
1	AC/GB	0.0187
1	AC/GB/TS	-0.0390
1	AC/PC	-0.0707
1	AC/SS	0.1476
1	AC/TB	0.0400
2	AC/AC	0.0515
2	AC/GB	0.0513
2	AC/GB/TS	-0.1753
2	AC/PC	-0.0526
2	AC/SS	0.2985
2	AC/TB	0.0086
3	AC/AC	0.0538
3	AC/GB	0.0327
3	AC/GB/TS	-0.1630
3	AC/PC	-0.0639
3	AC/SS	0.0798
3	AC/TB	0.0537
4	AC/AC	0.0988
4	AC/GB	0.0295
4	AC/GB/TS	-0.1947
4	AC/PC	-0.0291
4	AC/SS	0.1990
4	AC/TB	0.0216

Note 1: AC-Asphalt Concrete, PC-Portland Cement, GB-Granular Base, GS-Granular Subbase, TB-Treated Base, TS-Treated Subgrade, SS-Subgrade

Table 46. Comparison of Mean SDs of Moduli for each Layer by Environmental Zone

Layer Type	Environments with a Statistically Significant Difference	D-F	D-NF	W-F	W-NF
AC	All	4249	2711	6670	5457
GB	D-F/D-NF, D-F/W-NF, W-F/W-NF	916	370	773	245
GS	D-F/W-NF	2909	3500	1892	1409
SS	None	1137	1368	1405	1013
TB	W-NF/D-F, D-NF/D-F	1748	3381	3246	3743
TS	None	0	435	1039	2585

Table 47. Comparison of the Mean Standard Deviation of Moduli by Layer Type

Layer Type	Mean Standard Deviation (Pa)	Mean Coefficient of Variation (%)
AC	5137	36
GB	578	42
GS	1922	63
SS	1211	46
TB	3275	49
TS	2114	67

Table 48. Comparison of the Mean Standard Deviation of Moduli by Season

Layer Type	Seasons With a Statistically Significant Difference	Fall (Pa)	Spring (Pa)	Summer (Pa)	Winter (Pa)
AC	All except fall and spring	5195	5354	3419	7713
GB	Winter/Spring, Winter/Fall, Winter/Summer	415	451	330	1611
GS	None	1434	2008	2047	2440
SS	Summer/Fall, Summer/Spring	1435	1363	822	1291
TB	Summer/Spring, Summer/Winter	3168	3897	2540	3803
TS	Spring/Fall	3085	537	2617	2189

The results of the analysis indicate that for the granular base, the expected variability of the back-calculated moduli was significantly different in the winter than the other seasons. In addition, the back-calculated modulus of the AC layer was ***most*** variable during the winter months. All other combinations for the AC layer were statistically different except for the Fall/Spring comparison, which in terms of engineering intuition makes sense. Other layers did not provide clear, conclusive patterns.

From this analysis, it has been demonstrated that the expected variability associated with the AC layer's back-calculated modulus is primarily dependent upon the season in which the FWD measurements are taken. However, one may expect the AC variability to be nearly the same in the fall and spring, which makes logical sense because many environments have similar weather patterns during these two seasons. Most other layers constructed as the underlying foundation for AC-surfaced pavements do not have any discernable patterns when it comes to the expected variability of the modulus, with the exception of the granular base which seems to have significantly more variability during the Winter season.

Long-Term, Temperature-Dependent and Spatial Moduli Variations for AC Surfaced Pavements

Although this analysis was not possible to conduct during Phase I of this project due to lack of data at the time, the research team has now been able to briefly address this subject, to document the supposition that the expected variability in back-calculated modulus, for all pavement layers, may not stay the same over the performance life of a pavement. With time and traffic, a flexible pavement's "character" may change from a somewhat homogeneous structure to one with anomalies like longitudinal and transverse cracks, variations in structure due to patches, localized segregation, stripping and the degradation of layers due to water infiltration and pumping, etc. With each of these pavement anomalies, the expected variation in the back-calculated moduli will change.

One may guess that the expected variability in pavement layers will increase with time. In addition, the expected variability will also be a function of the season in which FWD measurements are undertaken. This was verified to some degree in the previous analysis, but without taking the age of the pavement into account. Therefore, the following (somewhat limited) analysis was conducted to help substantiate these suppositions.

Two typical GPS and two typical SPS pavement sections were selected from the suite of sections included in the LTPP Seasonal Monitoring Program (SMP) sections to study the long-term changed and the changes in spatial variability of back-calculated moduli. The criteria used to select the pavements were the age of the pavement (relatively new and old), the frequency and monitoring history of the deflection testing (as much as possible), and the region in which the pavements were placed (wide range of temperatures encountered throughout a typical year). Two of these pavements were SPS-1 sections from Arizona (04-0113 and 04-0114) that were recently constructed, while the other two sites were older GPS-1 sections from Minnesota (27-1028) and Montana (30-8129), respectively. The two Arizona sections were opened to traffic in 1993, the Minnesota section in 1988 and the Montana site in 1972.

Upper Subgrade Characterization for AC Surfaced Pavements

It should be noted that for each of the four pavement sections selected for analysis (with the exception of Section 27-1028), an upper subgrade layer has been back-calculated along with the moduli of other typically constructed pavement layers. Upper subgrade layers were added in an effort to more accurately model the subgrade stratification that is typically found under pavements and thus reduce the overall error associated with the back-calculation.[5] However, many times these layers indicate stiffness values well in excess of something that is reasonable, but at the same time the overall error term for the basin fitting may be very low. In these cases, a judgment is usually made regarding the reasonableness of the moduli assigned to this layer. If for some reason the modulus is considered to be in error or unreasonable, *all* of the pavement layer moduli are removed from the LTPP section characterization.

However, when there was no justifiable reason for removing the results, all of the layer moduli predicted for the basin were retained as part of the section characterization. This results in a wider spread in the back-calculated moduli of the upper subgrade layer, which in turn affects the expected coefficient of variation. This discussion is included to help explain the reasoning behind having the upper subgrade included in the back-calculation and to help the reader understand the difficulties that may be encountered when analyzing such data.

Evaluation of Long-Term Changes in Back-Calculated Modulus Variability

The first part of the analysis consisted of reviewing the changes in layer moduli, standard deviation and coefficient of variation (by drop height) over time for each of the pavement sections. Plots and tables were developed to complete this exercise. The tables are shown as Table 49, Table 50, and Table 51, while the plots are included as Figures C6 through C21 in Appendix C.

From these plots and tables, the following tentative conclusions may be drawn:

- For the AC layer, the coefficient of variation (averaged over all FWD measurements) is significantly different (at an alpha level = 0.05) between the newer pavements (04-0113 and 04-0114) and the older pavements (27-1028 and 30-8129).
- The granular base CV for section 27-1028 is statistically different from the other CV values but there was not a clear pattern between the 'old' and 'new' pavements.
- The upper subgrade variation is considerably different than the variation encountered for the lower subgrade. This was partially explained in the foregoing discussion.
- The subgrade CV for section 27-1028 is significantly different from the other CV values, but there was not a clear pattern between the 'old' and 'new' pavements.
- There does not seem to be a difference in variation based upon the FWD drop height (or imparted load). There are clear indications of stiffness non-linearity within the unbound materials (see Figures C13, C15 and C17) but this does not seem to have an effect on the expected variations in modulus.

- Although this analysis has been insightful, it is still somewhat limited. Therefore, the research team recommends that further study be continued on the long-term variation of moduli in AC-surface pavements as more data becomes readily available in the IMS.

Analysis of the CV of Back-Calculated Moduli for Use in the Design of AC-Surface Pavements

The analyses up to this point have primarily focused on evaluating the standard deviation of the back-calculated moduli based upon a series of FWD measurements taken along a pavement section. The objective of these analyses was to identify areas where the expected variability of the back-calculated pavement layer moduli that constitute typical AC-surfaced pavements may be influenced by factors other than the material makeup (i.e., the environmental zone, structural configuration, FWD load parameters etc.). However, a pavement designer must be able to utilize the information contained in these analyses in a manner suitable for the design of pavements. Generally, variability is quantified in design systems through the use of the coefficient of variation, or CV. The purpose of this section is to provide recommended ranges of the CV for back-calculated pavement layers found in AC-surfaced pavements for use in design.

The spatial variations (expressed by CV, for example) in back-calculation results are affected by a variety of factors, as previously discussed. One of these factors is the pavement surface course temperature at the time of FWD testing. The effect of pavement temperature was not part of this research, because only long-term variations in pavement design parameters are within the scope of work. Nevertheless, it is important to realize that the results of back-calculation are affected by AC pavement temperature. A brief introduction to this issue, and the moduli calculation results in terms of the variability that can be expected as a function of pavement temperature, is presented in Appendix E.

At a wide variety of test temperatures (with the moduli unadjusted for test temperature), Table 53 shows that most CV's for back-calculated AC layers are between 10 and 70%. The mean CV value of the pooled LTPP back-calculated modulus value for AC layers is ±36.7%, with a lower and upper 95% confidence interval of 35.8 and 37.6% respectively. In addition, the LTPP data demonstrates that 90% of the CV's calculated fall at or below a CV of 68.3%, while approximately 10% of the CVs are at 9.9% or less. Therefore, for purposes of pavement design, it is recommended that the CV for back-calculated AC layers is ~37% on average, with a maximum assigned value of 68% in extreme instances or if a very conservative design approach is needed. This CV may be applied to all AC moduli back-calculated from the deflection basins generated at the various drop heights or FWD test loads. Table 54 verifies that the expected CV at the four FWD drop heights used by LTPP are not significantly different, and therefore it may be assumed that the expected CV will not change with drop height.

Summary of Back-Calculated Moduli for Asphalt Surfaced Pavements

Similar analyses were completed for other pavement layers typically found in AC-surfaced pavements. A summary of the above data and conclusions on back-calculated moduli is presented in *Section 4.3 – Summary of Asphalt Concrete Parameter Variability.*

Table 49. AC Layer Statistics for Seasonal Sections 04-0113, 04-0114, 27-1028 & 30-8129

HMAC	*040113 Layer 1 (HMAC)*			*040114 Layer 1 (HMAC)*			*271028 Layer 1 (HMAC)*		
	Avg.	*Std. Dev.*	*CV*	*Avg.*	*Std. Dev.*	*CV*	*Avg.*	*Std. Dev.*	*CV*
Mean (MPa)	7,442.9	1,921.6	26.4	6,330.2	1,182.3	20.4	63,127.3	45,495.4	75.8
Standard Error (MPa)	409.3	107.5	0.8	353.9	53.9	0.9	4,511.6	3,035.3	2.0
Standard Deviation (MPa)	3,170.6	832.9	6.3	2,552.2	388.6	6.8	44,204.3	29,739.3	19.6
Kurtosis	-1.347	-1.158	-0.717	-1.217	-1.275	0.595	3.791	1.872	0.189
Skewness	0.049	0.092	0.202	-0.015	0.306	1.138	1.873	1.561	0.704
Range (MPa)	9,602.0	2,819.4	26.3	8,307.1	1,164.0	25.4	239,307.1	136,933.8	96.8
Minimum(MPa)	3,030.5	454.5	15.0	2,431.0	658.4	11.3	9,995.9	6,507.2	43.5
Maximum(MPa)	12,632.5	3,273.9	41.3	10,738.0	1,822.4	36.7	249,303.0	143,441.0	140.3
Count	60	60	60	52	52	52	96	96	96

HMAC (con't)	*271028 Layer 2 (HMAC Base)*			*308129 Layer 1 (HMAC)*		
	Avg.	*Std. Dev.*	*CV*	*Avg.*	*Std. Dev.*	*CV*
Mean (MPa)	14,982.8	11,701.1	81.9	31,342.9	19,461.1	46.1
Standard Error (MPa)	2,465.4	1,899.7	4.1	5,340.2	3,390.3	2.8
Standard Deviation (MPa)	24,156.4	18,613.0	40.3	58,498.7	37,139.0	30.7
Kurtosis	10.382	11.736	8.364	10.786	3.198	2.352
Skewness	3.338	3.353	2.154	3.203	2.166	1.787
Range (MPa)	120,182.5	107,262.6	266.9	341,367.2	134,866.0	152.5
Minimum(MPa)	1,390.5	1,136.4	33.2	2,632.8	0.0	0.0
Maximum(MPa)	121,573.0	108,399.0	300.1	344,000.0	134,866.0	152.5
Count	96	96	96	120	120	120

Table 50. Base Statistics for Seasonal Sections 04-0113, 04-0114, 27-1028 & 30-8129

Granular Base	*040113 Layer 2 (Gran Base)*			*040114 Layer 2 (Gran Base)*		
	Avg.	*Std. Dev.*	*CV*	*Avg.*	*Std. Dev.*	*CV*
Mean (MPa)	131.1	52.2	42.4	246.8	79.8	33.2
Standard Error (MPa)	6.2	1.8	1.4	7.2	2.3	1.1
Standard Deviation (MPa)	47.8	14.1	10.8	51.7	16.4	8.0
Kurtosis	2.024	0.882	-0.929	0.254	-0.616	1.512
Skewness	1.224	1.255	0.162	0.696	0.036	0.519
Range (MPa)	236.1	60.6	41.5	229.6	74.0	42.5
Minimum (MPa)	65.6	35.2	21.8	164.2	43.6	14.4
Maximum (MPa)	301.7	95.9	63.3	393.8	117.6	56.9
Count	60	60	60	52	52	52

Granular Base (con't)	*271028 Layer 3 (Gran Base)*			*308129 Layer 2 (Gran Base)*		
	Avg.	*Std. Dev.*	*CV*	*Avg.*	*Std. Dev.*	*CV*
Mean (MPa)	2,916.5	9,398.6	136.9	1,648.3	797.4	31.5
Standard Error (MPa)	661.7	2,349.6	22.8	327.6	158.3	1.5
Standard Deviation (MPa)	6,483.3	23,021.2	223.3	3,588.9	1,734.6	16.9
Kurtosis	11.953	6.887	4.165	8.237	5.669	0.653
Skewness	3.418	2.755	2.242	2.799	2.452	1.231
Range (MPa)	34,479.9	101,068.2	937.4	18,409.1	8,329.9	74.4
Minimum (MPa)	125.7	27.8	15.2	78.0	18.4	12.0
Maximum (MPa)	34,605.6	101,096.0	952.7	18,487.1	8,348.3	86.4
Count	96	96	96	120	120	120

Table 51. Upper Subgrade Stats for Seasonal Sections 04-0113, 04-0114, 27-1028 & 30-8129

Upper Subgrade	*040113 Layer 3 (Upper Subgrade)*			*040114 Layer 3 (Upper Subgrade)*		
	Avg.	*Std. Dev.*	*CV*	*Avg.*	*Std. Dev.*	*CV*
Mean (MPa)	394.5	309.4	71.1	2,125.4	10,003.8	203.3
Standard Error (MPa)	25.8	38.1	3.2	493.7	2,672.2	34.9
Standard Deviation (MPa)	199.7	294.8	25.1	3,560.5	19,269.2	251.5
Kurtosis	4.602	7.387	6.418	7.902	3.025	-0.055
Skewness	2.100	2.821	2.570	2.781	1.997	1.193
Range (MPa)	980.8	1,284.1	121.5	15,948.8	73,077.8	793.0
Minimum (MPa)	210.2	131.1	51.3	300.7	55.0	17.9
Maximum (MPa)	1,191.0	1,415.2	172.7	16,249.5	73,132.8	810.9
Count	60	60	60	52	52	52

Upper Subgrade (con't)	*308129 Layer 3 (Upper Subgrade)*		
	Avg.	*Std. Dev.*	*CV*
Mean (MPa)	2,209.4	10,689.4	185.7
Standard Error (MPa)	370.1	1,976.2	23.9
Standard Deviation (MPa)	4,054.1	21,648.6	261.6
Kurtosis	3.631	1.871	0.101
Skewness	2.169	1.861	1.316
Range (MPa)	15,682.9	72,169.5	860.6
Minimum (MPa)	73.1	8.9	12.0
Maximum (MPa)	15,756.0	72,178.4	872.7
Count	120	120	120

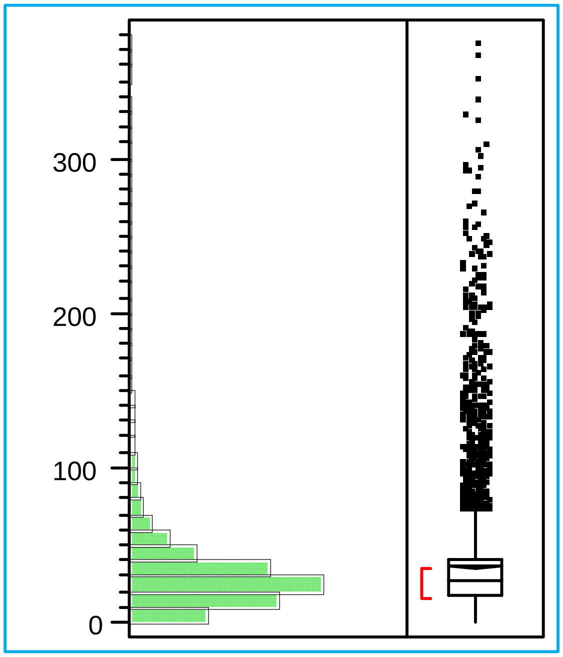
Table 52. Subgrade Statistics for Seasonal Sections 04-0113, 04-0114, 27-1028 & 30-8129

Subgrade	*040113 Layer 4 (Subgrade)*			*040114 Layer 4 (Subgrade)*		
	Avg.	*Std. Dev.*	*CV*	*Avg.*	*Std. Dev.*	*CV*
Mean (MPa)	238.6	98.1	41.9	421.7	136.0	32.9
Standard Error (MPa)	5.4	1.3	0.7	12.6	4.4	0.9
Standard Deviation (MPa)	41.7	10.1	5.6	91.2	32.1	6.6
Kurtosis	-0.877	-0.460	-0.201	0.088	4.026	2.186
Skewness	-0.132	-0.248	0.644	-0.166	1.298	0.841
Range (MPa)	160.9	45.7	23.4	389.8	176.9	34.7
Minimum (MPa)	160.5	73.6	33.4	248.1	90.5	20.5
Maximum (MPa)	321.4	119.3	56.8	637.9	267.4	55.1
Count	60	60	60	52	52	52

Subgrade	*271028 Layer 4 (Subgrade)*			*308129 Layer 4 (Subgrade)*		
	Avg.	*Std. Dev.*	*CV*	*Avg.*	*Std. Dev.*	*CV*
Mean (MPa)	325.0	335.2	61.5	108.3	86.4	34.5
Standard Error (MPa)	26.0	77.8	8.8	12.3	21.7	4.9
Standard Deviation (MPa)	254.7	762.6	86.2	134.6	237.4	54.0
Kurtosis	21.827	12.008	1.952	16.802	8.934	7.798
Skewness	4.118	3.309	1.863	3.971	3.185	2.979
Range (MPa)	1,849.5	4,524.9	309.4	815.7	1,073.7	266.0
Minimum (MPa)	168.3	11.2	5.2	34.4	5.4	8.3
Maximum (MPa)	2,017.8	4,536.1	314.6	850.1	1,079.1	274.3
Count	96	96	96	120	120	120

Table 53. Distribution of the CV for Back-Calculated AC Layers

ELASTIC_MODULUS_COV



Quantiles

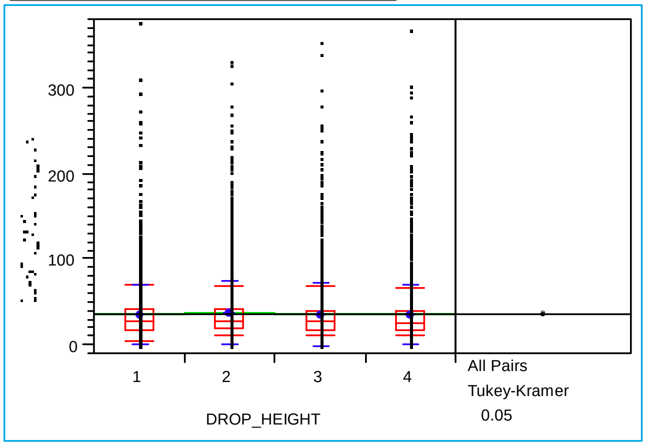
maximum	100.0%	377.52
	99.5%	244.39
	97.5%	144.27
	90.0%	68.33
quartile	75.0%	41.32
median	50.0%	27.86
quartile	25.0%	18.29
	10.0%	9.93
	2.5%	0.00
	0.5%	0.00
minimum	0.0%	0.00

Moments

Mean	36.727
Std Dev	36.880
Std Error Mean	0.453
Upper 95% Mean	37.616
Lower 95% Mean	35.838
N	6614.000
Sum Weights	6614.000

Table 54. Comparison of AC Layer CV's for All LTPP Sections at Each FWD Drop Height

ELASTIC_MODULUS_COV By DROP_HEIGHT



Quantiles

Level	minimum	10.0%	25.0%	median	75.0%	90.0%	maximum
1	0	5.499628	17.45878	28.30076	42.51811	70.11446	377.5227
2	0	11.08968	18.79433	28.66612	41.93567	68.15347	331.294
3	0	10.99918	18.38819	27.42777	40.51797	69.00881	355.0504
4	0	10.88567	18.55005	26.9784	39.89527	66.19775	369.0101

Oneway Anova

Summary of Fit

RSquare	0.00031
RSquare Adj	-0.00014
Root Mean Square Error	36.88305
Mean of Response	36.72708
Observations (or Sum Wgts)	6614

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	3	2787.3	929.11	0.6830
Error	6610	8991973.9	1360.36	Prob>F
C Total	6613	8994761.2	1360.16	0.5623

Means for Oneway Anova

Level	Number	Mean	Std Error
1	1701	36.1365	0.89428
2	1632	37.6735	0.91299
3	1644	37.0044	0.90965
4	1637	36.1187	0.91160

Std Error uses a pooled estimate of error variance

Means Comparisons

Dif=Mean[i]-Mean[j]	2	3	1	4
2	0.00000	0.66905	1.53693	1.55481
3	-0.66905	0.00000	0.86788	0.88577
1	-1.53693	-0.86788	0.00000	0.01788
4	-1.55481	-0.88577	-0.01788	0.00000

Alpha= 0.05

Comparisons for all pairs using Tukey-Kramer HSD

q*
2.56969

Abs(Dif)-LSD	2	3	1	4
2	-3.31789	-2.64278	-1.74714	-1.76054
3	-2.64278	-3.30576	-2.41006	-2.42352
1	-1.74714	-2.41006	-3.24990	-3.26362
4	-1.76054	-2.42352	-3.26362	-3.31282

Positive values show pairs of means that are significantly different.

2.3.2 AC Modulus Variability Based on Laboratory Tests

Laboratory Determined Moduli for AC Layers

The resilient modulus (M_r) of Hot Mix Asphalt (or AC) is a key design input parameter. It is used for predicting the response of the pavement under load, which in turn may be used to predict fatigue cracking. Other design parameters include resistance to fatigue and thermal cracking, and the resistance to rutting. Available data in the LTPP program includes the results of the P-07 LTPP test protocol, M_r and Creep Compliance data.

Similar to the analysis completed on the back-calculated moduli, the laboratory moduli data was evaluated in terms of expected variability for use in pavement design. However, one important note that needs to be made is related to storage of the AC samples cored from LTPP sections and the potential for erroneous test results. In many cases, the amount of time the cores were stored prior to testing was often several years. It would be good to conduct a short but controlled study where a number of cores are taken from a pavement section. One set of cores should be packed in dry ice immediately after extracting and stored in a frozen state until testing. The testing on this set should be conducted as soon as possible. The other sets should be cored, shipped and stored exactly the same as the LTPP cores used for this study (and other studies). These sets should be tested at one week, one month, six months, one year, two years and four years after sampling to determine what effect, if any, storage has on the test results.

Laboratory Test Data Available

The data available included 453 records from 111 sections. The P-07 test protocol calls for measurement at three temperatures: 5, 25 and 40 degrees Celsius. Out of the 453 records, 357 records contained results for all three temperatures, from 119 core samples. Out of these 119 samples, there were 26 paired sample sets. It is our understanding that the pairs represent sample sets from *both* ends of the 500-foot test section, which means that there is approximately 155 to 160 meters between the sample sets. This provides some opportunity to evaluate spatial variation. The between-sample precision of the test is not known at this time, so we cannot separate spatial variation from precision.

Laboratory Test Description

The test method is based on work done by Professor Reynaldo Roque and his colleagues while he was at Penn State University. A report describing the test can be found in a later publication by Roque.(6) As the title of this publication suggests, the stated purpose of the referenced study was to evaluate how the SHRP Indirect Tension Tester can be used to predict, and subsequently control, fatigue cracking.

The P-07 test method is based on the common indirect tension-loading concept. It uses diametral loading of core and laboratory samples, much as previous test methods have done (and still do). The primary difference is that strain response is measured on the horizontal and vertical axes by attaching measurement devices on the face of the sample. The gauge point spacing is 25 mm for 100-mm diameter samples and 37.5 mm for 150-mm diameter samples. The gauge points are centered on the face of the sample. The geometry of the test setup provides strain measurements from the center one-fourth of the sample, thereby avoiding measurements that are influenced by plastic deformations at the loading strips and problems with external measurements. Poisson's ratio calculations result in much more realistic values than did previous methods of modulus testing in the laboratory. A single test for resilient modulus requires the use of three samples, or six test "faces". Six faces are used because of the random influence of the coarse aggregate. To reduce the risk of having a large aggregate within the gauge zone influencing the results, the high and low strain values are not used in the calculations of moduli.

There are three types of test results from the P-07 test. These are:

- Resilient modulus
- Creep Compliance
- Indirect Tensile Strength

The results of these three tests may be used to predict AC performance. Knowledge of the variation of these performance prediction properties allows pavement design engineers to select design properties, with the expectation that the material will provide satisfactory performance over the design life of the pavement with some level of confidence or reliability.

All of the test measurements are made in an indirect tensile mode. Either pavement cores or laboratory prepared samples are trimmed to an appropriate thickness and diametrically loaded.

<u>Resilient Modulus Tests</u>

General Description of M_r Results

The distribution of all M_r results is shown in Figure 11 for each of the three test temperatures. Note that the abscissa is on a logarithmic scale. The results tend to be close to a log-normal distribution for the 25°C data and skewed to the right for the 40°C results and to the left for the 5°C results.

Although the information provided in Figure 11 is not directly useable for design purposes, it does serve a purpose of defining the range of values that we can expect to see from the P-07 test. Table 55 raises an issue as to how the M_r data are transformed for analysis of the coefficient of variation, or CV. The M_r data is in GPa in the database. Since the data is distributed in a log-normal fashion, the M_r data should be transformed to MPa before evaluating the variation of the log-transformed results. The Log of both the GPa and MPa M_r results are provided here only as a reminder that the units used are important when considering the variation of test value that are transformed. As can be seen in the Log (GPa) case for 40°C results, the Log of the mean is very close to 0 but the standard deviation for both the Log (MPa) and Log (GPa) are the same. The CV for the data must be applied to like units. Although the variability of the data is clearly the same, regardless of the units, when this variability is expressed in CV, the units become important.

Within the 453 available P-07 test results, there were sample pairs and results from multiple sections from SPS sites that are useful in defining the variation from the same mixes sampled in the field. All of these samples were obtained from cores, i.e., not from laboratory-compacted mixes.

Within Mix Variations: Within Each SPS-1 Site

Three SPS-1 sites had samples and layer thickness information from more than one section, allowing us to associate the test results with a specific mix and/or layer. These sites are in Oklahoma, Michigan and New Mexico. A summary of the P-07 M_r results for these three sites are shown in Table 56. An SPS-5 site in Manitoba also contained sufficient P-07 tests results to consider within-site variations, but layering data was not available that allowed us to associate the test results with a specific layer or mix used in the SPS sections at this site.

Oklahoma Site

The SPS-1 site in Oklahoma (State 40) has several test results from the same material within the site, but not in the same section. Generally, the SPS-1 sites were designed with one mix used for the top 100 mm and another mix used for the asphalt treated base. [The asphalt treated base is still a designed mix produced in a hot mix plant and placed and compacted as conventional hot mix. The terminology used within LTPP experiment could imply that the asphalt-stabilized base is a stabilized base rather than an HMA base, however this is not the case.] There were four wearing course samples that were tested. The CV of the resilient modulus (in arithmetic GPa) for the four tests is 15.5, 20.4 and 27.1 percent for 5, 25 and 40°C, respectively. The non-wear asphalt and the asphalt treated base have even lower CV values, as shown in Table 56.

Figure 12 shows the M_r results for the Oklahoma SPS-1 wearing course samples from four different sections. The larger CV values for the 5° and 25°C tests are due to lower stiffnesses for the sample from Section 40-1020.

Figure 13 shows the variation in terms of the average 5°C M_r and a plus and minus one standard deviation range for the wearing course from three SPS-1 sites to the variation for all of the P-07 results in terms of the arithmetic GPa units in 3a and Log (MPa) units in 3b.

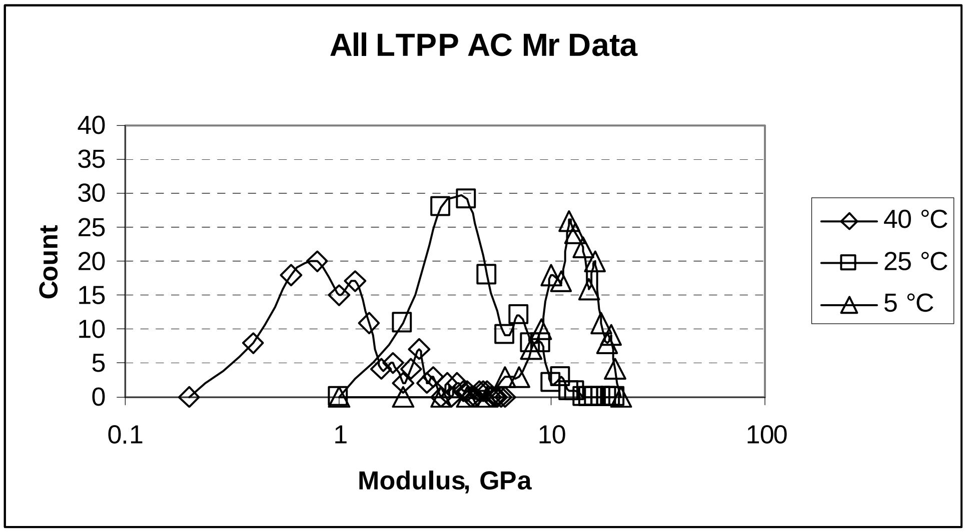


Figure 11. Hot Mix P-07 Moduli Distributions by Temperature

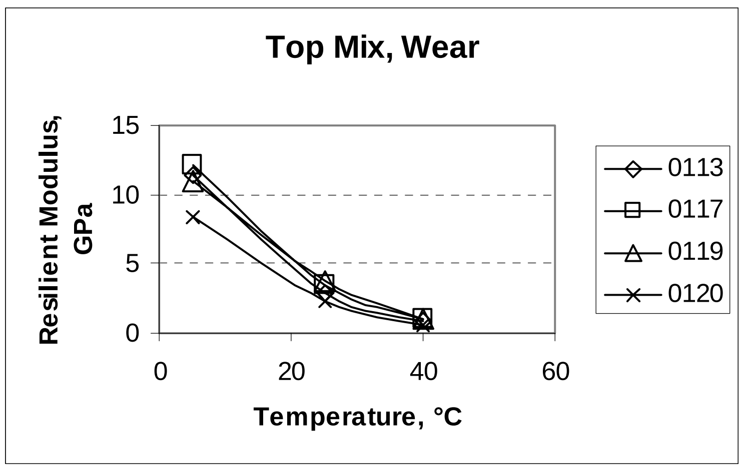
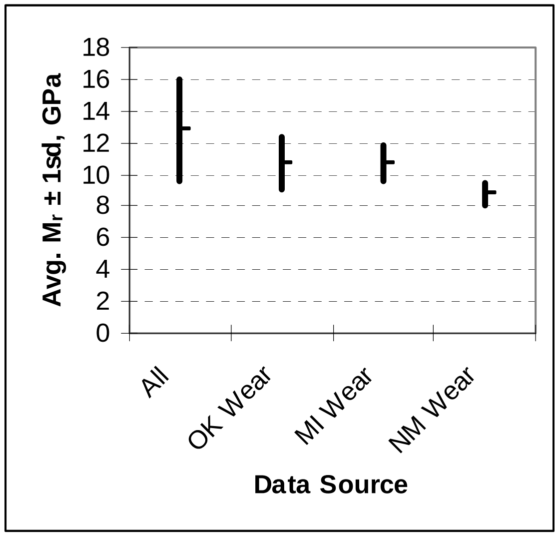


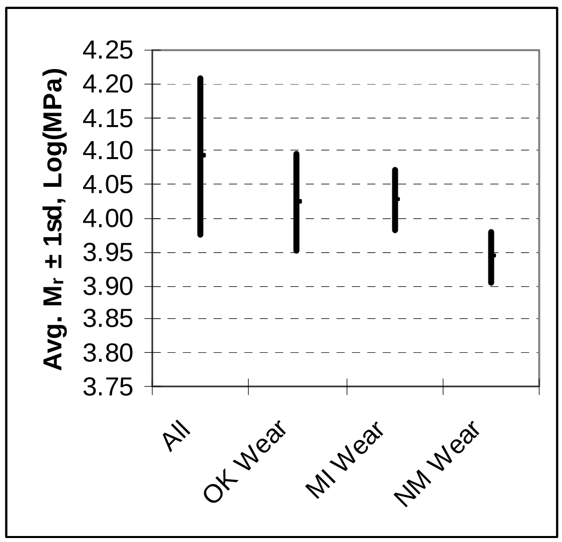
Figure 12. Oklahoma SPS-1 Wearing Course M_r Results

Table 55. Asphalt M_r Data Variation for All Available P-07 Data

	Arithmetic, GPa		
	5 °C	25 °C	40 °C
Average	12.78	4.65	1.31
Std. Dev.	3.19	2.44	0.96
CV	24.9%	52.5%	73.0%
	Log (MPa)		
Average	4.09	3.61	3.02
Std. Dev.	0.12	0.22	0.28
CV	2.8%	6.2%	9.4%
	Log (GPa)		
Average	1.09	0.61	0.02
Std. Dev.	0.12	0.22	0.28
CV	10.6%	36.8%	1281.4%



(a)



(b)

Figure 13. SPS-1 Wearing Course Variations

Table 56. Summary Data for All Sections from Three SPS-1 Sites

All P-07 Tests		Resilient Modulus, GPa			Resilient Modulus, Log (MPa)		
		5 °C	25 °C	40 °C	5 °C	25 °C	40 °C
All Layers	Average	12.78	4.65	1.31	4.09	3.61	3.02
	Std. Dev.	3.19	2.44	0.96	0.12	0.22	0.28
	CV	24.9%	52.5%	73.0%	2.8%	6.2%	9.4%
Oklahoma SPS-1		Resilient Modulus, GPa			Resilient Modulus, Log (MPa)		
		5 °C	25 °C	40 °C	5 °C	25 °C	40 °C
Wearing	Average	10.69	3.12	0.85	4.02	3.49	2.91
	Std. Dev.	1.65	0.64	0.23	0.07	0.09	0.14
	CV	15.5%	20.4%	27.1%	1.8%	2.7%	4.7%
Non-Wearing	Average	12.65	4.19	1.17	4.10	3.61	3.05
	Std. Dev.	1.77	0.95	0.38	0.06	0.10	0.14
	CV	14.0%	22.7%	32.6%	1.5%	2.9%	4.7%
Asphalt Treated Base	Average	14.81	5.03	1.45	4.17	3.70	3.16
	Std. Dev.	0.98	0.76	0.25	0.03	0.06	0.08
	CV	6.6%	15.2%	17.2%	0.7%	1.7%	2.5%
Michigan SPS-1		Resilient Modulus, GPa			Resilient Modulus, Log (MPa)		
		5 °C	25 °C	40 °C	5 °C	25 °C	40 °C
Wearing	Average	10.67	1.91	0.50	4.03	3.27	2.68
	Std. Dev.	1.13	0.52	0.18	0.04	0.11	0.15
	CV	10.6%	27.1%	36.6%	1.1%	3.4%	5.7%
Non-Wearing	Average	15.32	3.61	0.75	4.18	3.56	2.87
	Std. Dev.	1.20	0.43	0.18	0.03	0.05	0.10
	CV	7.8%	12.0%	23.7%	0.8%	1.5%	3.6%
Asphalt Treated Base	Average	14.78	3.82	1.09	4.17	3.58	3.04
	Std. Dev.	2.24	0.31	0.15	0.06	0.04	0.06
	CV	15.1%	8.1%	14.0%	1.5%	1.0%	2.1%
New Mexico SPS-1		Resilient Modulus, GPa			Resilient Modulus, Log (MPa)		
		5 °C	25 °C	40 °C	5 °C	25 °C	40 °C
Wearing	Average	8.78	2.31	0.53	3.94	3.36	2.72
	Std. Dev.	0.72	0.33	0.10	0.04	0.07	0.08
	CV	8.2%	14.5%	19.5%	0.9%	2.0%	3.0%
Non-Wearing	Average						
	Std. Dev.						
	CV						
Asphalt Treated Base	Average	8.89	2.48	0.60	3.95	3.39	2.77
	Std. Dev.	1.10	0.33	0.13	0.05	0.06	0.10
	CV	12.4%	13.4%	22.6%	1.4%	1.7%	3.6%

Within Mix Variation: Paired Tests

The variations from the paired samples are much smaller than the variation for the entire data set and generally smaller than the SPS-1 sites. Please note however that the paired tests may not have sufficient spatial separation to define the variation that may result in larger projects that have hot mix placed over several days and miles of pavement. The average variation of the arithmetic values of the 26 pairs is 7.1, 10.6 and 14.9 percent for the 5, 25 and 40°C results, respectively. The average variation of the Log (MPa) results are: 0.8, 1.3 and 2.1 percent for 5, 25 and 40°C, respectively, although these results may be misleading since they reflect logarithmic transformed variations, not actual variations in terms of modulus of elasticity. Thus the paired results may be more a reflection of within-laboratory repeatability of the P-07 M_r test rather than true site variability.

P-07 Creep Compliance

Creep compliance results are provided at -10, 5 and 25°C a total of seven times, between 1 and 100 seconds. Unlike the resilient modulus test results, there is no equivalent field method to test for creep compliance or to develop a master curve.

The data was processed through a lengthy procedure described in Roque's report[6] to provide an "m"-value (or slope) for the asphalt mix. Creep compliance and the m-value help define the mix properties that are useful to predict low temperature cracking, and they may be useful in predicting the development of fatigue cracking. The form of the power model used to described the master curve developed from the creep compliance data is:

$$D(\xi) = D_0 + D_1\,\xi^m$$

where:

D0, D1 and m = Model Coefficients, where "m" is the slope of the linear part of the log-log curve.

The process to calculate the coefficients for each data set is too lengthy to show here. Therefore, the variations in creep compliance are examined at specific temperatures and times. Figure 14 shows the creep compliance for a typical SPS-9 section. The data shown as Series 4 and 5 are the 5 and 25 °C data, respectively, shifted to the right by adding a constant value to the Log(time) values. In this case, based on a manual/visual selection of constants, 1.95 was added to the log(time) for the 5 °C data and 4.1 were added to the log(time) for the 25 °C data.

The data is re-configured in arithmetic fashion based on the time shift to create a new set of data that provides the master curve as shown in Figure 15. A visual selection of the D_0 value provided the following master curve formula.

$$D(\xi) = 0.073 + 0.0189\,\xi^{0.4526}$$

The m-value in this case of 0.4526 is typical of a hot-mix asphalt.

There were 29 paired data sets that had creep compliance data for all three temperatures. Table 57 contains the average, standard deviation, and coefficient of variation of the creep compliance data for all of the 58 test results (i.e., 29 pairs) for each of the test temperatures.

The within-pair variation was significantly lower, as it was for the resilient modulus. Table 58 shows the average within-pair variation for the three test temperatures. The variations within each pair are less than the overall data set, but the difference is not as great as it is for the resilient modulus variations. The impact of the variation on the m-value of a master curve, or the ability to predict performance, has not been investigated here since this is beyond the scope of the research.

Table 57. Creep Compliance Variation Statistics for 58 Samples

		Creep Compliance (1/GPa)							Poisson's Ratio, μ
		1 Sec.	2 Sec.	5 Sec.	10 Sec.	20 Sec.	50 Sec.	100 Sec.	
-10 °C	Avg.	0.049	0.060	0.070	0.078	0.088	0.106	0.127	0.326
	Std.Dev.	0.020	0.020	0.024	0.029	0.035	0.046	0.058	0.084
	CV	39.8%	32.7%	34.8%	37.1%	39.8%	43.1%	45.6%	25.7%
5 °C	Avg.	0.121	0.148	0.195	0.239	0.303	0.422	0.557	0.353
	Std.Dev.	0.066	0.084	0.122	0.158	0.213	0.316	0.433	0.101
	CV	54.4%	56.9%	62.7%	66.0%	70.3%	74.9%	77.7%	28.6%
25 °C	Avg.	0.756	1.055	1.643	2.276	3.150	4.808	6.655	0.377
	Std.Dev.	0.563	0.818	1.303	1.830	2.581	3.982	5.610	0.126
	CV	74.4%	77.5%	79.3%	80.4%	81.9%	82.8%	84.3%	33.4%

Table 58. Average Pair Coefficients of Variation for Creep Compliance

	Creep Compliance (1/GPa)							Poisson's Ratio, μ
	1 Sec.	2 Sec.	5 Sec.	10 Sec.	20 Sec.	50 Sec.	100 Sec.	
-10 °C	10.4%	9.6%	9.2%	8.8%	8.6%	8.8%	9.0%	15.3%
5 °C	8.0%	8.5%	9.5%	9.6%	9.8%	10.8%	11.7%	18.4%
25 °C	12.6%	13.2%	14.1%	15.5%	16.6%	18.3%	19.0%	22.7%

Table 59. Indirect Tensile Strength Summary

		Avg. Strength MPa	Poisson's Ratio		Tangent Modulus MPa
			μ Calc.	μ Used	
57 Tests	Avg.	0.905	0.362	0.345	0.907
	Std	0.277	0.144	0.118	0.537
	CV	30.6%	39.6%	34.0%	59.2%
6 Pair	Avg.	0.918	0.34	0.331	0.883
	Std	0.264	0.148	0.132	0.495
	CV	28.7%	43.4%	39.9%	56.1%
Within Pair	Avg.	10.1%	11.8%	9.5%	4.0%

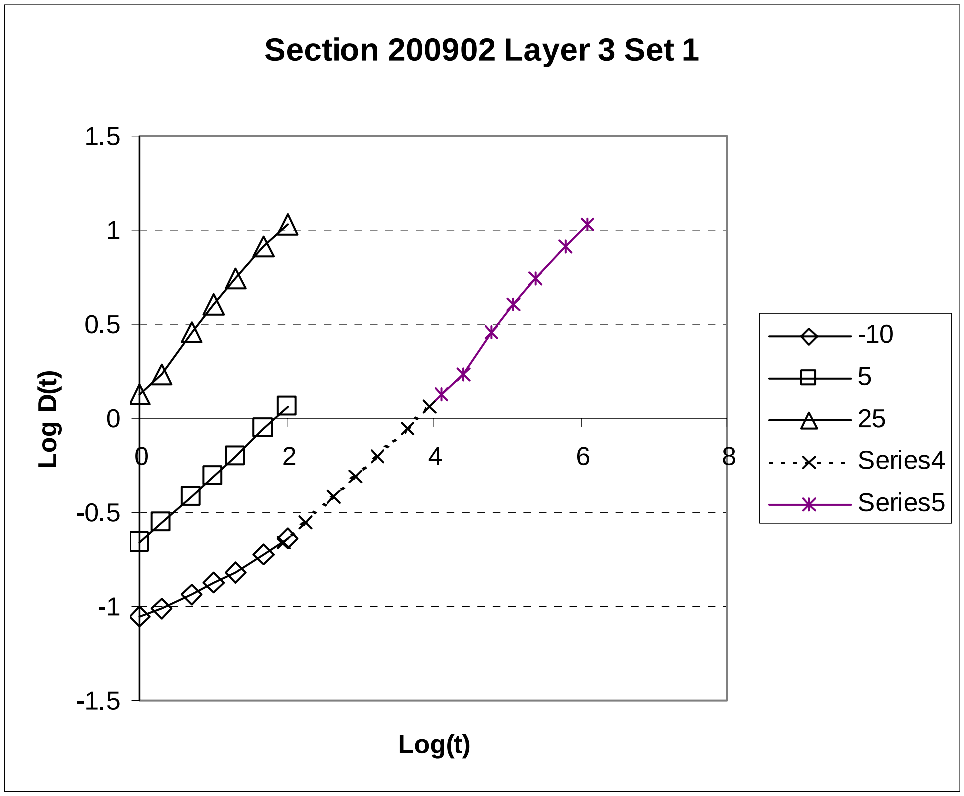


Figure 14. Example of the Development of a Master Curve

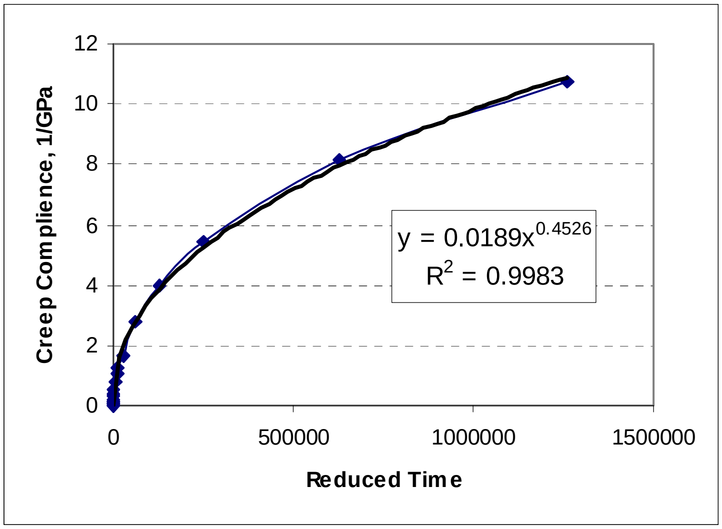


Figure 15. Power (Master) Curve from Data in Previous Figure

Indirect Tensile Strength

The last test result from the P-07 test is the indirect tensile strength. In this test, the sample is diametrically loaded at a constant strain rate of 12.5 mm/minute until tensile failure occurs. The test temperature is 25°C only. There were 57 tests available in the data at the time of analysis. Of those 57 tests, 12 were paired samples, for a total of six pairs. Table 59 summarizes the variation in the indirect tensile results for the total sample set, the total paired set, and the average within-pair variation in the last row. The Poisson's ratios shown are not part of the indirect tensile measurement, but are included here from the creep compliance where they are calculated, since they are necessary for calculating the strength and tangent modulus values.

The within-pair variations for the six pairs evaluated show much lower variations than the overall sample does. This is similar to the results obtained from the resilient modulus tests.

2.4 VARIABILITY OF BASE, SUBBASE AND SUBGRADE DESIGN PARAMETERS

This section delineates the construction and long-term variability associated with bases and subgrades (i.e., in most cases unbound materials) as they relate to pavement design parameters. The analyses conducted include the following:

1. The development of a table containing the Universal Model coefficients and related M_r-values at specific, pre-selected stress states. The table also contains location identification, moisture-density information, and material class codes.
2. The results shown in the Item 1 table are evaluated as a function of SPS site and material type. Density influences are also evaluated.
3. The development of a table containing the field moisture-density data and the corresponding laboratory moisture-density information. The variations in field densities for the SPS-1, -2 and -8 sites are also evaluated.
4. The effects of the variation in densities, as defined in Item 3, above, are applied to the results from Item 2, above, to describe the resulting variation in M_r that may be expected for pavement design purposes.
5. Back-calculated modulus variations (included in the previous Section 2.3).

The results of the base, subbase and subgrade related design parameter analyses are used to establish a set of recommended ranges, in terms of percentile and/or coefficient of variation, for the (generally) unbound material properties, applicable globally, regionally, and on a project-specific basis. The various factors that influence this variability are also isolated and reported to the extent the available LTPP data tables allowed.

2.4.1 Moduli of Unbound Materials and the Universal Model

The LTPP Test Protocol P46 was used to measure the resilient modulus (M_r) of the unbound materials, which include subgrade soils, aggregate subbase, and aggregate base materials. The deflection and strain response of a pavement system to applied loads may be estimated using mechanistic models, such as linear layered elastic models or finite element models, generally termed M_r. M_r is an input parameter in mechanistic pavement design models that are used to calculate stresses, deformations and strains within a pavement system.

The P46 test protocol results in M_r values for 15 different stress states. The stress states used depend on the type of material tested. Tests on Type 1 materials (basically 'non-cohesive' subbase and base materials) consist of five confining pressures and three axial loads at each confining pressure; different sets of axial loads are used for different confining pressures. Tests on Type 2 materials (generally 'cohesive' subgrade soils with more than 20 percent passing the 75 µm or #200 sieve) are made up of three different confining pressures with five axial loads, which are repeated for each confining pressure. Table 60 and Table 61 are examples of the stress states for Type 1 and Type 2 materials from actual test data. The strain values are shown in microstrain, rather than how they appear in the LTPP database, for visual purposes.

Table 60. Sample Summary Values for a P46 Type 1 Test

LTPP Section	Confinement Pressure, kPa	Nominal Axial Stress, kPa	Summary of Test Values and Result			
			Applied Axial Stress, kPa		Resilient	
			Cyclic	Contact	µStrain	Modulus, MPa
48-0001	20.7	20.7	18.3	2.1	160	115
48-0001	20.7	41.4	37	4.1	269	138
48-0001	20.7	62	56.3	6.2	371	152
48-0001	34.5	34.5	31.1	3.4	212	146
48-0001	34.5	68.9	62.8	6.9	345	182
48-0001	34.5	103.4	94.3	10.3	511	185
48-0001	68.9	68.9	63	6.9	297	212
48-0001	68.9	137.9	126.2	13.8	525	240
48-0001	68.9	206.8	189.5	20.7	775	244
48-0001	103.4	68.9	63.1	6.9	283	223
48-0001	103.4	103.4	94.7	10.3	360	263
48-0001	103.4	206.8	189.9	20.7	621	306
48-0001	137.9	103.4	94.9	10.3	329	288
48-0001	137.9	137.9	126.6	13.8	389	325
48-0001	137.9	275.8	253.2	27.6	703	360

Table 61. Sample Summary Values for a P46 Type 2 Test

LTPP Section	Confinement Pressure, kPa	Nominal Axial Stress, kPa	Summary of Test Values and Result			
			Applied Axial Stress, kPa		Resilient	
			Cyclic	Contact	µStrain	Modulus, MPa
48-0001	13.8	13.8	12.8	1.4	123	105
48-0001	13.8	27.6	25.3	2.8	246	103
48-0001	13.8	41.4	37.7	4.1	374	101
48-0001	13.8	55.2	50.1	5.5	507	99
48-0001	13.8	68.9	62.5	6.9	651	96
48-0001	27.6	13.8	12.8	1.4	113	113
48-0001	27.6	27.6	25.3	2.8	222	114
48-0001	27.6	41.4	37.7	4.1	340	111
48-0001	27.6	55.2	50	5.5	467	107
48-0001	27.6	68.9	62.5	6.9	603	104
48-0001	41.4	13.8	12.8	1.4	110	116
48-0001	41.4	27.6	25.2	2.8	215	117
48-0001	41.4	41.4	37.6	4.1	335	112
48-0001	41.4	55.2	49.9	5.5	459	109
48-0001	41.4	68.9	62.3	6.9	594	105

Evaluation of the variability of the results for unbound materials, as expressed by the results of the P46 test, needs to be done at pre-selected stress states. The selection of one stress state from the data would need to be checked, and perhaps adjusted, for applied confining and axial loading stresses.

The Universal Model

There are several models that have historically been used to consolidate the results of resilient modulus tests to two coefficients. The two models commonly used in the past are generically called the "bulk stress model" and the "deviator stress model". The bulk stress model is used to relate the moduli values to the sum of the principal stresses as shown in Equation 1 (in Table 60 and Table 61, the bulk stress would be the sum of the confining stress the contact axial stress and the cyclic axial stress).

$$M_R = k_1 \theta^{k_2} \quad \textbf{(1)}$$

$$M_R = k_1 \sigma^{k_3} \quad \textbf{(2)}$$

Equation 1 is most often used to characterize granular materials that have been tested as a Type 1 or non-cohesive material. These materials respond mostly to confining stresses ($\theta = \sigma_1 + \sigma_2 + \sigma_3$) and not as much to deviator (axial) stresses (σ). Equation 2 is most often used to characterize fine-grained soils that have been tested as a Type 2 or cohesive material. The deviator stress corresponds to the cyclic applied axial stresses in Table 60 and Table 61. These materials respond mostly to the axial stress and not as much to the confining stresses. The k_1, k_2 and k_3, coefficients are determined through linear regression of the log transform of the relevant variables. Both Equations 1 and 2 only allow the use of bulk stresses or deviator stresses; however, we often find that many unbound materials don't clearly fit with either the bulk stress or deviator stress models.

Another model that has been used more recently is a simplified version of what is called the "universal model". The universal model includes the bulk stress, but expresses the deviator stress as an octahedral shear stress (τ_{oct}). The model also normalizes all the stresses to atmospheres (divide the moduli or stresses by 101.325 kPa) to avoid problems with units. The Universal Model is shown in Equation 3.

$$M_R = k_1 \theta^{k_2} \tau_{oct}{}^{k_3} \quad \textbf{(3)}$$

Equation 4 shows the octahedral shear stress calculation for a triaxial sample.

$$\tau_{oct} = \frac{1}{3}\left[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2\right]^{\frac{1}{2}} \quad \textbf{(4)}$$

The k_1 through k_3 coefficients were calculated by linear regression. The M_r, θ and τ_{oct} values were all normalized by dividing by one atmosphere, p_{atm}, (101.325 kPa), followed by converting the values to base 10 logarithms $\{\log(M_R/p_{atm}) = \log(k_1) + k_2\log(\theta/p_{atm}) + k_3\log(\tau_{oct}/p_{atm})\}$. The independent variables used in the regression analysis were the logarithms of the transformed bulk and octahedral shear variables, and the dependent variable was the transformed resilient modulus. The regression values calculated included the Log (k_1), the k_2 and k_3 coefficients, the t-statistics for each of the coefficients, the regression R-squared values, and the standard error of the estimate. The R-squared results were not as good as expected. A brief examination of the results obtained show that the log moduli-log octahedral shear relationship was not always linear.

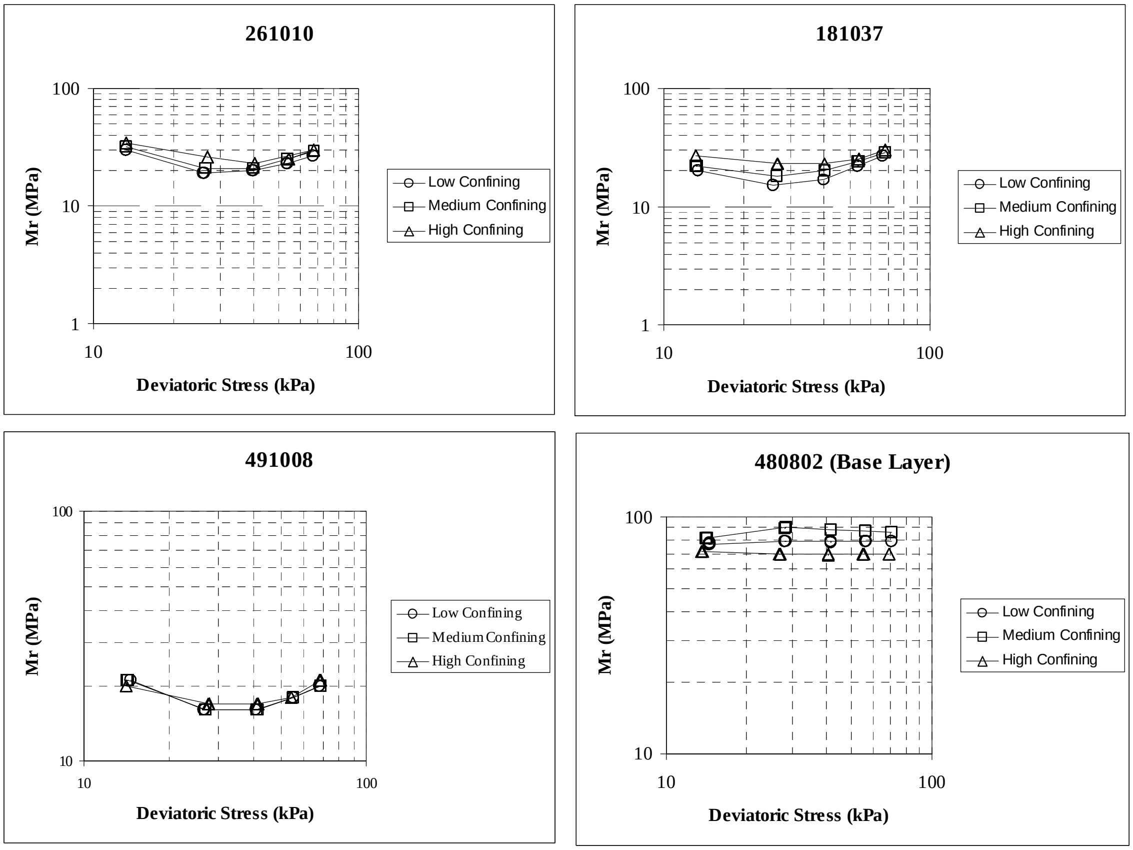


Figure 16. Examples of Non-linear Log-Log Relationship between M_r and Deviatory Stress

Figure 16 shows that the samples from Sections 26-1010, 18-1037 and 49-1008 still show non-linear behavior in a log-log plot (the plots are shown at the lowest, middle, and highest confining pressures used for the P46 test). Evidently, the fit of each example can be significantly improved by introducing a second order term of the Log (τ_{oct}) value [note that the actual regression used atmospheric normalized terms]. The last plot of the sample from 48-0802 was from an uncrushed gravel that showed almost no sensitivity to deviator stresses, but would have had improved regression results if a second order bulk stress term was included. Such cases were however few and far between, so a second order bulk stress term was not included. The resulting regression relationship used is:

$$\log(M_R/p_{atm}) = \log(k_1) + k_2\log(\theta/p_{atm}) + k_3\log(\tau_{oct}/p_{atm}) + k_4(\log(\tau_{oct}/p_{atm}))^2$$

As a result, regressions with a second order octahedral shear term significantly improved the results obtained. The second order term was significant at the 95th percentile level for about 39 percent of the test samples. The signs of the second order coefficient were both positive and negative. It is clear from the test results that many of the unbound materials tested did not always exhibit a continuous stress stiffening or stress softening behavior, but rather shifted from one behavior to the other as the stresses increased. It is conceivable that changes in soil properties, such as moisture and density, may also influence this behavior; however, the testing done for LTPP was not designed to evaluate such influences.

To ascertain the roll such properties (moisture and density for a given material) have on the type of behavior observed should be evaluated on a number of split samples of a specific material. Whether this behavior is unique to the P46 test or if it also occurs in the field cannot be determined based on the available LTPP data, and may be a good topic for future research. We have reason to believe that some materials behave similarly in the field, but this is not obvious because field tests are by necessity conducted on the entire pavement system rather than directly and solely on one specific material or pavement layer.

2.4.2 Variations in Laboratory Moduli

The variations in the M_r results from the P46 test protocol was evaluated by variability of general soil groupings, of specific soil classifications, within SPS site, and within test section. The variation of the subgrade layer within SPS sites was evaluated even if the soil classification was not the same throughout the site. One of the selection criteria for the SPS sites was that each site (usually consisting of 12 or more 500 foot test sections) should have relatively uniform soil conditions throughout. However, soil conditions at some of the sites changed, thus resulting in greater variations in modulus values.

<u>Distribution of Laboratory Moduli for All Soils</u>

Figure 17 and Figure 18 show the overall distribution of fine-grained soils (more than 20 percent passing the 0.075 mm sieve). The Figure 17 distribution is plotted on an arithmetic scale, while Figure 18 shows the distribution of the moduli on a semi-log scale. It can be seen that neither plot shows a totally normal distribution, with Figure 17 showing a tail to the right and Figure 18 a tail to the left. A visual comparison of these two figures show the bulk of the data tends to be normally distributed on an arithmetic scale.

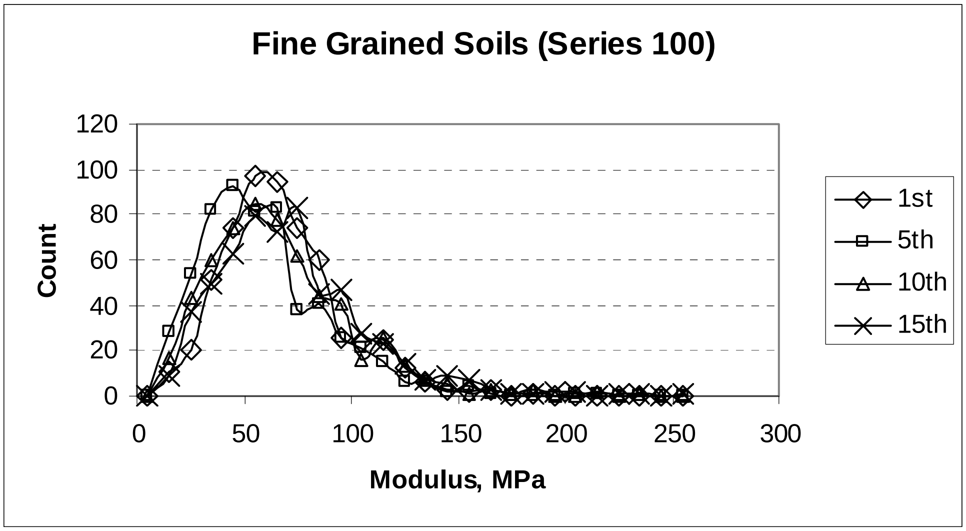


Figure 17. Laboratory Moduli Distributions for Fine-grained Soils

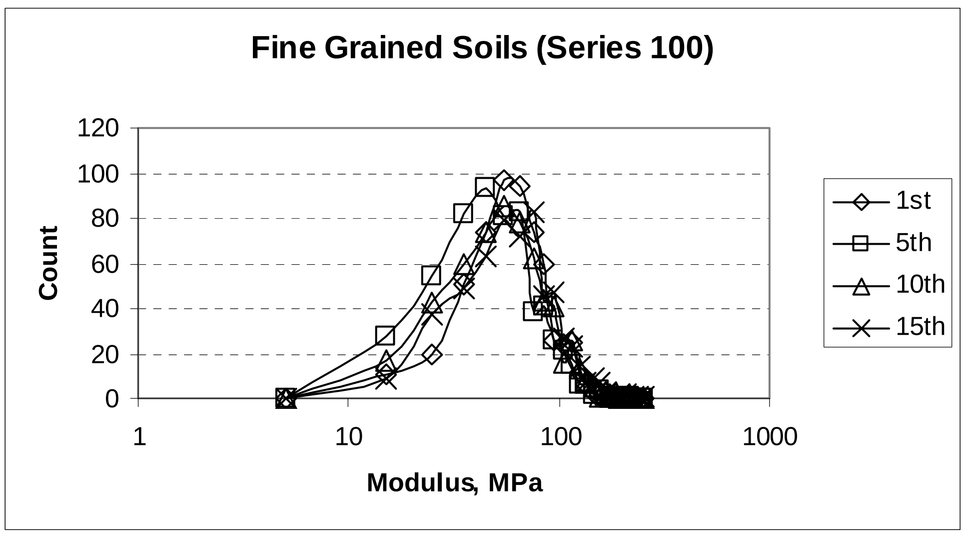


Figure 18. Laboratory Moduli for Fine-grained Soils based on a Log Distribution

For comparison purposes, the typical average moduli for the fine grain soils was around 71 MPa and its standard deviation was around ±32 MPa, which means the entire soil series had a coefficient of variation of ±45 percent.

Material dependent factors that are thought to influence the test results include grain size and composition, density and moisture content. While density, moisture and grain size (gradation) information is included in the LTPP database, grain composition (mineralogy), shape, and texture information are not. The LTPP database does include Atterberg limits, which can be used to describe some of the behavioral characteristics of the materials, but it does not appear that the liquid limit and plasticity index are good predictors of unbound material stiffness. A correlation matrix of the typical descriptive characteristics of unbound materials, such as absolute and relative moisture and density, Atterberg limits, percent fines and percent clay, along with the coefficients of the universal model are included in Appendix D. The matrix does not show a useful correlation between any of these properties and the modulus variability coefficients.

2.4.3 Variations Within SPS Sites

The variation in laboratory material moduli ranged from surprisingly low to quite high. The Texas SPS-1 site showed almost no variation in moduli (coefficients of variation ranging from two to three percent, depending on stress state), based on six samples tested throughout the site. The SPS-1 sections in Iowa and Mississippi, on the other hand, showed subgrade moduli results with coefficients of variation (CVs) ranging from 31 to 70 percent. Much of this variation can be attributed to differences in soil type; Iowa's SPS-1 had four soil classifications: LTPP material codes 101 (clay), 104 (clay with gravel), 107 (clay with sand), and 131 (clay with gravel). Mississippi's SPS-1 had three soil types.

Multiple soil types do not necessarily mean large variations, as shown by New Mexico. New Mexico had no less than seven soil types, but their typical CVs ranged between 22 and 30 percent. All of the Texas SPS-1 soils had the same classification (145), a sandy silt, which showed almost no variation between the six tests recorded, as mentioned above. Other sites showing very low variations in laboratory moduli included the Alabama SPS-1 site and the Delaware SPS-2 site.

The average CVs for the low and high stress states for all the SPS-1 and -2 sites were 27 and 25 percent, respectively. Based on this research, we consider this to be fairly typical of the modulus variation of unbound materials throughout the country (see Table 62).

It should be kept in mind, however, that most (if not all) of these SPS soil samples were reconstituted in the laboratory at 95% relative compaction, and at optimum moisture content. Thus the variations shown are more-or-less "ideal" in the sense that this is what can be achieved with proper and consistently even compaction and moisture. Other variations undoubtedly exist due to in-situ variations not reflected in the laboratory P46 test results.

Table 62. SPS Site Moduli and Variations

State	Site	Material	Material Description	No. of Tests	Low Stress Moduli[a], MPa			High Stress Moduli[b], MPa		
					Avg.	Std. Dev.				
Alabama	SPS-1	131	Silty Clay	7	63.5	4.4	7%	74.0	8.2	11%
Delaware	SPS-1	202	Poorly Graded Sand	14	40.0	8.6	21%	78.3	10.4	13%
Louisiana	SPS-1	101	Clay	6	64.6	18.2	28%	90.2	22.2	25%
Iowa	SPS-1	101	Clay	6	123.2	38.3	31%	143.3	80.6	56%
		104	Silty Clay							
		107	Clay w/Sand							
		131	Clay w/Gravel							
Nebraska	SPS-1	131	Silty Clay	5	66.1	18.9	29%	75.6	23.7	31%
Nevada	SPS-1	214 & 216	Silty Sand & Clayey Sand	5	64.0	34.6	54%	90.7	17.6	19%
New Mexico	SPS-1	102	Lean Inorganic Clay	11	53.0	11.4	22%	74.3	22.1	30%
		103	Fat Inorganic Clay							
		108	Lean Clay with Sand							
		109	Fat Clay with Sand							
		114	Sandy Lean Clay							
		115	Sandy Fat Clay							
		216	Clayey sand							
Oklahoma	SPS-1	113	Sandy Clay	8	63.1	10.9	17%	84.9	30.2	36%
Texas	SPS-1	145	Sandy Silt	6	36.8	1.2	3%	83.3	1.4	2%
Delaware	SPS-2	210	Well Graded Sand w/Silt	4	36.9	4.1	11%	82.5	9.6	12%
Delaware	SPS-2	214	Silty Sand	7	40.1	7.7	19%	79.1	4.8	6%
North Carolina	SPS-2	101 & 145	Clay and Sandy Silt	5	40.4	18.1	45%	48.6	21.7	45%
North Dakota	SPS-2	101	Clay	4	56.9	8.4	15%	50.1	11.7	23%
Mississippi	SPS-5	102	Lean Inorganic Clay	7	48.7	33.9	70%	73.9	29.4	40%
		107	Clay with Sand							
		108	Lean Clay with Sand							
Missouri	SPS-6	113	Sandy Clay	4	82.0	24.7	30%	83.8	16.9	20%

[a] low stress level corresponds to 13.8 kPa (2 psi) for both confining and axial stresses
[b] high stress level corresponds to 68.9 kPa (10 psi) for both confining and axial stresses

2.4.4 General Variations of Paired Samples

Much of the P46 resilient modulus data consisted of two, or sometimes more, samples tested from a specific layer within a section. The "paired data" showed a large amount of variation.

There were a large number of pairs available to study. "Pairs" means that there were two or more samples from 753 materials taken from the same layer (and section) that were tested. There were 1203 Type 1 (fine grain soils in a 71 by 142 mm sample) and 328 Type 2 tests (aggregate soils or bases in a 150 by 300 mm sample) that were tested, resulting in 590 and 163 test sets, respectively. Some sections had three or four tests for the same layer.

In the foregoing, within SPS site variations in unbound material moduli were investigated based on reconstituted samples, and it was found that the variations could be very small in some cases and very large in others. To look at what factors may contribute to these variations, we submit that the results from the SPS testing protocols indicate that the P46 test method is quite repeatable.

Another source of testing variability, however, could be differences between laboratories. There were two laboratories involved in the LTPP laboratory testing: Law Engineering in Atlanta, Georgia (Lab #1311) and Braun Intertec in Minneapolis, Minnesota (Lab #2711). We evaluated how the variations of the P46 test results compared from one lab to the other. It turns out that they were very similar. Overall variations for the fine grained soils (Code 100) and coarse grained soils (Code 200) series materials for paired tests from Braun Intertec were 24.7 and 25.6 percent, for the low and high stress levels, respectively. The corresponding variations for the Law Engineering lab were 22.9 and 22.6 percent, respectively. Thus the two laboratories provided similar distributions of the P46 test results.

Since there is quite a lot of data to look at, the distribution of the CVs for the tests conducted may be of interest. Histograms were developed, two for bulk samples that had to be remolded in the lab, and then tested, and two for thin wall samples that were extruded from the thin wall tube, trimmed, and prepared for testing to represent an "in-situ" condition. For each preparation method, in-situ or bulk, the distribution of the CVs of the between-pair M_r was developed for both the low and high stress states.

Histograms of the CVs obtained are shown in Figure 19 for the series 100 and 200 soils. The purpose of these histograms is to show the general variation of the P46 test results. However, a couple of general observations can be made:

- There is more consistency for the bulk samples than for the in-situ samples.
- There is more consistency for the high stress states than for the low stress states.

Factors that contribute to variability in M_r are numerous. First, the samples are often from different locations within the test section, such as from the start (Test No. 1), from the end (Test No. 2), and/or from within the section (Test No. 3). The moisture and densities may be different and the materials themselves are likely to be different. As far as within lab and within sample repeatability, we can refer to the results from Texas and Alabama's SPS-1 sites where the results were repeatable from samples taken throughout the site. We can, however, be assured that there is some variation that could be defined if a formal study was conducted to address the precision of the test method, such as the ASTM ruggedness test. Such a study is quite expensive and has not been conducted to date.

The histograms in Figure 19 for the laboratory molded bulk samples, half of the samples had less than 17 percent coefficient of variation for the low stress states and less than 12½ percent for high stress states. For the in-situ thin wall samples, the 50th percentile increases to 19 and 20 percent coefficients of variation for the low and high stress states, respectively. So the paired variability is less than the variability for the entire sample population for fine-grained soils of in-situ samples.

Moisture and density are normally thought to be major factors in the resilient modulus, and there are prior studies to support this conclusion. The roll that variations in moisture and density play in the variation of M_r is illustrated by comparing the CVs of M_r vs. the CVs of moisture and density. The expectation was that, at least, a high variation in either moisture or density would result in high variation in M_r. Unfortunately, this was not the case.

Figure 20 and Figure 21 are scatter plots of the within-pair coefficients of variation for density plotted against the variation of M_r (Figure 20) and the variation in M_r plotted against the variation in moisture content (Figure 21). The premise is that within-pair differences in either moisture or density would result in difference in M_r; however these plots do not support this premise. This raises the possibility that physical variations within the same class of materials may be more dominant than is indicated by a variation in moisture and/or density for LTPP tests. This is not to say that moisture and density are not significant. Tests on carefully split (identical) samples are needed to measure these effects. This kind of test, unfortunately, was not a part of the overall LTPP testing program.

2.4.5 Variation by Material Class

Traditional soils characterization is based on grain size distribution and Atterberg limits, resulting in classification of these materials by descriptive terms as used in LTPP and shown in Table 63, or by class codes as used by AASHTO. Many agencies assign default support values to soils on the basis of AASHTO classification. Aggregates for subbase and base courses are more commonly assigned support values on the basis of their gradations.

Fine-grained soils (100 code series), as a group, have about the same overall variation as all of the materials tested, with 42 and 70 percent for coefficients of variation for the low and high stress states respectively. The variation for the coarse-grained soils (200 code series) and materials used for unbound base and subbase (300 code series) were lower and very similar at 41 and 39 percent, respectively, for low stress states and 36 and 35 percent, respectively, for high stress states. The individual classes generally show slightly lower variation for tests conducted on materials from within a class than for the broader soil groupings for the 200 and 300 series soils.

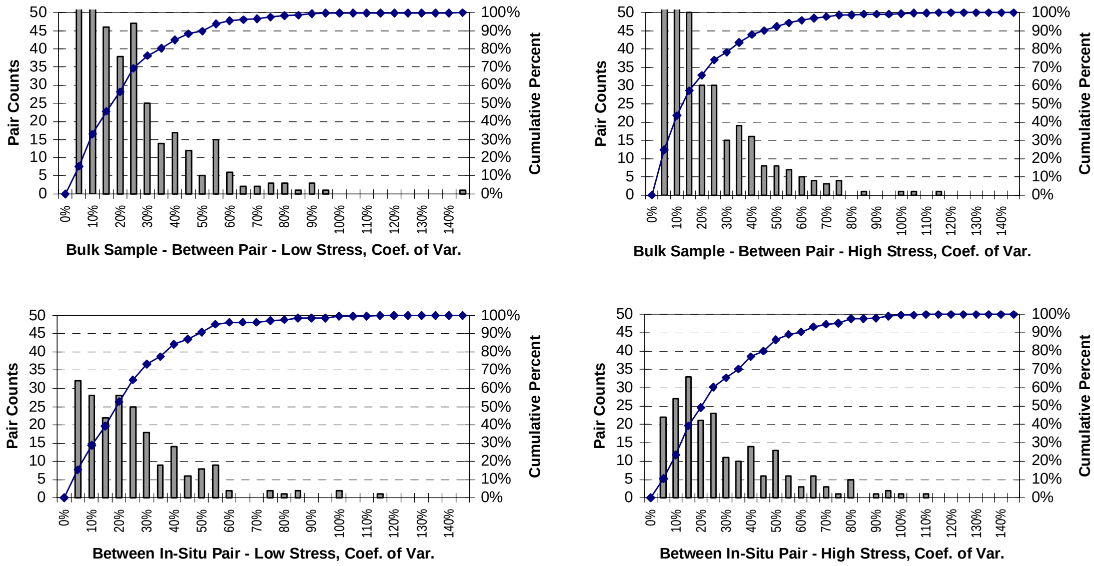


Figure 19. Distribution of Coefficients of Variation for Paired Soil Test Results

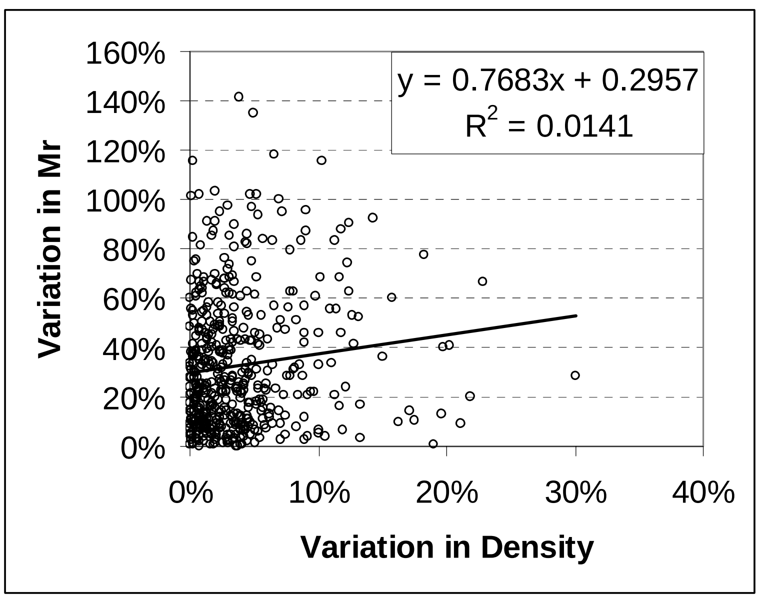


Figure 20. Variations in Density

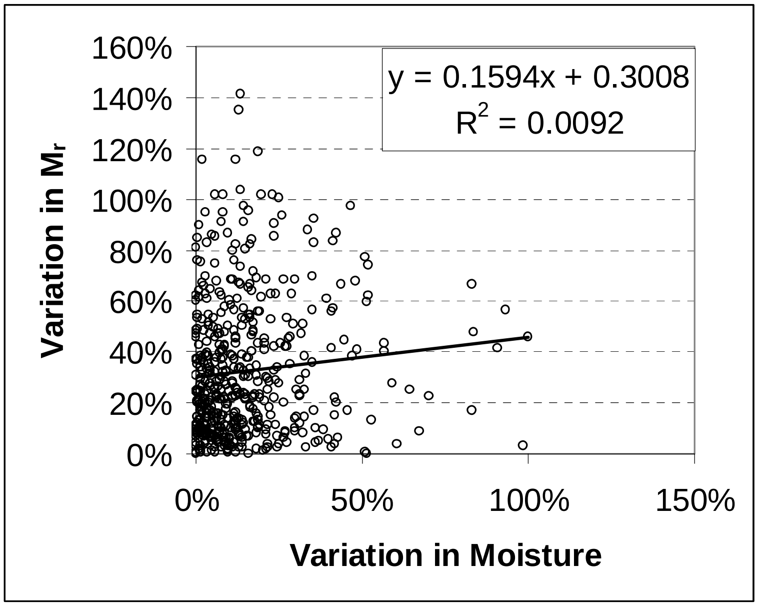


Figure 21. Variations in Moisture

Table 63. Variation of M_r by Same Class Materials

Code	Description	Count	Averages and Standard Deviations are in MPa					
			Low Stress State			High Stress State		
			Avg.	Std. Dev.	CV	Avg.	Std. Dev.	CV
ALL	All Soils and Aggregates Tested	1905	67.7	28.6	42.2%	123.6	74.8	60.5%
Fine-Grained Soils:								
101	Clay	20	71.6	27.3	38.1%	83.9	39.3	46.8%
102	Lean Inorganic Clay	86	66.0	23.9	36.2%	67.1	28.8	43.0%
103	Fat Inorganic Clay	31	58.7	16.2	27.6%	61.8	25.3	40.9%
104	Clay with Gravel	3	95.5	24.0	25.1%	85.4	39.3	46.0%
105	Lean Clay with Gravel	2	109.7	22.4	20.4%	136.8	7.7	5.7%
107	Clay with Sand	12	71.4	45.1	63.1%	81.9	66.2	80.8%
108	Lean Clay with Sand	91	66.6	32.3	48.5%	73.7	46.2	62.7%
109	Fat Clay with Sand	21	65.0	23.9	36.8%	70.1	26.4	37.6%
111	Gravelly Lean Clay	10	76.4	31.9	41.7%	84.6	31.8	37.6%
112	Gravelly Fat Clay	2	41.7	9.0	21.6%	43.6	15.5	35.5%
113	Sandy Clay	31	75.1	27.9	37.1%	90.6	34.9	38.5%
114	Sandy Lean Clay	115	75.2	29.1	38.7%	86.9	43.7	50.2%
115	Sandy Fat Clay	4	65.1	26.8	41.1%	77.4	29.8	38.6%
116	Gravelly Clay with Sand	2	79.0	12.2	15.5%	123.6	47.5	38.4%
117	Gravelly Lean Clay with Sand	4	94.8	13.6	14.4%	124.9	33.0	26.4%
118	Gravelly Fat Clay with Sand	2	54.6	26.4	48.4%	57.9	44.3	76.5%
119	Sandy Clay with Gravel	1	50.5	NA	NA	46.2	NA	NA
120	Sandy Lean Clay with Gravel	9	74.4	39.8	53.5%	200.9	326.7	162.6%
131	Silty Clay	31	78.7	25.2	32.1%	86.1	32.2	37.4%
133	Silty Clay with Sand	8	57.7	28.5	49.4%	56.0	23.1	41.3%
134	Gravelly Silty Clay	2	81.8	19.3	23.6%	95.9	20.1	20.9%
135	Sandy Silty Clay	11	59.0	35.1	59.5%	86.0	55.9	65.0%
136	Gravelly Silty Clay with Sand	2	44.3	12.5	28.2%	62.7	10.6	16.9%
137	Sandy Silty Clay with Gravel	1	71.7	NA	NA	79.8	NA	NA
141	Silt	30	51.2	21.5	41.9%	78.7	40.6	51.6%
142	Silt with Gravel	1	58.9	NA	NA	93.1	NA	NA

Code	Description	Count	Averages and Standard Deviations are in MPa					
			Low Stress State			High Stress State		
			Avg.	Std. Dev.	CV	Avg.	Std. Dev.	CV
143	Silt with Sand	15	51.7	11.2	21.7%	74.7	20.0	26.7%
144	Gravelly Silt	2	75.4	13.7	18.1%	96.5	24.6	25.5%
145	Sandy Silt	59	52.1	22.4	43.0%	75.0	36.4	48.5%
146	Gravelly Silt with Sand	6	85.8	16.0	18.7%	118.4	29.9	25.3%
147	Sandy Silt with Gravel	1	59.9	NA	NA	107.8	NA	NA
148	Clayey Silt	3	44.7	26.0	58.1%	52.9	26.9	50.8%
Course-Grained Soils:								
201	Sand	8	64.5	36.6	56.6%	117.9	25.5	21.6%
202	Poorly Graded Sand	84	52.4	13.3	25.4%	112.6	30.5	27.1%
203	Poorly Graded Sand with Gravel	7	48.1	2.6	5.4%	126.2	4.4	3.5%
204	Poorly Graded Sand with Silt	53	49.0	11.6	23.7%	104.3	25.4	24.4%
205	Poorly Graded Sand with Silt and Gravel	5	45.2	12.5	27.6%	96.9	12.6	13.0%
210	Well-Graded Sand with Silt	11	42.0	7.1	17.0%	95.7	16.0	16.7%
211	Well-Graded Sand with Silt and Gravel	1	36.4	NA	NA	76.6	NA	NA
214	Silty sand	167	53.0	20.8	39.2%	88.4	27.8	31.4%
215	Silty sand with gravel	83	55.3	23.9	43.2%	87.5	30.2	34.6%
216	Clayey sand	81	66.5	24.5	36.8%	89.6	31.6	35.2%
217	Clayey sand with gravel	51	72.2	27.4	38.0%	94.3	39.1	41.5%
252	Poorly graded gravel	2	73.0	2.3	3.2%	56.1	8.1	14.5%
254	Poorly graded gravel with silt	2	60.0	24.0	40.0%	86.2	30.7	35.6%
255	Poorly graded gravel with silt and sand	1	140.9	NA	NA	142.7	NA	NA
256	Poorly graded gravel with clay	2	80.1	19.6	24.5%	103.0	27.6	26.8%
257	Poorly graded gravel with clay and sand	2	89.4	15.9	17.8%	89.9	4.3	4.7%
259	Well-graded gravel with sand	1	47.0	NA	NA	162.4	NA	NA
264	Silty gravel	1	36.8	NA	NA	54.3	NA	NA
265	Silty gravel with sand	36	64.3	25.2	39.3%	92.3	34.1	36.9%
266	Clayey gravel	5	71.9	41.3	57.5%	91.1	72.1	79.1%
267	Clayey gravel with sand	70	78.8	28.2	35.8%	104.9	48.6	46.3%
282	Rock	2	88.1	47.3	53.6%	115.9	48.4	41.8%
294	Other (specify if possible or unknown)	1	61.3	NA	NA	55.9	NA	NA
Materials Used as Subbase or Base:								
302	Gravel (uncrushed)	82	80.1	39.7	49.6%	196.8	45.1	22.9%
303	Crushed Stone	94	87.5	30.0	34.3%	256.9	62.8	24.5%
304	Crushed Gravel	47	78.5	23.8	30.3%	205.5	47.7	23.2%
306	Sand	56	56.1	21.0	37.5%	127.5	25.3	19.8%
307	Soil-Aggregate Mixture (predominantly fine-grained)	29	66.8	21.7	32.5%	121.5	35.1	28.8%
308	Soil-Aggregate Mixture (predominantly coarse-grained)	185	80.9	28.9	35.8%	219.0	68.2	31.2%
309	Fine-grained Soils	102	68.5	22.3	32.5%	152.5	32.6	21.4%
310	Other (Specify if possible)	1	84.1	NA	NA	122.4	NA	NA
337	Limerock, Caliche	15	110.9	37.5	33.8%	280.6	30.1	10.7%

2.4.6 Variation in In-Situ Moisture and Density

The field moisture and densities were evaluated by SPS site and material type. Table 64 summarizes both the moisture and density by SPS experiment and the soil classification for subgrade (Layer 1) materials only. It should be noted that many SPS sites had a second subgrade layer (Layer 2) that was either a different material or treated subgrade soil. Layer 2 results are not included.

Laboratory data on maximum density and optimum moistures for most of these soils were not available to provide information on the basis of relative moisture and density.

The distributions of the site CV for moisture and density are shown in Figure 22 and Figure 23. The majority of the SPS sites had density coefficients of variation of four percent or less, while moisture coefficients of variation were 25 percent or less. The standard deviation values for both the density and moisture are independent of the actual (average) moisture and density.

Table 65 summarizes the average, standard deviation, and coefficients of variation of the density and moisture content of the granular base materials used in the SPS sites. Site by site, the field moisture and density tests show fairly consistent results. The density coefficient of variations range from 0.7 to 5.2 percent with an average CV of 2.6 percent for the 34 SPS experiment sites data were available. The average site moisture content showed a range of 1.8 to 8.1 percent and a CV range of 5 to 62 percent. Relative densities and moisture contents were not available.

The distribution of density and moisture CV for the 34 SPS 1, 2 and 8 sites with adequate data are shown in Figure 24 and Figure 25, respectively. Figure 24 shows that the most common CV for density is 2.5 percent, with the bulk of the CVs ranging between 1 and 4 percent. The CVs of the measured moisture contents at 33 SPS sites are shown in Figure 25. Twelve sites had CV values for moisture between 15 and 20 percent, which were the most common CV values, while nearly all the sites had CV values below 35 percent. It should be noted that the CV for the moisture corresponds to a relatively low standard deviation. It may also be interesting to look at the standard deviation of moisture content of the aggregate base materials within a site. For these materials, the standard deviation ranged between 0.2 and 3.6 percent and averaged 1.6 percent.

Based on the available data, we cannot evaluate how changes in moisture or density affect the stiffness of unbound materials. Other studies are needed to establish how sensitive different materials are to such variation. The LTPP data, however, does show what the variations in moisture and density are for the SPS-1, -2 and -8 sites. These SPS sites are typical of a highway construction projects that are designed on the basis of consistent material properties for the subgrade soil and granular base materials.

Table 64. Variations in Field Subgrade Moisture and Density

State	SPS Exp.	Material Code	Density, kg/m³				Moisture, percent			
			Count	Avg.	Std Dev.	CV.	Count	Avg.	Std Dev.	CV.
Arizona	1	Well-Graded Sand with Silt and Gravel	10	2,079	82	4.0%	10	5.0	1.8	36.8%
Arizona	1	Silty sand with gravel	19	2,099	49	2.3%	19	3.4	0.5	16.1%
Arizona	1	Clayey sand with gravel	3	2,086	19	0.9%	3	3.8	0.2	4.4%
Arizona	1	Well-graded gravel with silt and sand	7	2,057	56	2.7%	7	7.2	1.8	24.5%
Arizona	2	Silty sand with gravel	24	1,980	70	3.5%	21	5.6	1.2	21.2%
Arizona	2	Clayey sand with gravel	12	1,948	62	3.2%	3	4.9	0.8	17.3%
Colorado	2	Poorly Graded Sand with Silt	6	1,856	27	1.4%	6	6.6	1.0	15.5%
Colorado	2	Clayey sand with gravel	9	1,835	54	2.9%	9	10.9	3.3	30.0%
Colorado	2	Sandy Clay	3	1,839	67	3.6%	3	9.0	3.5	39.2%
Colorado	2	Sandy Lean Clay	9	1,766	65	3.7%	9	13.9	2.4	17.4%
Colorado	2	Clayey sand	3	1,745	31	1.8%	3	13.5	0.8	5.6%
Delaware	1	Poorly Graded Sand	1	2,106	-----	-----	1	9.8	-----	-----
Delaware	2	Well-Graded Sand with Silt	1	1,942	-----	-----	1	9.2	-----	-----
Delaware	2	Silty sand	1	2,145	-----	-----	1	10.9	-----	-----
Florida	1	Silty sand with gravel	21	2,101	52	2.5%	21	7.3	1.2	16.0%
Florida	1	Poorly Graded Sand with Silt and Gravel	14	2,090	52	2.5%	14	7.3	1.8	25.0%
Iowa	2	Clay with Gravel	4	1,811	85	4.7%	4	16.6	2.0	12.1%
Kansas	2	Silty Clay	4	1,699	89	5.2%	4	19.6	2.1	10.6%
Louisiana	1	Lean Inorganic Clay	1	1,516	-----	-----	1	24.3	-----	-----
Michigan	1	Sandy Clay	40	1,984	70	3.5%	40	7.9	1.5	19.2%
Michigan	2	Silty Clay	27	2,030	115	5.7%	27	9.8	2.0	20.7%
Michigan	2	Sandy Clay	7	1,928	153	7.9%	7	11.9	2.3	19.7%
Montana	1	Poorly Graded Sand with Silt	59	1,971	193	9.8%	59	7.7	3.6	47.4%
Montana	8	Poorly graded gravel with silt	4	2,137	31	1.5%	4	6.1	1.0	17.1%
Nevada	1	Silty sand	3	1,654	110	6.6%	3	12.5	1.5	11.9%
Nevada	1	Clayey sand	3	1,734	52	3.0%	3	16.9	1.2	7.2%
Nevada	2	Silty sand with gravel	1	1,690	-----	-----	1	10.6	-----	-----
Nevada	2	Sandy Silt	4	1,624	56	3.4%	4	18.3	2.1	11.3%
New Jersey	8	Poorly Graded Sand with Silt	6	1,896	75	4.0%	6	9.8	1.7	17.0%
New Mexico	1	Sandy Lean Clay	4	1,900	10	0.5%	4	16.5	1.5	9.2%
New Mexico	1	Fat Inorganic Clay	10	1,911	14	0.7%	10	17.6	2.1	12.2%
New Mexico	1	Lean Clay with Sand	3	1,903	10	0.5%	3	13.2	2.6	19.8%
New Mexico	1	Sandy Fat Clay	3	1,945	35	1.8%	3	18.6	0.2	1.0%
New Mexico	1	Clayey sand	4	1,817	128	7.1%	4	22.7	1.5	6.8%
New Mexico	1	Lean Inorganic Clay	3	1,800	30	1.6%	3	24.8	0.5	1.9%
New York	8	Clayey sand	4	1,917	63	3.3%	4	8.4	1.0	11.8%
North Carolina	8	Sand	4	1,740	18	1.1%	4	7.6	1.6	20.5%
North Carolina	8	Silty sand	4	1,913	126	6.6%	4	6.9	0.6	8.5%
North Dakota	2	Clay	27	1,542	54	3.5%	27	24.8	1.9	7.7%
Ohio	1	Silty Clay	38	1,931	67	3.5%	38	8.8	1.4	15.5%
Ohio	2	Silty Clay	36	1,905	57	3.0%	36	9.8	1.8	17.9%
Ohio	8	Silty Clay	3	1,890	109	5.8%	3	14.6	2.8	19.3%
Oklahoma	1	Clayey sand	10	1,795	53	3.0%	10	13.6	1.4	10.6%
Texas	1	Poorly Graded Sand with Silt	39	1,559	103	6.6%	39	9.1	2.5	27.3%
Texas	8	Sandy Silt	6	1,624	37	2.3%	6	20.8	2.4	11.6%

State	SPS Exp.	Material Code	Density, kg/m^3				Moisture, percent			
			Count	Avg.	Std Dev.	CV.	Count	Avg.	Std Dev.	CV.
Utah	8	Clayey sand with gravel	5	1,742	131	7.5%	5	13.3	1.4	10.5%
Washington	2	Poorly graded gravel	3	1,882	36	1.9%	3	9.6	0.4	4.6%
Wisconsin	1	Silty sand	33	2,242	84	3.7%	6	3.0	0.7	23.1%
Wisconsin	2	Silty sand	33	2,100	99	4.7%	33	4.4	1.8	40.1%
Wisconsin	8	Silty sand	30	2,167	81	3.8%	18	4.7	1.1	23.9%

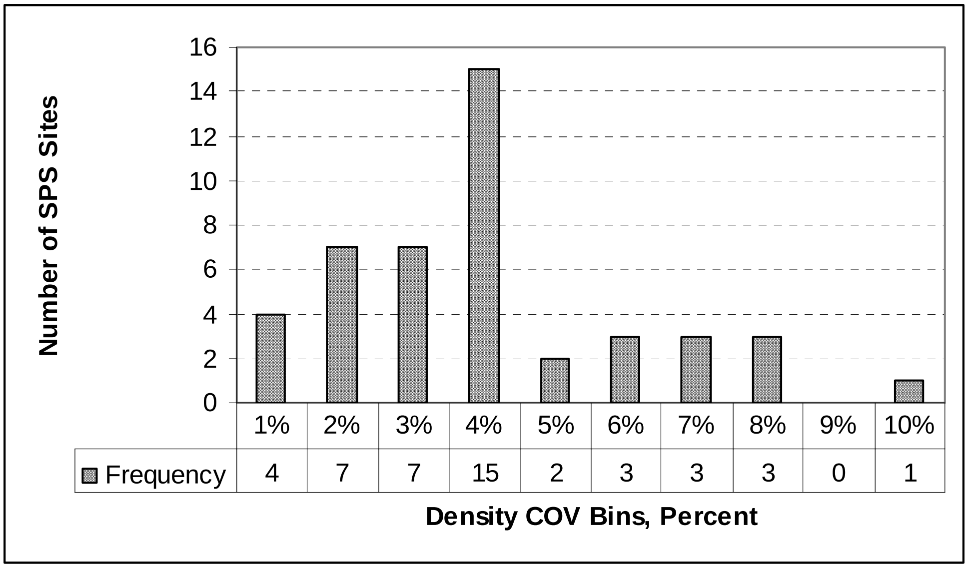


Figure 22. Subgrade Density Variations for SPS Sites

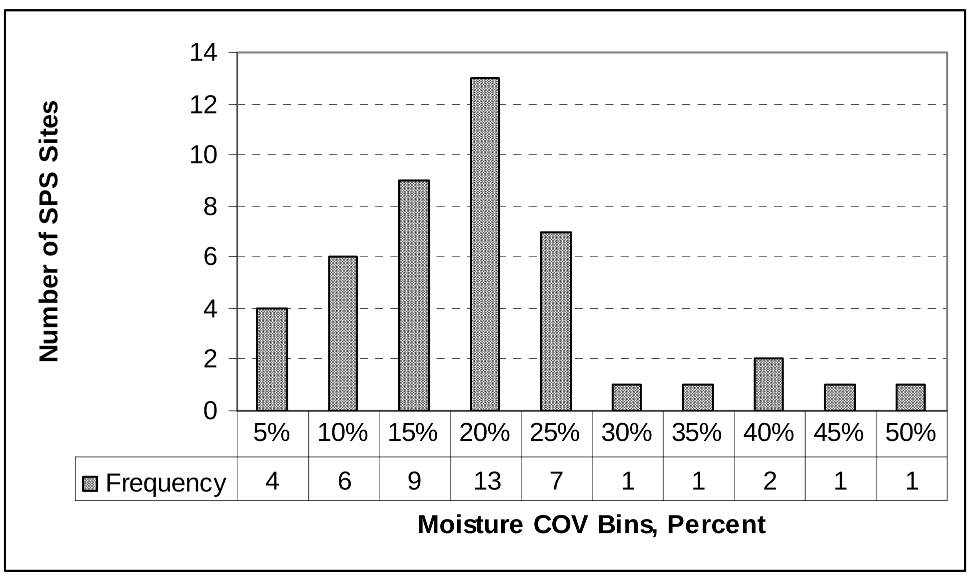


Figure 23. Moisture Variation for SPS Sites

Table 65. Variation of Moisture and Density for Granular Bases

State	SPS	Material	Density, pcf				Moisture			
			Count	Avg.	Std.Dev.	CV	Count	Avg.	Std.Dev.	CV
Arizona	1	Cr. Gravel	26	2154	49	2.3%	25	2.9	0.6	20.6%
Arizona	2	Cr. Gravel	24	2050	47	2.3%	18	3.3	0.8	23.3%
Colorado	2	Cr. Gravel	21	2108	51	2.4%	21	4.5	0.9	19.2%
Delaware	1	Cr. Stone	15	2488	51	2.1%	15	5.5	0.5	8.8%
Delaware	2	Cr. Stone	23	2406	50	2.1%	23	5.1	0.7	14.4%
Florida	1	Cr. Stone	27	2196	50	2.3%	27	4.4	1.2	26.1%
Iowa	1	Cr. Stone	18	2138	35	1.6%	18	3.6	0.6	15.6%
Iowa	2	Cr. Stone	27	2069	48	2.3%	27	4.8	1.1	23.1%
Kansas	1	Cr. Stone	22	2200	97	4.4%	22	3.4	2.1	62.2%
Kansas	2	Cr. Stone	15	1934	101	5.2%	15	7.5	1.2	15.8%
Louisiana	1	Cr. Stone	12	1934	36	1.9%	12	5.0	0.7	13.2%
Michigan	1	Cr. Stone	24	2209	63	2.9%	24	3.3	0.9	26.5%
Michigan	2	Cr. Stone	28	2194	56	2.6%	28	3.2	1.1	34.6%
Montana	1	Cr. Gravel	20	2226	16	0.7%	20	5.5	0.3	5.4%
Montana	8	Cr. Gravel	12	2149	52	2.4%	12	5.3	0.9	17.6%
Nebraska	1	Cr. Stone	4	2142	20	0.9%	4	2.3	0.4	17.3%
Nevada	1	Cr. Gravel	24	2071	54	2.6%	24	4.7	0.6	11.9%
Nevada	2	Cr. Gravel	25	2158	59	2.8%	25	5.6	0.5	8.1%
New Jersey	8	Cr. Gravel	6	2276	55	2.4%	6	1.8	0.2	9.6%
New Mexico	1	Cr. Stone	24	2052	45	2.2%	24	4.1	1.3	30.7%
New York	8	Cr. Gravel	8	2201	22	1.0%	8	2.4	0.2	9.2%
North Carolina	2	Cr. Gravel	24	2222	106	4.8%	24	5.5	1.0	18.6%
North Carolina	8	Cr. Gravel	8	2197	45	2.1%	8	2.4	0.1	6.0%
North Dakota	2	Cr. Stone	15	2202	85	3.9%	15	3.6	0.7	18.3%
Ohio	1	Cr. Stone	22	1988	78	3.9%	22	5.8	1.7	29.1%
Ohio	2	Cr. Stone	26	1979	70	3.6%	26	3.8	1.0	25.3%
Ohio	8	Cr. Stone	15	2098	75	3.6%	15	6.7	1.5	22.4%
Oklahoma	1	Cr. Stone	24	2160	78	3.6%	24	4.1	1.2	30.7%
Texas	1	Gravel	15	2107	72	3.4%	15	5.4	1.2	22.6%
Texas	8	Gravel	6	2287	25	1.1%	0	----	----	----
Utah	8	Cr. Gravel	9	2065	19	0.9%	9	3.7	0.6	16.2%
Virginia	1	Cr. Stone	24	2048	63	3.1%	24	4.9	0.7	13.8%
Washington	2	Cr. Stone	25	2208	40	1.8%	25	4.9	0.6	11.7%
Washington	8	Cr. Stone	9	2336	45	1.9%	9	8.1	0.9	11.0%

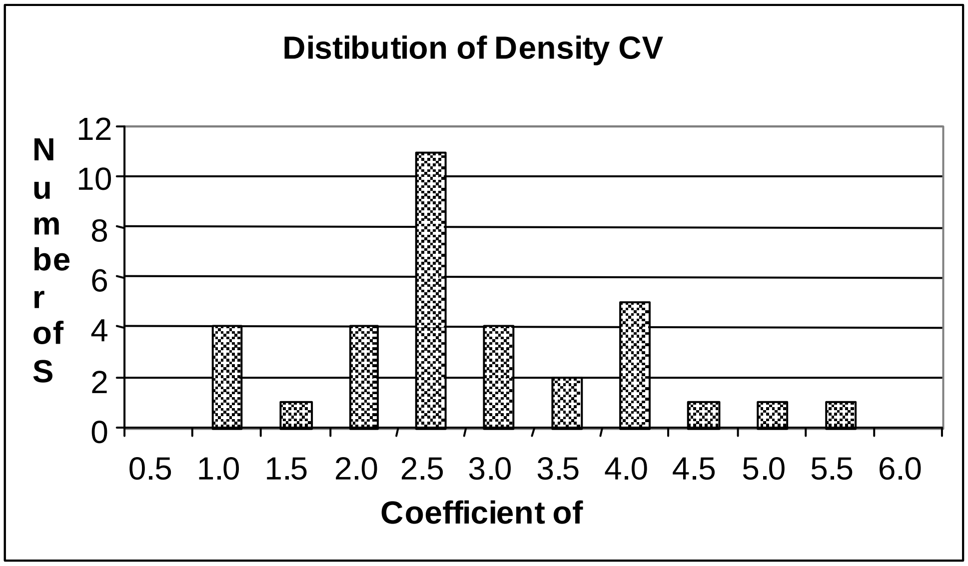


Figure 24. CVs for Density on SPS-1, -2 and -8 Sites

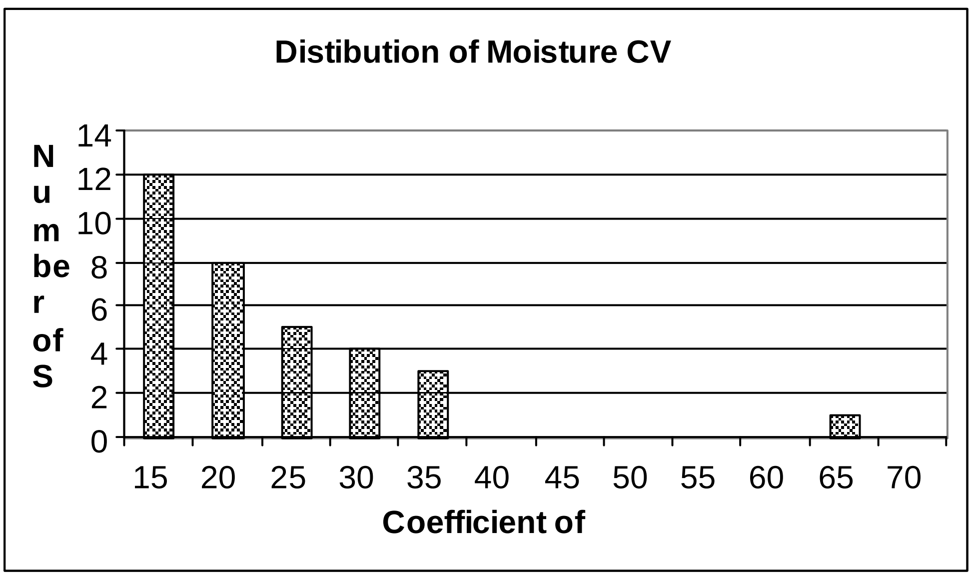


Figure 25. CVs for Moisture on SPS-1, -2 and -8 Sites

2.4.7 Relationship between Maximum Density and Optimum Moisture

Within the LTPP data, there were a number of laboratory moisture/density test results for the resilient modulus data, however not for the SPS sites discussed above. Higher density materials such as aggregates for base and subbase typically have lower optimum moisture content, while lower density materials such as fine-grained soils tend to have high optimum moisture contents. Although this relationship doesn't directly relate to variation of design data inputs, it does provide information that is useful as a reference (see Figure 26). Although there is a wide range of optimum moisture values that exist for a maximum density, and/or a wide range of densities for a given optimum moisture content, the relationship may be useful in establishing a general quality range check for laboratory moisture density data.

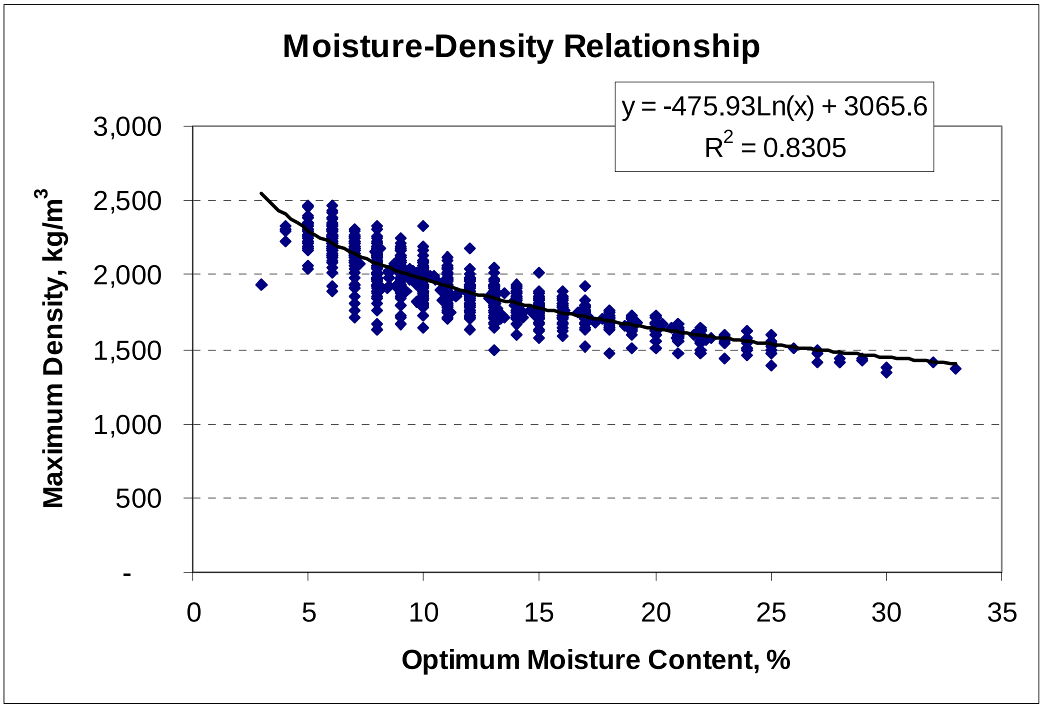


Figure 26. Relationship between Optimum Moisture and Density for Unbound Materials

2.5 VARIABILITY OF TRAFFIC LOAD DATA

The traffic data were analyzed to examine trends in long-term variability and to look for the existence of patterns or trends on a state or regional level. If stable trends exist, they may be very helpful in estimating traffic values for use in pavement design, thus allowing huge savings in traffic data collection costs.

2.5.1 Data Source

The data were obtained directly from the LTPP Information Management System (IMS). The following three criteria were used for the initial selection of test sections used to investigate the variability in traffic data:

- A minimum of 210 days of Automatic Vehicle Classification (AVC) data per year.
- A minimum of 210 days of weigh in motion (WIM) data per year.
- A minimum of 2 years of data meeting the first two criteria.

Of the 177 potential LTPP test sections, 150 sections had sufficient AVC data and 148 sections had sufficient WIM data to meet the minimum criteria. Ninety-three AVC sites and 91 WIM sites contained 3 or more years of data. Emphasis was placed on sites with 3 or more years of data, as they provided much better indications of long-term (temporal) trends in traffic load variability.

Data for single axles, tandem axle groups and tridem axle groups were analyzed separately to identify trends in weight distributions. While information for quad axle groups was available, the occurrence of this loading group was insufficient to draw clear conclusions. Therefore, efforts were focused on the single, tandem and tridem axle configurations, which account for most of the axle types encountered on typical US primary roadways.

2.5.2 Vehicle Class Data

The AVC data within LTPP were used to examine trends in truck axle and total load distributions. The data were normalized on an annual basis to take out seasonal variability (which was not examined in this analysis). In this section, normalization refers to the process of determining the percentage contribution of each vehicle class to the total number of vehicles counted in a year.

Figure 27 provides the trend for Test Section 26-1010 located in Michigan. The plot shows the normalized data by year. This section has five years of available data. As can be seen, the data for each of these years follows the same general trend.

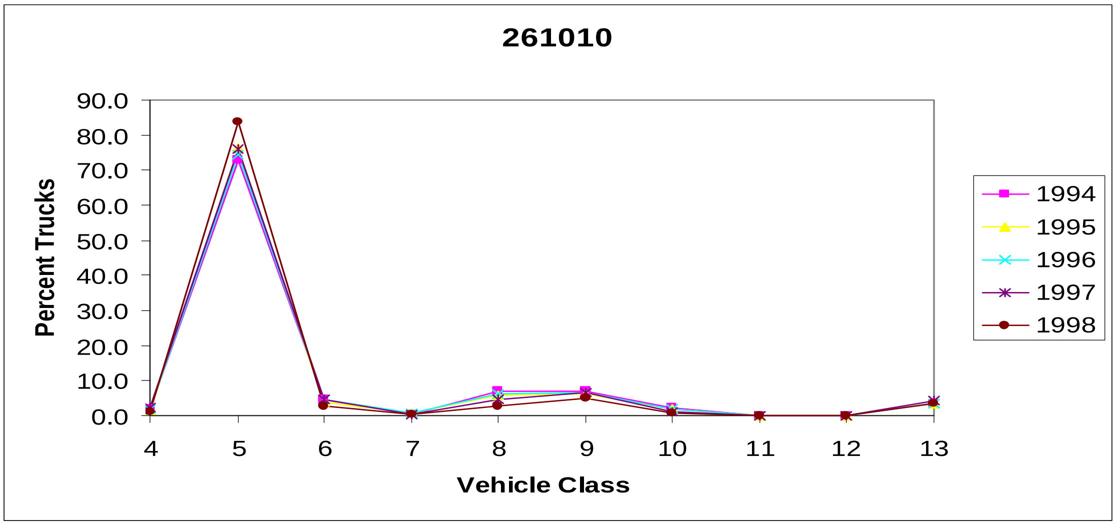


Figure 27. Normalized AVC Data for Test Section 26-1010

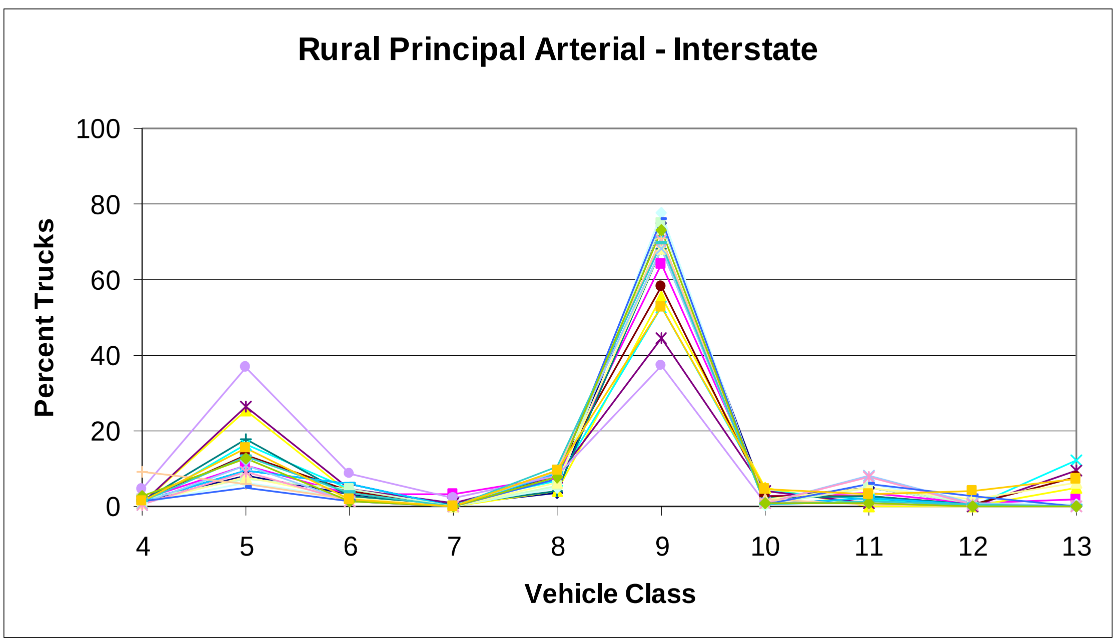


Figure 28. Vehicle Class Distributions for Test Sections in the Rural Arterial Class

The data for each individual section were graphed and representative graphs are provided in Appendix E. All of the graphs indicate that the trends for a single section were fairly consistent across measurement years.

The data for each section were averaged and graphs were prepared in terms of functional class. Figure 28 provides the graphs for the "rural principal arterial" functional class. This graph shows that there are substantial differences in variability between test sections within the same functional class. For instance, vehicle class 9 accounts for between 40 percent and 80 percent of all trucks on a section. This is equivalent to a doubling (or halving) of the expected number of heavy loads between sections having the same functional class.

The variability seen in Figure 28 is indicative of the variability seen on the other similar graphs, shown in Appendix E. Hence, assuming a distribution based on the functional class of a roadway can lead to erroneous traffic estimates.

The data were also examined by State or Province. The average vehicle distributions for each test section were plotted for a given state. As an example, Figure 29 provides the graph for Mississippi. The variability seen here is typical of that found in other States. Hence, as with the functional class, assuming a vehicle distribution based on a state or region can lead to erroneous traffic estimates.

Test sections with four or more years of AVC data were analyzed further. Statistical analyses were conducted to determine if the trend from one year was the same as that from another. Each pair of years was considered in this analysis. For all 40 test sections having four or more years of traffic load data, there are no statistically significant differences observed between any two (or more) years. Therefore, once the distribution of vehicle classes is known for a roadway for a year, the distribution can be used as a good estimate for other years, as long as the demographics and/or usage of the roadway have not changed.

2.5.3 Weigh-In-Motion Data

The weigh-in-motion (WIM) data were also normalized. In this section, normalization refers to the process of determining the percentage contribution of each axle load group to the total number of axles counted in a year for each axle type. Annual trends for the single, tandem and tridem axle types were examined for variability.

Graphs were produced for each of the three axle load configurations studied. Figure 30 illustrates the typical data for tandem axles measured on Test Section 06-2040 in California. The data shows a very consistent trend between years.

These data were also plotted by functional class and by state or province. As noted with the AVC data, the variability associated with a functional class, or with a state or province, is so large that it would be difficult to assume a distribution based on functional class or State (or region). Figure 31 and Figure 32 provide examples of the typical variability seen within a functional class or a state.

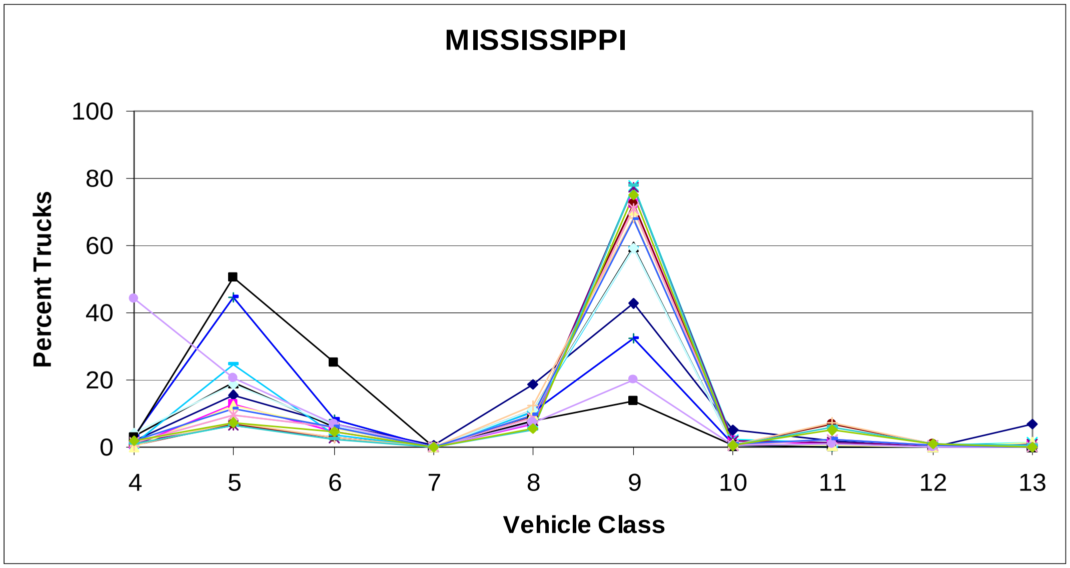


Figure 29. Vehicle Distributions for the State of Mississippi

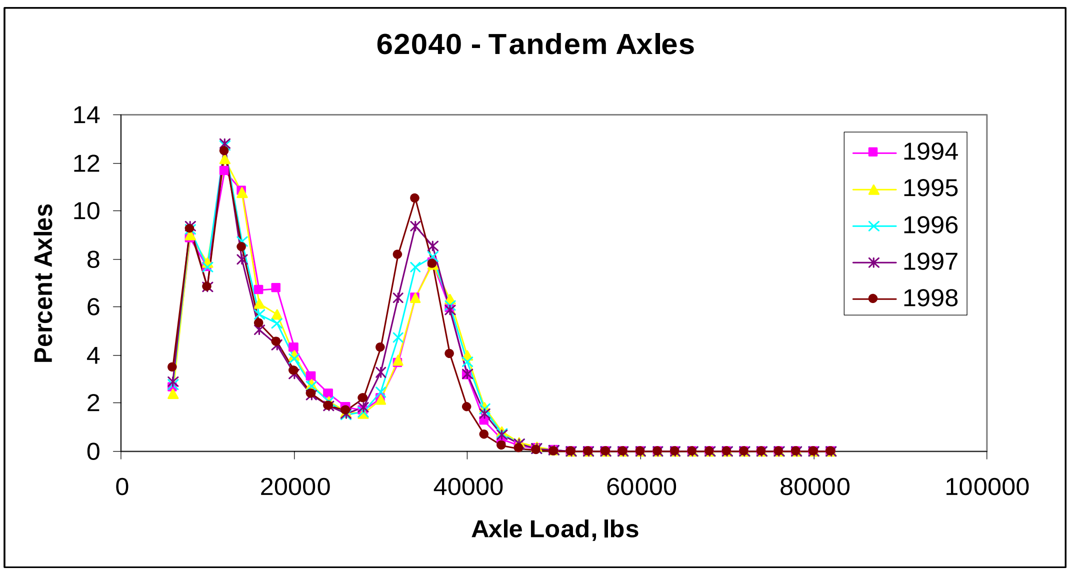


Figure 30. Distribution of Tandem Axle Loads on Test Section 06-2040

Table 66. Number of Available Test Sections for Each Axle Configuration

Axle Configuration	No. of Test Sections
Single	46
Tandem	46
Tridem	44

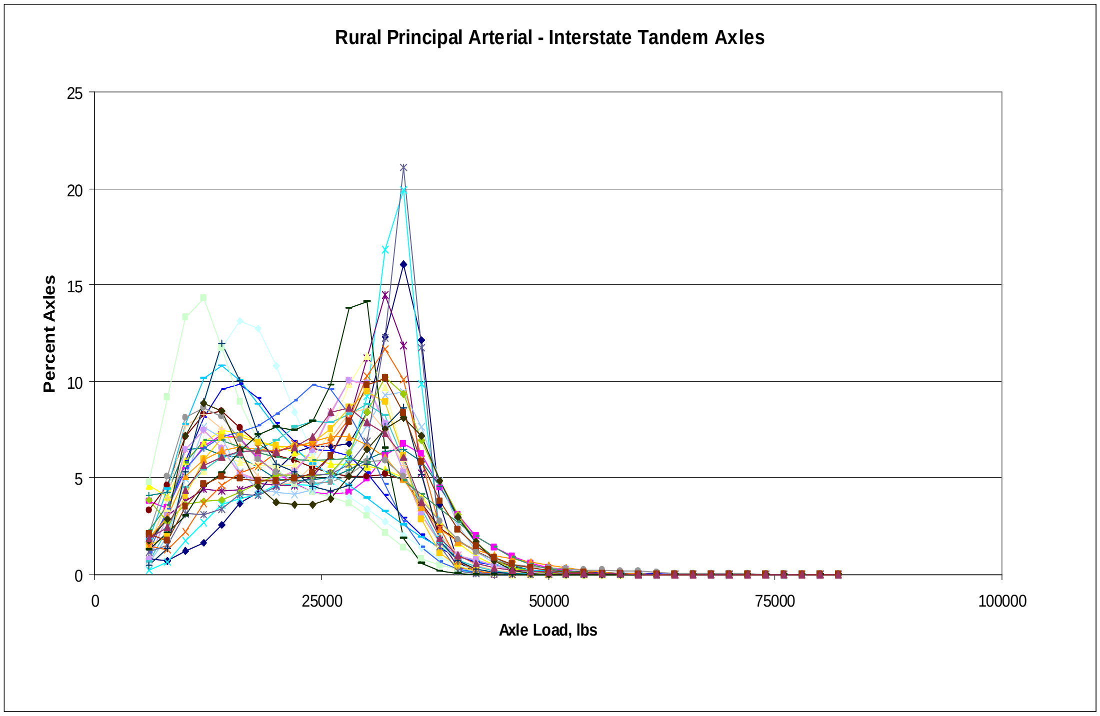


Figure 31. Axle Load Spectra for Tandem Axles on Rural Principal Arterials

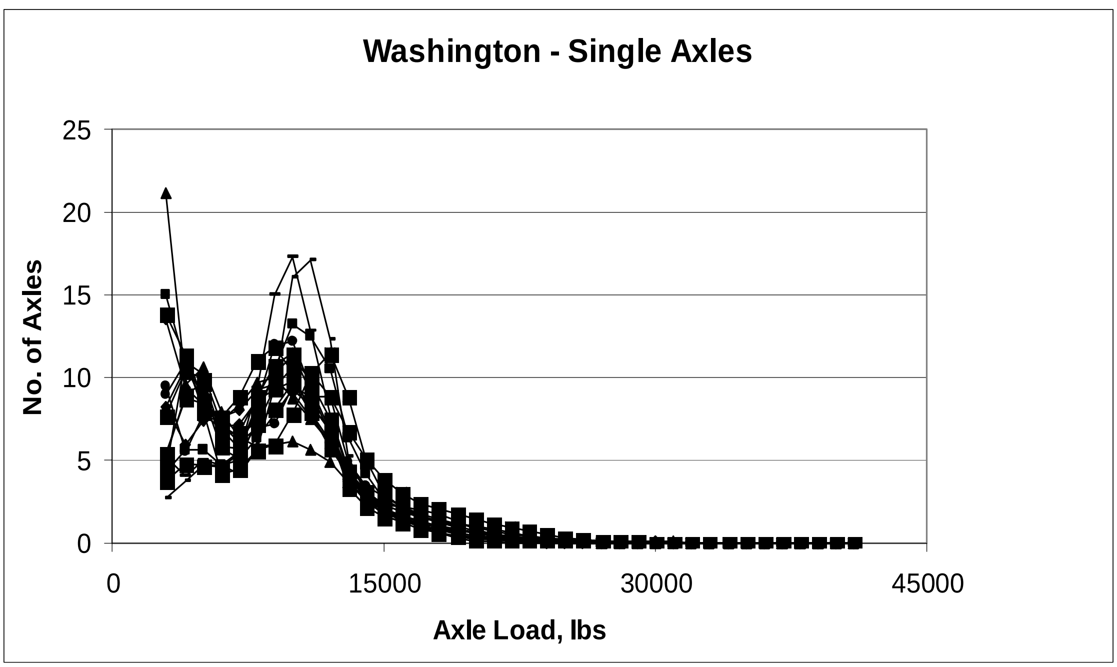


Figure 32. Axle Load Spectra for Single Axles on Test Section in Washington State

A statistical analysis was conducted to compare load spectra between years, for each axle configuration on each test section, with 4 or more years of available WIM data. Table 66 provides the number of test sections available for this comparison, for each axle configuration. The comparisons were made for each pair of years available on these test sections. For all of these comparisons, the normalized load spectra were not found to be statistically different. Therefore, once a normalized distribution is known for a single year that distribution can reasonably be assumed to represent other years on that roadway segment.

CHAPTER 3 – HOW VARIABILITY AFFECTS PAVEMENT DESIGN

This section is very brief, in that the scope of this research is to determine the variability inherent in pavement design inputs, not how these inputs are used or the implications of variability with respect to pavement design. Presented below is a brief narrative about how pavement design input parameters are normally used and why they are important to the pavement design engineer.

The interdependence of certain design input variables (such as layer thickness and modulus of elasticity) is an important consideration, as is the variability of each design input parameter as a stand-alone variable. These considerations apply equally to new and rehabilitated flexible or rigid pavements.

Typically, two pavement design input categories are regarded as very important when designing new or rehabilitated pavements to carry traffic loads:

1. ***Structural*** or ***Functional*** pavement deterioration, and
2. ***Traffic Loads*** over the design life of the pavement.

Using mechanistic or analytical-empirical pavement design and performance models, by ”structural” parameters we usually mean the stiffnesses or elastic moduli of the various pavement layers, along with their layer thicknesses, as all of these are affected by passing traffic loads. It is thought that these stiffness parameters, which can and do vary as a function of time and space (stationing), are related to the bearing capacity of a pavement section. Further, depending on the pavement's bearing capacity, structural failure will eventually ensue in the form of fatigue cracking and/or other forms of structural deterioration resulting from the accumulated traffic loads.

By “functional” pavement deterioration, we usually mean the roughness or ride quality of the pavement surface as traffic loads and the mechanistic properties of the pavement section affect the ride quality over time. Again, functional properties can vary both spatially and temporally, all of which further contribute to variability and, ultimately, pavement performance.

By “traffic loadings” we usually mean the number and configuration of passing wheel or axle loads, at various load levels, over the design life of the pavement.

Obviously, it is important to know not only average or typical design values for these various pavement parameters, but their variations as well. In fact, it can be argued that the variations in materials properties may dictate the performance of a pavement section to an even greater extent than typical design, or average, values.

This report has outlined the variability of most of the important input design variables. It is expected that this knowledge will assist the pavement engineer in designing new or rehabilitated pavements, not only considering average or even "weighted-average" design input values for a given segment of pavement, but allowing for the actual variations that can be expected in the various input design parameters used.

The following subsection (Section 4.1 – Conclusions) delineates the salient findings of this research, while also suggesting several key pavement design input variations, in terms of coefficients of variation, standard deviations, or confidence level(s) for pavement design inputs based on the research conducted.

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CHAPTER 4 — CONCLUSIONS AND SUGGESTED RESEARCH

This chapter outlines the general findings of the research, as presented in Chapter 2 under its various sections. Pavement design input parameter variability is covered in a more general sense than the findings of the LTPP data analyses themselves. A narrative on sources of variability, especially when a given variable has multiple sources of this variability, is also presented in this chapter.

4.1 SUMMARY AND CONCLUSIONS

4.1.1 Variability in Layer Thicknesses

After a thorough study of all conceivable sources of variation in layer thicknesses, as presented in Chapter 2.1, it is now possible to make recommendations to pavement designers for various levels of confidence that a given layer thickness will be at or above a desired design input thickness. While modifications to the design thickness table presented (Table 67) can be made to allow for more specific (and lower) adjustments in some cases, the changes are sufficiently minor that the table may be regarded as immediately useful without adjustment.

The 85th and 95th percentiles presented in Table 67 have been modified slightly upward from the averages of the 85th and 95th percentiles of all the sectional data. For each site, the adjustment to insure that 85% of the data at that site has an actual thickness equal to the design input is computed.

In general, the average of the individual adjustments for each site across the nation would not necessarily be the same as the one adjustment that would insure that 85% of the data for *all* sites across the nation are above the targets for all sites in the database. However, the specific adjustments that would work for the entire nation were computed for each of the 80th, 85th, 90th and 95th percentiles studied. Only for the 95th percentile was this (further) adjustment larger than the average of the 95th percentiles at each site, and then the change was only relatively minor. Accordingly, the numbers in Table 67 are usually rounded up to the nearest 0.05 inches or 1 mm.

Some types of layer thicknesses (e.g., 8" or 200 mm of AC over PCC) are not represented in the table due to insufficient data. All available data, including the smaller subsets, are shown in the complete set of tables presented in Appendix A.

Even when rounded slightly upwards, the layer thickness increases shown in Table 67 still may be unrealistically "accurate". In practice, the design engineer will have to decide whether to round up or down to a realistic, rounded value depending on the circumstances and calculations, and also depending on the units (U.S. Customary or SI) used.

The calculations to arrive at any given design pavement thickness, plus the necessary adjustment to achieve the desired confidence level, assume that there is no “bias” or conservatism in the contractor’s average layer thickness. If the contractor has been instructed to construct a particular thickness (including adjustment from Table 67), then the assumption is that the theoretical average layer thickness for the project will be the target value.

The currently popular concept of Coefficient of Variation (CV) was not used to characterize layer thickness variability, because the standard deviations for the various layer thicknesses, for each material type, were by and large _not_ directly proportional to layer thickness. Thus to utilize a CV would require a different CV for each and every conceivable layer thickness, for each material type and application. Instead, in the case of layer thicknesses it was found that the standard deviations were relatively independent of the target layer thicknesses.

Table 67. Layer Thickness Adjustments for Various Materials and Confidence Levels

Nominal or Design Thickness	Adjustment for 85th Percentile (Rounded), in.	SI Units, Unrounded Conversion, mm	Adjustment for 95th Percentile (rounded), in.	SI Units, Unrounded Conversion, mm
Unbound Base Layers:				
Nom. 4 - 14 in. GB & GS	0.55		0.85	
Nom.100 – 360 mm GB		14		22
Treated Base Layers:				
Nom. 4 - 6 in. TB & TS	0.40		0.60	
Nom. 100 – 150 mm TB		11		16
Nom. 7 - 10 in. TB & TS	0.50*		0.70*	
Nom. 180 – 250 mm TB		13*		18*
Asphalt Bound Layers:				
Nom. 4 - 7 in. ATB & AC	0.30		0.50	
Nom. 100 - 180 mm AC		8		13
Nom. 8 - 12 in. ATB & AC	0.45		0.70	
Nom. 200 - 300 mm AC		12		18
Portland Cement Concrete Layers:				
Nom. 7 - 12 in. PCC	0.40		0.65	
Nom. 180 – 300 mm PCC		11		17
Asphalt Bound Overlays:				
Nom. 2 - 5 in. AC over AC	0.35		0.55	
Nom. 50 - 130 mm AC over AC		9		14
Nom. ~4 in. AC over PCC	0.50		0.75	
Nom. ~100 mm AC over PCC		13		19
Note: *Extrapolated from available SPS data based on GPS findings.				

The variability reflected in Table 67 is in some instances somewhat better (less variation) and in other instances somewhat worse than similar data reported in the literature based on previous studies. For example, Hughes[7] reported on the variability of some of the layer thicknesses (as considered in Table 67) from several State studies. In the case of asphalt-bound thicknesses, the variability reported by Hughes was somewhat greater than was found through the LTPP layer thickness data. This is probably due to improvements in hot mix asphalt construction practices in recent years rather than specifically better pavements built under the LTPP study. The study of the Hawthorne Effect reported in Section 2.1 confirms this conclusion. Hughes also reported that the CV for asphalt-bound layers was perhaps a better indicator than standard deviations or percentiles, being the standard deviation appeared to be directly proportional to average layer thickness in most cases. In this research, we found exactly the opposite; i.e., the standard deviation for a given layer type and application increases only a minor amount as the average layer thickness increases.

4.1.2 Variability in Portland Cement Concrete Design Parameters

In Section 2.2, variability data were presented for three categories of PCC-related pavement design parameters. These categories were laboratory measured strengths, load transfer efficiency, and back-calculated moduli. The conclusions pertain to these three categories of design parameters as summarized below.

<u>Summary of Concrete Materials (Laboratory Strength) Data Analyses</u>

Data variability was studied for concrete compressive, flexural and split-tensile strengths, and for concrete modulus of elasticity compressive strength data variability.

Compressive Strength Data Variability:

The data indicate reasonably good quality control for concrete used to construct both the GPS and SPS test sections. For GPS sections, the average coefficient of variation for compressive strength was about 10%, while most of the CV values were under 15%. The CV values were similar for the 7-day and 28-day strength tests. Also, higher strength concrete (> ~5,000 psi) exhibited less variability than lower strength concrete (< ~4,000 psi). For the SPS projects, it was found that the CV values were also independent of test age (7 days, 21 days and 1 year) and specimen type (cylinders versus cores). The average coefficient of variation was about 10 to 12%, irrespective of test age. For the SPS projects, the higher strength concrete (900-psi flexural strength) was also found to be less variable than the lower strength concrete (550-psi flexural strength)

Typically, on concrete paving projects, it takes a contractor a few days to exercise good control over a new concrete mix. For the SPS-2 projects incorporating typically six 152.5-m length sections for each concrete mix and with paving spread out over several days, it would be expecting too much to have a contractor achieve a much tighter control over the two different concrete mixes. However, it appears that on most SPS-2 projects, good control was achieved with concrete production (as evidenced by low variability in cylinder strength data) and in the concrete placement process (as evidenced by the low variability in core test data).

Based on the variability data analysis, it can be concluded that on well-controlled construction projects, it would not be unreasonable to produce concrete that has a coefficient of variation of 15% or less for compressive strength.

Flexural Strength Data Variability:

Similar to compressive strength data, analysis of flexural strength variability indicates reasonably good quality control for concrete used to construct both the GPS and SPS test sections. For both the GPS and SPS test sections, the CV for the flexural strength appears to be independent of age at time of testing and has an average value of about 10%.

It should be noted that while the 550-psi concretes for the SPS-2 projects typically achieved the target value of 550-psi at 14 days; this was not the case with the 900-psi concretes, however. Many of these concretes were well below their target value of 900-psi at 14 days.

Also, at 1 year of age, both the 550- and 900-psi concretes at many of the SPS-2 projects exhibited similar flexural strength values and some 900 psi concretes did not exhibit significant increase in strength between 14 days and 1 year.

Split-Tensile Strength Data Analysis:

Only the SPS-2 data were available for analysis. These data also verified that the SPS-2 projects were reasonably well controlled. The average CV was found to be about 9 to 12% and the CV values were independent of age at testing.

Modulus of Elasticity Data Analysis:

Only the SPS-2 data were available for analysis. The average CV for the modulus of elasticity data was found to range between approximately 13% (at 28 days and at 1 year) for the 550-psi concrete, and about 12% (at 28 days), and about 11% (at 1 year) for the 900-psi concrete. However, it should be noted that there was not a significant increase in the modulus value between 28 days and 1 year.

Recommended CV Values for Concrete Strengths:

Based on the research, the CV values shown in Table 68 are recommended for concrete pavement design purposes and for construction quality management needs.

Table 68. Recommended CV Values for Concrete Strength Design Parameters

Parameter	CV, %
Compressive Strength	15
Flexural Strength	12
Split-Tensile strength	15
Modulus of Elasticity	15

Summary of PCC Load Transfer Efficiency Data Analyses

FWD deflection data across joints or cracks in the LTPP database were extracted using DataPave 2.0, and were used to calculate the Load Transfer Efficiency (LTE) across these joints or cracks. The deflection data were obtained from four LTPP experiments, GPS-3 and SPS-2 for JPCP, GPS-4 for JRCP, and GPS-5 for CRCP. For each test section, FWD tests were conducted on different numbers of joints or cracks, depending on the joint or crack spacing. The maximum number of joints or cracks tested in any pavement section was limited to 20. The tests were performed with the FWD loads applied at the approach slab and the leave slab, with an FWD deflection sensor placed on both sides of the joint tested. Further, for each test section FWD testing was conducted at several different times.

For this study, pavement test sections were classified into three groups: jointed pavements with plain (aggregate-interlock) joints, jointed pavements with doweled joints, and CRC pavements. The variability of the average LTE was analyzed for each of these three groups of test sections. The following subsections present conclusions based on the analyses conducted.

Jointed Concrete Pavements with Plain Joints:

1. The results of the t-test and F-test indicate that the average LTEs calculated from the approach slab loading data (J4) and the leave slab loading data (J5) came from different populations for 99 out of the 182 data sets (about 54%). Although the range of the variations was wide, the majority of differences between the average LTEs calculated from the J4 deflection data and those from the J5 deflection data were within ±10%. Also, the average LTE calculated using the combined data obtained under J4 and J5 loadings is typically used in concrete pavement engineering. Therefore, the average LTEs calculated using the combined data were used for further analyses in this study.
2. The variability of the average LTE, expressed in terms of the coefficient of variation (CV), was inversely correlated to the average LTE. As the average LTE increased the CV decreased, which is as expected.
3. The CV of jointed pavements with plain joints showed a wide range of variations, ranging from a low of 2% to a high of 91%, with an average value of 23% (± 3% at a 95% confidence level). However, 75% of the CVs were found to be less than 38%.
4. The average joint spacing base type and outside shoulder type did not show any effect on the variability of the average LTE.
5. The average LTEs of pavements with subsurface drainage systems showed more variability than those of pavements without subsurface drainage systems, perhaps contrary to expectations.
6. The average LTEs of pavements with a granular soil subgrade (A-1, A-2 and A-3, as classified using the AASHTO soil classification system) showed more variability than pavements with a silty-clay subgrade (A-4 to A-7).
7. The amount of annual precipitation, the number of annual freeze-thaw cycles, and the average mean annual temperature did not show any effect on the variability of the average LTE. However, the variability of the average LTE seemed to decrease as the annual freezing index increased.
8. No direct relationships between pavement age and the variability of LTE were observed. Rather, the variability of the average LTE was indirectly related to the pavement age through the changes of the average LTE. For constant values of average LTE over a period of time, the variability associated with the average LTE remained constant. As the average LTE started decreasing, the variability increased accordingly. This result was as expected.

Jointed Concrete Pavements with Doweled Joints:

1. The results of the t-test and the F-test indicated that the average LTEs calculated from the approach slab loading data (J4) and the leave slab loading data (J5) came from different populations for 202 out of the 653 data sets (about 31%). Over 94% of the data of the differences between the average LTEs calculated from the J4 deflection data and those from the J5 deflection data were within ± 10%. Also, since the average LTE calculated using the combined data obtained under the J4 and J5 loads would normally be used in concrete pavement engineering, the average LTEs calculated using the combined data were used for further analyses in this study
2. The variability of the average LTE, expressed in terms of the coefficient of variation (CV), was inversely correlated to the average LTE. As the average LTE increased, the CV decreased, as expected.
3. The CV of jointed pavements with doweled joints showed a wide range of variation, ranging from a low of 1% to a high of 97%, with an average value of 12% (± 1% at the 95% confidence level). However, 75% of the CVs were found to be less than 15% and 85% of CVs were less than 21%.
4. The average joint spacing and base type did not show any effect on the variability of the average LTE.
5. The average LTE of pavements with a concrete shoulder showed more variability than those of pavements with an asphalt shoulder.
6. The average LTE of pavements with subsurface drainage systems showed less variability than those pavements without a subsurface drainage system.
7. The amount of annual precipitation, the number of annual freeze-thaw cycles, and the average mean annual temperature did not show any effect on the variability of the average LTE. However, the variability of the average LTE seemed to decrease as the annual freezing index increased. Almost all of the CVs were less than 20% when the pavements were located in a region with an annual freezing index greater than about 600°C-day.
8. No direct relationship between pavement age and the variability of LTE were observed. Rather, the variability of the average LTE was indirectly related to the pavement age through changes in average LTE, as would be expected. For constant values of average LTE over a period of time, the variability associated with the average LTE remained constant. As the average LTE began to decrease, the variability increased accordingly.

Continuously Reinforced Concrete Pavements:

1. The results of the t-test and F-test indicate that the average LTEs calculated from the approach slab loading data (C4) and the leave slab loading data (C5) came from different populations for 78 out of the 187 data sets (about 42%). However, the range of the differences between the average LTEs calculated from the C4 deflection data and those calculated from the C5 deflection data was very small, from a low of –6% to a high of +3%, with an average value of

–1%. Therefore, the average LTEs calculated using the combined data were used for further analyses in this study.

2. No apparent relationship between the variability of the average LTE, expressed in terms of the coefficient of variation (CV), and the average LTE were observed. This might be expected, since the ranges of both the values of the average LTE and the CV were very narrow.
3. The CV of CRC pavements showed a very narrow range of variations, ranging from a low of 3% to a high of 13%, with an average value of 0.8% (± 0.2% at 95% confidence level). Furthermore, 85% of the CVs were found to be less than 4%.
4. The average crack spacing of the CRC pavement 'slabs', base type, slab stiffness, and outside shoulder type did not have any effect on the variability of the average LTE.
5. The average LTEs of pavements with subsurface drainage systems showed less variability than those of pavements without subsurface drainage systems, as expected.
6. The amount of annual precipitation did not show any effect on the variability of the average LTE. The variability of the average LTE seemed to decrease as the annual freezing index increased, as expected.
7. No relationships between the pavement age and the variability of the LTE were observed. Again, part of the reason may be the narrow ranges of the CV values observed for the CRC pavements.

LTE Comparisons Between Plain-Jointed, Doweled-Jointed, and CRC Pavements:

A summary of the variability of the average LTE calculated for the three types of pavements is presented in Table 69. As can be observed from this table, the average LTE for CRC type pavements showed the least variability, followed by pavements with doweled joints. The average LTE of the plain-jointed pavements showed the most variability.

Table 69. Summary Statistics of CV Associated with Load Transfer Efficiencies

Statistical Parameters	Plain-Jointed Pavements			Doweled-Jointed Pavements			CRC Pavements		
	Average Section LTE, %	Section Std. Deviation, %	Section CV, %	Average Section LTE, %	Section Std. Deviation, %	Section CV, %	Average Section LTE, %	Section Std. Deviation, %	Section CV, %
Mean	67.9	11.2	22.5	80.2	7.8	11.7	91.1	2.7	3.0
Maximum	96.8	34.0	90.8	97.9	39.9	96.6	100.4	9.9	12.7
Minimum	11.3	1.5	1.6	8.9	1.0	1.0	77.4	0.7	0.8
75 Percentile Point	>42.8	<17.4	<37.6	>73.6	<10.8	<14.6	>89.6	<3.2	<3.5
95% Confidence Interval for Mean	± 3.7	± 1.2	± 3.0	± 1.1	± 0.5	± 1.0	± 0.4	± 0.2	± 0.2
No. of Data Sets	182			653			187		

Summary of PCC Moduli Analyses

The spatial variability associated with the back-calculated parameters for PCC pavements was found to be relatively low considering the evolutionary nature of the PCC back-calculation analysis technology. For JPC pavements, the average variability (in terms of CV) was found to be between 11% and 15% for the four moduli parameters considered (k, E_{c-l}, E_{s-es}, E_{c-es}), while the average variability for these four parameters for CRC pavements was found to be between 12% and 18%. The spatial variability in these parameters remains fairly consistent from one testing time to another over a period of years, typically ranging within the aforementioned 18% or better CV range.

4.1.3 Variability in Asphalt Concrete Design Parameters

As reported in Section 2.3, the goal of analyzing the back-calculated and laboratory determined moduli (and the other related parameters) of AC-surfaced pavements is to determine the expected variability associated with these methods for use in the design of new asphalt pavements (or overlays) using asphalt-bound materials. Several correlations and comparisons were completed in an effort to determine if external factors outside of the back-calculation process or laboratory testing methods affect the expected variability.

In general, no strong correlations between the variability in back-calculated surface moduli and other variables could be established. The following summarizes the results of the back-calculated analysis for AC-surfaced pavements:

- There is significant variability in the standard deviation of the back-calculated moduli for AC pavements.
- The variability of the back-calculated AC modulus is independent of the drop height (or the imparted FWD load).
- It *cannot* be assumed that there is a direct correlation between the AC modulus variability and the subgrade variability [i.e., when the AC variability is high (or conversely, low) the subgrade variability may or may not be high (or conversely, low)].
- It is apparent that further study of the correlation between the AC layer modulus and AC thickness is needed. This is an extremely important consideration in the design and overall performance of asphalt pavements. Once sufficient LTPP data is available, this analysis should be continued.
- It can be reasonably concluded that the expected variability of the AC layer may in fact be different based upon the environmental region in which the pavement is constructed.
- The LTPP data suggests that the expected variability of the back-calculated subgrade modulus is independent of the environmental zone in which the pavements are located.
- It can reasonably be concluded that the expected variability of the layers typically constructed in AC-surfaced pavements will be different from one another.
- It can also be assumed that the asphalt concrete layers as well as the granular base layers will typically have lower back-calculated variability in relation to the other underlying layers.

- It has been demonstrated that the expected variability associated with the AC layer back-calculated modulus is primarily dependent upon the season in which the FWD measurements are taken.
- Most other layers (besides the AC) constructed in AC-surfaced pavements do not have any discernable patterns when it comes to the expected variability of the modulus in relation to the season in which the FWD measurements were taken.
- For the AC layer, the coefficient of variation (averaged over all FWD measurements) is statistically different (at an alpha level = 0.05) between newer pavements and older pavements.
- The magnitude of the standard deviation is directly correlated with the magnitude of the modulus (i.e., when the modulus is higher the standard deviation is typically greater). This is evidently independent of the age of the pavement.
- The variability associated with the laboratory tests of modulus of elasticity and other related parameters are less than the variability derived through back-calculation.

Recommendations for Further Study of AC-Surfaced Pavement Design Parameters

It is clear that further investigations need to be conducted to validate many of the research results presented in Chapter 2.3. While this study looked at the available data from many different perspectives, the available data and budgeting constraints did not allow other aspects to be investigated. Some of the other factors to consider in future studies should include:

- Look at smaller regional areas to account for construction practice differences when studying modulus variability.
- More detailed pavement structure definitions (e.g., thickness ranges, soil type, etc.) must be investigated.
- Subgrade condition (moisture content, compaction effort, etc.) must be included.
- Validate that the variability associated with the MODCOMP4 back-calculation program would be applicable to other back-, forward-, or iterative-calculation programs.
- Look at different groupings (e.g., group modulus values for each layer into low, medium, and high stiffness categories) to investigate variability.
- Investigate separating the SPS from the GPS pavements or investigate grouping information based on pavement age, for example.
- Try grouping the data by temperature ranges instead of environmental zones or seasons.
- Long-term variation analysis should include "correcting" the moduli to a standard temperature, or temperatures, in order to investigate the change in variability over time.

Such detailed analyses (which were not possible in this study) should shed more light on the data available and provide more insight into the variability of the back-calculated and laboratory-determined AC pavement strength parameters.

Recommendations for Use of Back-calculated Moduli Variability in AC Pavement Design

Based on the analyses conducted, a summary of the Coefficients of Variation of the various typical pavement layers in asphalt-surfaced pavements is presented in Table 70. Since these values are based on variations in the back-calculated elastic layer parameters found in the LTPP database, it is recommended that the *average* CVs recommended for design (boldfaced, in the third column from the right) are used for pavement design purposes. This recommendation is based on the fact that all values in the table also reflect the effect of several factors other than moduli, for example spatial variations in layer thickness and the unknown lack of precision resulting from the process of back-calculation itself.

It should be noted, however, that the CV values in Table 70 for layers *other* than the lower, semi-infinite subgrade and, possibly, the asphalt surface course are to a great extent influenced by the values from lower-lying layers. This is called the "compensating layer effect", whereby a small "error" or bias in the back-calculation of the subgrade modulus is "reversed" and magnified when (most) back-calculation programs derive the modulus of the second layer from the bottom, etc., on through to the surface (AC) layer.

Therefore, when using these CVs to quantify layered elastic variability based on back-calculated moduli for pavement design, it is suggested that they are not seen as independent variations in moduli, but rather as variations that influence one-another, somewhat offsetting up through the layered elastic system due to the compensating layer effect. As mentioned, probably the most reliable values in Table 70, when seen in isolation, are the average CVs of the subgrade and asphalt concrete layers.

Table 70. Recommended Back-Calculated Modulus CVs for Use in Pavement Design

			95% Confidence Interval				CVs Recommended for Pavement Design		
Layer	Range of Data	Mean	Lower	Upper	10% Quantile	90% Quantile	**Average (%)**	Lower (%)	Upper (%)
Asphalt Concrete	0.2 - 377	39.4	38.5	40.3	14.3	71.7	**39**	14	72
Granular Base	0.2 - 339	49.7	48.7	50.7	16.7	91.6	**50**	17	92
Granular Subbase	1.0 - 701	73.8	70.4	77.1	15.7	149.5	**74**	16	150
Subgrade	0.3 - 470	35.3	34.2	36.4	6.4	92.0	**35**	6	92
Treated Base	0.5 - 379	68.5	66.3	70.7	23.9	115.6	**68**	24	116
Treated Subgrade	05 - 501	90.7	85.7	95.7	29.4	157.7	**91**	30	158

4.1.4 Variability in Base, Subbase and Subgrade Parameters

In Section 2.4, Resilient Moduli data from both GPS and SPS sections were evaluated. There were 753 sections that had laboratory determined modulus values from both ends of the section. Additionally, there were 15 SPS sites that had four or more subgrade modulus tests from samples taken at the site (12 or more individual 500 ft. sections). Accordingly, SPS sites represent a greater distance between sampling locations, but like the GPS sections, these sites were selected by State and Provincial DOTs to be in areas of similar subgrade materials (a criteria for site and section selection). The results from the SPS sites fall within the range of variation seen from individual sections, the majority of which were from GPS sections.

For design purposes, the data from the remolded samples taken from the ends of the sections would best describe the variations found, and best match sampling and testing methods likely to be used for pavement design purposes. The data brings up a question regarding how much construction control can reduce modulus variability of unbound materials, and there will likely be developments in this area as emphasis switches from density and moisture control to stiffness. However, the data available is reflective of the variation in unbound materials that results from current construction practices. The CV of the unbound subgrade soils were less than 60 percent for 95 percent of the sections, below 40 percent for 85 percent of the sections, and below 17 percent for half of the sections. Thus the subgrade variation recommendations at the 50 percent confidence level shown in Table 70 (based on back-calculation) are in line with the findings of the laboratory study at about the 80 percent confidence level, which is reasonable for pavement design purposes.

Also in Section 2.4, field density and moisture variations within SPS sites were evaluated. Both the densities and moisture (based on moisture content rather than the CV of moisture) were reasonably consistent for all of the sites. The subgrade density CV values ranged from 0.5 percent to 9.8 percent, with an average CV of 3.6 percent. The standard deviation of the moisture contents ranged from 0.2 percent to 3.6 percent for the subgrade soils from the SPS sites.

The base materials were more consistent, with density CV values ranging from 0.7 to 5.2 percent and averaging 2.6 percent. The standard deviation of the moisture contents ranged from 0.1 percent to 2.1 percent, with an average standard deviation of 0.8 percent.

Since no relationship between density or moisture and stiffness could be developed for this study, the variation of the density and moisture contents is provided so it can be used to adjust stiffness variations where those relationships are known.

4.1.5 Variability in Traffic Loads

In Section 4.5, the data analyses was presented and the following general conclusions were reached, based on all LTPP traffic load spectrum data analyzed:

- In pavement design, neither the vehicle class distribution nor the axle load spectrum can be reasonably assumed using a “default” or single load distribution for either the functional class of a roadway or by state/province (or region).
- Once the vehicle class distribution and the load spectra for a roadway are known through surveys for a single year, those distributions can confidently be used to predict the distributions for other years.

In other words, it is necessary to accurately measure the number, axle loadings and axle load configurations for a given roadway segment before these can be predicted and used in pavement design with confidence. On the other hand, once these three factors are known, they can be extrapolated to other years with confidence unless the demographics or usage of the roadway changes.

4.2 SHORTCOMINGS AND SUGGESTED RESEARCH

The variables studied in this research covered a wide range of pavement design input parameters, from traffic loads on the one hand to PCC joint efficiency on the other, along with the many pavement design parameters that lie in between these extremes. Some of these variables are well defined in the LTPP database while others are not. While spatial variability was relatively easy to quantify due to the experimental nature of the LTPP program and the extensive database it encompasses, long-term variability was not (except in the case of traffic loads). A disconcerting and confounding factor was the inclusion of temporal changes that are *not* related to short-term (daily or seasonal) changes in the variables that are also clearly related to short-term changes. This can—and in fact did—severely confound the analyses of several variables, especially PCC load transfer efficiency and back-calculated moduli.

It can be argued that load transfer efficiency (LTE) is not a direct pavement design input parameter. Instead, it is more a result of the selection of a jointed concrete pavement than a PCC design variable *per se*. In other words, LTE is a result of a particular construction technique that will certainly affect long-term pavement performance, but it is not used as an input variable in the design process. A similar quantity that is more of a result than a design input is surface roughness or smoothness. Smoothness of the pavement is, initially, very important, and everyone knows that a smoothly constructed pavement generally lasts longer. But smoothness is not generally used as an input parameter, except to require a certain minimum smoothness on the part of the contractor once the pavement design has been carried out and construction is underway.

In LTPP, most of the PCC sections included in the study have not been in service long enough to even begin to quantify long-term trends in LTE, as important as these sections may be. The long-term analysis of LTE is further confounded by the fact that—even during a single day—LTE can vary widely as a result of changing temperature and moisture gradients, and the resulting curling and warping that take place. Since the FWD testing program did not generally require a specific environmental condition during the testing of joints or cracks, it was indeed quite difficult to glean much in the way of long-term trends in LTE, although spatial variability was possible (albeit in a general fashion) to define.

Figure 4 showed three reasonably sensible examples of long-term LTE, which again are few and far between in the LTPP database. In these cases, it is clear that LTE did not change rapidly over time in the way that traffic loads or PCC modulus, for example, may vary. It was also fortunate that these examples were most likely tested on days where similar temperature and moisture gradients existed in the slab from one test date to the next. These rare examples are meant to illustrate that long-term variability of LTE is "not in the cards", at least not under the present LTPP FWD testing protocols that are in place at this time.

As a much-needed research modification, the SPS-2 jointed PCC pavements should continue to be monitored with the FWD, both for LTE and other FWD tests at the various locations on the slab. However to better define the long-term variability of LTE, similar environmental conditions should be present during LTE testing with the FWD. Unfortunately, most SPS-2 pavements (where FWD deflections have been measured since construction) are still less than 10 years old, so little if any long-term changes have taken place, with few exceptions. Future testing should be modified to facilitate further analyses of long-term changes in LTE and other PCC properties.

The same long-term arguments can also be made with respect to back-calculated moduli. Here again, if one is to monitor long-term changes in the *in situ* stiffnesses, or moduli, similar temperature and moisture conditions must be present from one FWD test date to the next. Except for the short-term measurements carried out under the Seasonal Monitoring Program (SMP), such detailed FWD measurements have not been conducted.

Further LTPP or other research is needed to clarify and better quantify the above-outlined shortcoming surrounding the long-term performance of both AC and PCC pavements and their associated pavement design input parameters.

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APPENDIX A – LAYER THICKNESS – RELATED DATA TABLES

Appendix A contains the following tables and figures related to layer thickness pavement design input parameters:

Table	Title of Table (Abbreviated)
A1	Hawthorne Effect Comparison for Paired SPS-1 Projects
A2	Average Grid vs. Core Layer Thickness Differences for SPS-1 Data Sets
A3	Hawthorne Effect Comparison for Paired SPS-2 Projects
A4	Average Grid vs. Core Layer Thickness Differences for SPS-2 Data Sets
A5	Section by Section Thickness Summary Statistics for SPS-5 Projects
A6	Section by Section Thickness Summary Statistics for SPS-6 Projects
A7	Variations in Layer Thickness for 2-point GPS Data for Flexible Pavements
A8	Variations in Layer Thickness for 2-point GPS Data for Rigid Pavements

Table A1 – Section by section Hawthorne effect comparison from all paired SPS-1 projects (for new flexible pavements).

SHRP ID#	State Code	Layer Type	LTPP Material Code	Core Thickness Average Sta. 0 (in.)	Core Thickness Average Sta. 5 (in.)	Core Section Average (in.)	Core Section Standard Deviation (in.)	Notes	Grid Section Average (in.)	Grid Section Standard Deviation (in.)
Nom. 4" AC Over Weak Subbase:										
0102	10	AC	1	3.9	4	3.950	0.071		4.087	0.757
0102	12	AC	1	3.7	4.1	3.900	0.283		3.671	0.163
0102	32	AC	1	4.3	4.2	4.250	0.071		4.095	0.232
0102	35	AC	1	4.2	*(7.2)	4.200	*(2.12)	*excluded	4.122	0.641
0107	10	AC	1	5	4.4	4.700	0.424		4.807	0.430
0107	12	AC	1	3.8	4.1	3.950	0.212		3.660	0.266
0107	32	AC	1	4.4	4.5	4.450	0.071		4.187	0.258
0107	35	AC	1	5.6	6	5.800	0.283		4.316	0.582
0113	4	AC	1	4.8	4.3	4.550	0.354		4.225	0.428
0113	51	AC	1	3.8	4	3.900	0.141		3.955	0.433
0120	4	AC	1	4	4	4.000	0.000		4.065	0.343
0120	51	AC	1	4.1	3.7	3.900	0.283		4.145	0.269
0121	4	AC	1	4.3	3.9	4.100	0.283		4.159	0.268
0121	51	AC	1	3.5	3.7	3.600	0.141		3.782	0.499
Nom. 7" AC Over Weak Subbase:										
0101	10	AC	1	7.1	7.4	7.250	0.212		6.922	0.375
0101	12	AC	1	6.8	6.8	6.800	0.000		6.620	0.150
0101	32	AC	1	7	7.2	7.100	0.141		7.193	0.432
0108	10	AC	1	7.3	7.3	7.300	0.000		6.998	0.265
0108	12	AC	1	6.6	6.3	6.450	0.212		6.376	0.232
0108	32	AC	1	7	7.2	7.100	0.141		6.985	0.286
0108	35	AC	1	7.2	7.5	7.350	0.212		6.833	0.265
0109	10	AC	1	7.5	7.8	7.650	0.212		7.325	0.226
0109	12	AC	1	7.3	7	7.150	0.212		7.138	0.148
0109	32	AC	1	7	7.2	7.100	0.141		6.989	0.303
0109	35	AC	1	7.6	7.5	7.550	0.071		7.169	0.350
0114	4	AC	1	6.8	6.6	6.700	0.141		7.131	0.836
0114	51	AC	1	5.8	5.3	5.550	0.354		5.555	0.386
0119	4	AC	1	6.3	6.5	6.400	0.141		6.178	0.333
0119	51	AC	1	6.5	6.4	6.450	0.071	Count =	6.398	0.454
Averages						**5.626**	**0.174**	**29**	**5.486**	**0.366**
Nom. 4" AC Over Strong Subbase:										
0103	10	AC	1	4	4.1	4.05	0.07		4.76	0.48
0103	12	AC	1	4	4.3	4.15	0.21		3.99	0.18
0103	32	AC	1	4.1	4.1	4.1	0.00		4.17	0.11
0103	35	AC	1	4.4	4.4	4.4	0.00		5.18	0.24
0105	10	AC	1	3.9	4.2	4.05	0.21		4.44	0.62
0105	12	AC	1	3.6	4	3.8	0.28		3.81	0.19

0105	32	AC	1	4	4.3	4.15	0.21		4.16	0.22
0105	35	AC	1	5.3	5.3	5.3	0.00		4.67	0.48
0111	10	AC	1	3.7	3.8	3.75	0.07		3.73	0.37
0111	12	AC	1	3.9	3.9	3.9	0.00		3.91	0.25
0111	32	AC	1	4.3	4.1	4.2	0.14		4.13	0.13
0111	35	AC	1	4.4	4.2	4.3	0.14		4.25	0.22
0112	10	AC	1	4.9	4.5	4.7	0.28		4.51	0.39
0112	12	AC	1	4.1	3.8	3.95	0.21		3.97	0.29
0112	32	AC	1	4.6	4.4	4.5	0.14		4.24	0.20
0112	35	AC	1	4.2	4.5	4.35	0.21		4.40	0.26
0116	4	AC	1	4.1	4.2	4.15	0.07		3.74	0.24
0116	51	AC	1	4.6	4.6	4.6	0.00		4.43	0.38
0118	4	AC	1	4.1	4	4.05	0.07		3.74	0.26
0118	51	AC	1	4	4.2	4.1	0.14		4.13	0.28
0122	4	AC	1	4.2	4.3	4.25	0.07		3.95	0.32
0122	51	AC	1	4.2	3.9	4.05	0.21		3.85	0.39
Nom. 7" AC Over Strong Subbase:										
0104	10	AC	1	7.1	6.2	6.65	0.64		6.73	0.22
0104	12	AC	1	6.9	6.7	6.8	0.14		6.89	0.16
0104	32	AC	1	7.3	7.2	7.25	0.07		7.31	0.28
0104	35	AC	1	6.5	8.2	7.35	1.20		7.76	0.18
0106	10	AC	1	7.5	7.3	7.4	0.14		6.87	0.31
0106	12	AC	1	7	7.3	7.15	0.21		7.02	0.24
0106	32	AC	1	7.3	7.1	7.2	0.14		7.18	0.18
0106	35	AC	1	6.2	7.2	6.7	0.71		7.59	0.32
0110	10	AC	1	6.4	6.8	6.6	0.28		7.21	0.45
0110	12	AC	1	7.1	7.4	7.25	0.21		6.98	0.23
0110	32	AC	1	6.6	6.6	6.6	0.00		7.05	0.25
0110	35	AC	1	7.6	7.2	7.4	0.28		7.05	0.23
0115	4	AC	1	6.8	6.5	6.65	0.21		6.44	0.20
0115	51	AC	1	6.4	6.5	6.45	0.07		6.35	0.50
0117	4	AC	1	7.4	7.6	7.5	0.14		7.10	0.30
0117	51	AC	1	6.8	6.8	6.8	0.00		6.65	0.48
0123	4	AC	1	6.9	6.8	6.85	0.07		6.68	0.18
0123	51	AC	1	6.8	6.7	6.75	0.07		6.54	0.53
0124	4	AC	1	6.8	6.6	6.7	0.14		6.84	0.26
0124	51	AC	1	6.4	6.3	6.35	0.07	Count =	6.25	0.53
Averages					**5.506**	**5.506**	**0.180**	**42**	**5.492**	**0.298**
Nom. 4" ATB Over Weak Subbase:										
0105	10	ATB	319	3.7	4.2	3.95	0.35		4.38	0.37
0105	12	ATB	319	4	4	4	0.00		4.06	0.19
0105	32	ATB	319	5.3	4.7	5	0.42		4.85	0.21
0105	35	ATB	319	4	4.2	4.1	0.14		3.84	0.36
0110	10	ATB	319	5.4	5.3	5.35	0.07		4.11	0.33
0110	12	ATB	319	4.2	4.1	4.15	0.07		3.87	0.29
0110	32	ATB	319	4.1	4.3	4.2	0.14		3.98	0.29
0110	35	ATB	319	4.5	4.4	4.45	0.07		4.79	0.22
0117	4	ATB	319	4.3	3.8	4.05	0.35		3.93	0.35
0117	51	ATB	319	4.5	4	4.25	0.35		4.01	0.33
0122	51	ATB	319	4.1	4	4.05	0.07		3.86	0.41
Nom. 8" ATB Over Weak Subbase:										

0103	10	ATB	319	7.9	8.7	8.3	0.57		7.98	0.39
0103	12	ATB	319	8.1	7.9	8	0.14		8.01	0.21
0103	32	ATB	319	8.7	9	8.85	0.21		8.61	0.27
0103	35	ATB	319	7	8	7.5	0.71		6.65	0.50
0106	10	ATB	319	8.2	8	8.1	0.14		8.52	0.51
0106	12	ATB	319	8.3	8.5	8.4	0.14		8.04	0.25
0106	32	ATB	319	8.8	8.8	8.8	0.00		8.79	0.38
0106	35	ATB	319	7.6	8.6	8.1	0.71		7.86	0.54
0111	10	ATB	319	9.4	9.6	9.5	0.14		8.74	0.40
0111	12	ATB	319	8.4	8.1	8.25	0.21		7.84	0.21
0111	32	ATB	319	8.5	8.2	8.35	0.21		8.37	0.57
0111	35	ATB	319	7.4	7.2	7.3	0.14		8.02	0.28
0115	51	ATB	319	8.5	8.9	8.7	0.28		8.58	0.34
0118	4	ATB	319	7.8	7.7	7.75	0.07		7.72	0.22
0118	51	ATB	319	8	8	8	0.00		8.02	0.43
0123	4	ATB	319	8.1	7.9	8	0.14		7.82	0.27
0123	51	ATB	319	8.5	8.1	8.3	0.28		8.06	0.33
Nom. 12" ATB Over Weak Subbase:										
0104	10	ATB	319	12.4	11.3	11.9	0.78		12.05	0.36
0104	12	ATB	319	12.2	11.9	12.1	0.21		12.15	0.24
0104	32	ATB	319	12	12.7	12.4	0.49		12.45	0.36
0104	35	ATB	319	9.5	12	10.8	1.77		11.70	0.97
0112	10	ATB	319	12.9	12.3	12.6	0.42		12.33	0.45
0112	12	ATB	319	12.5	12.2	12.4	0.21		11.85	0.24
0112	35	ATB	319	11.2	12.2	11.7	0.71		11.49	0.46
0116	4	ATB	319	12.5	11.9	12.2	0.42		11.78	0.42
0116	51	ATB	319	14.4	14.4	14.4	0.00		12.39	0.44
0124	4	ATB	319	11.7	12	11.9	0.21		11.54	0.28
0124	51	ATB	319	12.5	12	12.3	0.35	Count =	12.47	0.61
Averages						**8.259**	**0.301**	**39**	**8.090**	**0.366**
Nom. 4" PATB Over Weak Subbase:										
0107	10	PATB	325	4.1	4.3	4.200	0.141		3.760	0.420
0107	35	PATB	325	3.7	3.9	3.800	0.141		4.236	0.287
0108	10	PATB	325	3.6	3.7	3.650	0.071		3.711	0.391
0108	35	PATB	325	*(6.6)	*(4.0)	*(5.3)	*(1.84)	*excluded	*(0.28)	*(4.35)
0109	10	PATB	325	4	4	4.000	0.000		4.224	0.301
0109	35	PATB	325	4.6	4.8	4.700	0.141		3.951	0.382
0110	10	PATB	325	3	3.1	3.050	0.071		3.640	0.363
0111	10	PATB	325	3.4	4	3.700	0.424		3.911	0.393
0111	35	PATB	325	3.4	3.9	3.650	0.354		3.736	0.468
0112	35	PATB	325		*(2.8)	*(2.8)		*excluded	*(3.41)	*(0.40)
0119	51	PATB	325	4.4	4.3	4.350	0.071		4.431	0.313
0120	51	PATB	325	3.9	4.3	4.100	0.283		4.333	0.437
0121	51	PATB	325	4.3	4.3	4.300	0.000		4.311	0.285
0122	51	PATB	325	4	4	4.000	0.000		3.945	0.325
0124	51	PATB	325	3.5	3	3.250	0.354	Count =	3.371	0.852
Averages						**3.904**	**0.158**	**13**	**3.966**	**0.401**

Table A2 – Summary of average grid vs. core layer thicknesses differences between two SPS-1 data sets.

Description of Layer	Grid Average Thickness	Core Average Thickness	Difference (Core-Grid Thickness)	Value of T-statistic	Number of Sections	P-value for two-tailed test	Significant at level 0.05?
4 inch AC over weak subbase (1)	4.091	4.232	0.141	1.227	14	0.241	↓
7 inch AC over weak subbase	6.787	6.927	0.139	2.353	15	0.034	↓
Combined 4" and 7" AC over weak subbase (1)	5.486	5.626	0.140	2.254	29	0.032	Yes
4 inch AC over strong base	4.189	4.220	0.031	0.448	22	0.659	↓
7 inch AC over strong base	6.924	6.920	-0.004	-0.055	20	0.957	↓
Combined 4" and 7" AC over strong subbase	5.492	5.506	0.014	0.274	42	0.786	No
4 inch ATB	4.154	4.323	0.169	1.300	11	0.223	↓
8 inch ATB	8.095	8.247	0.152	1.684	17	0.112	↓
12 inch ATB	12.016	12.214	0.197	0.902	11	0.388	↓
Combined 4", 8" and 12" ATB	8.090	8.259	0.169	2.134	39	0.039	Yes
4 inch PATB (2)	3.966	3.904	-0.062	-0.651	13	0.843	No

Notes:

1) Excludes Station 5+00 from nominal 4" Section 350102 where an extreme outlier exists (~7"). Evidently, the transition zone to a thicker section begins at grid Sta.5, which was also much thicker than the rest of the section (~5.5").
2) Excludes Sections 350108 and 350112 where there was either no data from one end of the test section, or one set of end-point data appeared to be in a transition zone.

The conclusion from the above table is:

Although most of the smaller subsets of AC or ATB over "weak" base show a two-tailed test result >0.05, when these two materials are (individually) combined for all thicknesses, there is in fact a statistically significant difference for either type of asphalt-bound layer constructed over weak subbase. On the other hand, there is no significant difference in AC layer thicknesses in any of the cases studied over “strong” or bound subbases, nor for PATB (which is not compacted under construction).

Table A3 – Section by section Hawthorne effect comparison from all paired SPS-2 projects (for new rigid pavements).

SHRP ID#	State Code	Layer Type	LTPP Material Code	Core Thickness Average Sta. 0 (in.)	Core Thickness Average Sta. 5 (in.)	Core Section Standard Deviation (in.)	Grid Section Average (in.)	Grid Section Standard Deviation (in.)	Core Section Average (in.)	Core Section Standard Deviation (in.)
Nom. 8" PCC Over Weak Base:										
0201	10	PC	4	8.6	8.5	0.071	8.338	0.386	8.550	0.071
0201	37	PC	4	10.3	8.3	1.414	9.004	0.544	9.300	1.414
0201	53	PC	4	9.5	9	0.354	8.662	0.534	9.250	0.354
0202	10	PC	4	8.5	8.5	0.000	8.765	0.372	8.500	0.000
0202	20	PC	4	7.9	7.7	0.141	7.425	0.164	7.800	0.141
0202	37	PC	4	10.3	8.3	1.414	10.153	0.838	9.300	1.414
0202	53	PC	4	8.6	8.6	0.000	8.285	0.311	8.600	0.000
0209	10	PC	4	8.7	8.2	0.354	8.216	0.414	8.450	0.354
0209	37	PC	4	8.9	10	0.778	8.627	0.655	9.450	0.778
0209	53	PC	4	9.4	9.5	0.071	8.496	0.340	9.450	0.071
0210	10	PC	4	7.9	8.4	0.354	8.296	0.558	8.150	0.354
0210	37	PC	4	11	9.8	0.849	8.456	0.752	10.400	0.849
0210	53	PC	4	8.6	9	0.283	8.293	0.325	8.800	0.283
0213	4	PC	4	7.6	8	0.283	8.017	0.263	7.800	0.283
0213	5	PC	4	6.8	8.5	1.202	7.360	0.538	7.650	1.202
0213	8	PC	4	8.8	8.5	0.212	8.709	0.322	8.650	0.212
0213	38	PC	4	7.7	8.3	0.424	7.997	0.198	8.000	0.424
0214	5	PC	4	8	8.7	0.495	7.738	0.378	8.350	0.495
0214	8	PC	4	8.3	8.5	0.141	8.356	0.166	8.400	0.141
0214	38	PC		7.8	7.9	0.071	8.120	0.202	7.850	0.071
0221	4	PC	4	8.3	8.4	0.071	7.757	0.265	8.350	0.071
0221	5	PC	4	8.5	8	0.354	7.684	0.419	8.250	0.354
0221	8	PC	4	7.5	8.8	0.919	8.628	0.508	8.150	0.919
0222	4	PC	4	8.3	8.9	0.424	8.540	0.339	8.600	0.424
0222	8	PC	4	8.5	9	0.354	8.555	0.182	8.750	0.354
Averages					**8.592**	**0.441**	**8.339**	**0.399**	**8.592**	**0.441**
Nom. 8" PCC Over Strong Base:										
0205	10	PC	4	8.6	8.4	0.141	9.176	0.424	8.500	0.141
0205	37	PC	4	8.7	8.5	0.141	8.369	0.501	8.600	0.141
0205	53	PC	4	8.5	8.5	0.000	8.413	0.173	8.500	0.000
0206	10	PC	4	8.6	8.6	0.000	8.931	0.338	8.600	0.000
0206	20	PC	4	7.8	7.5	0.212	7.933	0.258	7.650	0.212
0206	37	PC	4	8.9	8.9	0.000	8.398	0.407	8.900	0.000
0206	53	PC	4	8.6	8.5	0.071	8.627	0.212	8.550	0.071
0217	4	PC	4	7.9	8.2	0.212	8.207	0.298	8.050	0.212
0217	5	PC	4	7.6	7.5	0.071	7.225	0.265	7.550	0.071
0217	19	PC	4	8.2	7.9	0.212	8.067	0.296	8.050	0.212
0217	38	PC	4	7.7	8	0.212	7.815	0.136	7.850	0.212

0218	4	PC	4	8.2	8.2	0.000	8.390	0.173	8.200	0.000
0218	5	PC	4	7.4	7.4	0.000	7.818	0.202	7.400	0.000
0218	8	PC	4	7.9	7.5	0.283	7.716	0.296	7.700	0.283
0218	38	PC		7.7	8.2	0.354	7.910	0.205	7.950	0.354
Averages					**8.137**	**0.127**	**8.200**	**0.279**	**8.137**	**0.127**
Nom. 11" PCC Over Weak Base:										
0203	10	PC	4	11.6	11.6	0.000	11.736	0.520	11.600	0.000
0203	20	PC	4	11	11.4	0.283	11.051	0.185	11.200	0.283
0203	37	PC	4	10.9	12.1	0.849	11.124	0.563	11.500	0.849
0203	53	PC	4	11.8	11.5	0.212	10.978	0.360	11.650	0.212
0204	10	PC	4	10.6	10.4	0.141	11.016	0.438	10.500	0.141
0204	20	PC	4	11.4	11.4	0.000	11.260	0.192	11.400	0.000
0204	37	PC	4	11.4	11.3	0.071	11.175	0.233	11.350	0.071
0204	53	PC	4	11.7	11.4	0.212	11.105	0.397	11.550	0.212
0211	10	PC	4	11.6	11.5	0.071	11.751	0.315	11.550	0.071
0211	37	PC	4	11.4	12.2	0.566	11.451	0.445	11.800	0.566
0211	53	PC	4	12	12.2	0.141	11.282	0.313	12.100	0.141
0212	10	PC	4	11.8	11.5	0.212	12.405	1.178	11.650	0.212
0212	37	PC	4	11.9	10.2	1.202	10.960	0.315	11.050	1.202
0212	53	PC	4	12	11.5	0.354	10.911	0.336	11.750	0.354
0215	4	PC	4	11	10.6	0.283	11.310	0.245	10.800	0.283
0215	5	PC	4	11.5	11	0.354	10.805	0.486	11.250	0.354
0215	8	PC	4	11.4	11.5	0.071	11.407	0.208	11.450	0.071
0215	38	PC	4	11.2	11.1	0.071	10.992	0.217	11.150	0.071
0216	4	PC	4	11.3	11.2	0.071	11.147	0.172	11.250	0.071
0216	5	PC	4	11.1	10.8	0.212	10.485	0.358	10.950	0.212
0216	8	PC	4	12	11.9	0.071	11.616	0.207	11.950	0.071
0216	19	PC	4	12	12	0.000	11.420	0.214	12.000	0.000
0216	38	PC	4	11.4	11.3	0.071	11.220	0.222	11.350	0.071
0219	4	PC	4	10.5	11	0.354	10.970	0.245	10.750	0.354
0223	4	PC	4	11.4	11	0.283	10.973	0.311	11.200	0.283
0223	5	PC	4	10.9	10.8	0.071	10.595	0.382	10.850	0.071
0223	8	PC	4	11.8	12	0.141	11.722	0.269	11.900	0.141
0224	4	PC	4	10.5	10.8	0.212	10.623	0.305	10.650	0.212
0224	5	PC	4	10.7	11	0.212	10.975	0.405	10.850	0.212
0224	8	PC	4	11.3	12.5	0.849	11.282	0.840	11.900	0.849
Averages					**11.363**	**0.255**	**11.192**	**0.363**	**11.363**	**0.255**
Nom. 11" PCC Over Strong Base:										
0207	10	PC	4	11.6	11.4	0.141	11.335	0.303	11.500	0.141
0207	20	PC	4	10.7	10.7	0.000	11.285	0.288	10.700	0.000
0207	37	PC	4	11.6	11.8	0.141	11.633	0.387	11.700	0.141
0207	53	PC	4	10.8	11.3	0.354	11.124	0.257	11.050	0.354
0208	10	PC	4	11.8	11.2	0.424	12.062	0.414	11.500	0.424
0208	20	PC	4	10.8	10.5	0.212	11.029	0.299	10.650	0.212
0208	37	PC	4	11.1	11.3	0.141	11.125	0.314	11.200	0.141
0208	53	PC	4	11.4	11.5	0.071	10.724	0.298	11.450	0.071
0219	5	PC	4	10.9	11.2	0.212	10.575	0.308	11.050	0.212
0219	8	PC	4	10.5	11.3	0.566	11.564	0.315	10.900	0.566
0219	19	PC	4	11.5	11.6	0.071	11.495	0.270	11.550	0.071
0219	38	PC	4	10.8	10.8	0.000	10.873	0.157	10.800	0.000
0220	4	PC	4	11.4	10.9	0.354	11.483	0.339	11.150	0.354

0220	5	PC	4	10.8	10.6	0.141	10.433	0.450	10.700	0.141
0220	8	PC	4	10.9	11	0.071	11.331	0.218	10.950	0.071
0220	19	PC	4	11	11	0.000	11.547	0.149	11.000	0.000
0220	38	PC	4	11.1	10.8	0.212	10.988	0.218	10.950	0.212
Averages					**11.106**	**0.183**	**11.212**	**0.293**	**11.106**	**0.183**
Nom. 6" LCB Over Weak Base:										
0205	10	TB	334	6.3	5.9	0.283	5.478	0.363	6.100	0.283
0205	37	TB	334	6.6	7.8	0.849	6.531	0.585	7.200	0.849
0205	53	TB	334	6.7	6.1	0.424	6.089	0.463	6.400	0.424
0206	10	TB	334	6.1	6.1	0.000	6.107	0.421	6.100	0.000
0206	37	TB	334	6.8	6.4	0.283	6.776	0.349	6.600	0.283
0206	53	TB	334	6.4	6.1	0.212	6.180	0.321	6.250	0.212
0207	10	TB	334	6.1	6.1	0.000	6.865	0.477	6.100	0.000
0207	53	TB	334	5.8	5.3	0.354	6.056	0.443	5.550	0.354
0208	10	TB	334	5.5	6.2	0.495	5.984	0.522	5.850	0.495
0208	37	TB	334	6	6	0.000	5.842	0.319	6.000	0.000
0208	53	TB	334	6.8	6.8	0.000	6.238	0.340	6.800	0.000
0217	4	TB	334	6.2	6	0.141	5.877	0.150	6.100	0.141
0217	19	TB	334	5.8	6.6	0.566	6.380	0.359	6.200	0.566
0217	38	TB		6.6	6.3	0.212	6.488	0.229	6.450	0.212
0218	8	TB	334	6.1	6.1	0.000	6.473	0.273	6.100	0.000
0218	19	TB	334	5.7	6.8	0.778	6.216	0.265	6.250	0.778
0218	38	TB		6.4	6.5	0.071	6.588	0.235	6.450	0.071
0219	4	TB	334	6.4	6	0.283	6.067	0.188	6.200	0.283
0219	8	TB	334	6	6	0.000	6.300	0.496	6.000	0.000
0219	38	TB		6.3	6.3	0.000	6.332	0.259	6.300	0.000
0220	8	TB	334	6.5	6	0.354	6.296	0.257	6.250	0.354
0220	19	TB	334	7	7	0.000	6.522	0.235	7.000	0.000
Averages					**6.284**	**0.241**	**6.258**	**0.343**	**6.284**	**0.241**

Table A4 – Summary of average grid vs. core layer thicknesses differences between two SPS-2 data sets.

Description of Layer	Grid Average Thickness	Core Average Thickness	Difference (Core-Grid Thickness)	Value of T-statistic	Number of Sections	P-value for two-tailed test	Significant at level 0.05?
8 inch PCC over weak subbase	8.339	8.592	0.253	2.318	25	0.029	Yes
8 inch PCC over strong subbase	8.200	8.137	-0.063	-0.819	15	0.427	No
11 inch PCC over weak subbase	11.193	11.363	0.172	2.487	30	0.019	Yes
11 inch PCC over strong subbase	11.212	11.106	-0.106	-1.111	17	0.283	No
6 inch lean concrete base	6.258	6.284	0.026	0.333	22	0.743	No

Notes:

None

The conclusion from the above table is:

There is a significant difference in PCC layer thicknesses in both cases where the PCC layer was constructed over "weak" or unbound subbase. On the other hand, there is no significant difference in PCC layer thicknesses in either of the cases studied, over “strong” or bound subbases, nor for LCB (which evidently is not "fluid" enough to seep into the unbound subbase below).

Table A5 – Section by section thickness summary statistics from all available SPS-5 projects (flexible pavement with AC overlays).

SHRP ID#	State Code	Section Average (in.)	Section Standard Deviation (in.)	Degrees of Freedom	80th Percentile Deviation from Mean (in.)	85th Percentile Deviation from Mean (in.)	90th percentile deviation from mean (in.)	95th Percentile Deviation from Mean (in.)	Coefficient of Variation
Nominal 2" Overlays:									
0502	30	1.069	0.168	50	0.143	0.176	0.218	0.282	15.7%
0509	30	1.095	0.216	50	0.183	0.226	0.280	0.362	19.7%
0506	30	1.102	0.293	50	0.249	0.307	0.380	0.491	26.6%
0502	1	1.304	0.204	50	0.173	0.214	0.265	0.342	15.7%
0505	1	1.375	0.313	50	0.265	0.327	0.406	0.524	22.7%
0505	30	1.438	0.342	50	0.291	0.359	0.445	0.574	23.8%
0509	34	1.602	0.230	50	0.195	0.241	0.299	0.386	14.4%
0502	34	1.691	0.459	50	0.389	0.480	0.596	0.769	27.1%
0505	40	1.693	0.271	50	0.230	0.284	0.352	0.455	16.0%
0509	1	1.753	0.157	50	0.134	0.165	0.205	0.264	9.0%
0509	24	1.762	0.143	50	0.121	0.150	0.186	0.240	8.1%
0502	24	1.795	0.150	50	0.127	0.157	0.195	0.251	8.3%
0502	40	1.820	0.308	50	0.262	0.323	0.400	0.516	16.9%
0505	24	1.902	0.153	50	0.130	0.160	0.199	0.257	8.1%
0506	34	1.933	0.236	50	0.201	0.247	0.307	0.396	12.2%
0505	12	1.955	0.144	50	0.123	0.151	0.188	0.242	7.4%
0502	12	1.958	0.313	50	0.266	0.328	0.406	0.524	16.0%
0506	1	2.051	0.194	50	0.165	0.203	0.252	0.325	9.5%
0509	23	2.055	0.192	50	0.163	0.201	0.250	0.322	9.4%
0505	34	2.075	0.306	50	0.260	0.321	0.398	0.513	14.8%
A509	48	2.129	0.321	49	0.273	0.337	0.417	0.539	15.1%
0506	23	2.138	0.236	50	0.200	0.247	0.307	0.396	11.0%
0506	24	2.184	0.175	50	0.149	0.184	0.228	0.294	8.0%
A502	48	2.293	0.194	50	0.165	0.203	0.252	0.325	8.5%
A506	48	2.307	0.237	50	0.201	0.248	0.308	0.397	10.3%
A505	48	2.329	0.343	50	0.291	0.359	0.446	0.575	14.7%
0509	12	2.545	0.197	50	0.167	0.206	0.256	0.330	7.7%
0506	12	2.569	0.142	50	0.121	0.149	0.185	0.239	5.5%
0509	8	2.578	0.362	50	0.307	0.379	0.470	0.606	14.0%
0505	23	2.731	0.185	50	0.157	0.194	0.240	0.310	6.8%
0506	35	2.800	0.144	50	0.122	0.151	0.187	0.241	5.1%
0501	35	2.844	0.349	50	0.296	0.365	0.453	0.585	12.3%
0506	8	2.885	0.436	44	0.371	0.458	0.568	0.733	15.1%
0505	8	3.028	0.266	48	0.226	0.278	0.345	0.446	8.8%
0509	40	3.073	0.306	50	0.260	0.320	0.397	0.513	10.0%
0502	35	3.142	0.244	50	0.207	0.256	0.317	0.409	7.8%
0509	35	3.156	0.263	50	0.223	0.275	0.341	0.441	8.3%
0505	35	3.298	0.411	50	0.349	0.431	0.534	0.689	12.5%
0502	8	3.487	0.220	50	0.187	0.230	0.286	0.369	6.3%
0502	23	3.573	0.235	50	0.200	0.246	0.305	0.394	6.6%

0506	40	3.891	0.491	50	0.416	0.514	0.637	0.822	12.6%
Averages		**2.254**	**0.257**		**0.218**	**0.270**	**0.334**	**0.431**	**12.4%**
Nominal 5" Overlays:									
0503	1	3.953	0.336	50	0.285	0.352	0.436	0.563	8.5%
0504	1	4.313	0.380	50	0.322	0.397	0.493	0.636	8.8%
0507	1	4.835	0.262	50	0.222	0.274	0.340	0.439	5.4%
0508	1	4.484	0.149	50	0.127	0.156	0.194	0.250	3.3%
0503	8	5.296	0.384	50	0.326	0.402	0.498	0.643	7.2%
0504	8	6.073	0.518	50	0.439	0.542	0.672	0.867	8.5%
0507	8	5.880	0.462	50	0.392	0.484	0.600	0.775	7.9%
0508	8	5.267	0.317	49	0.269	0.332	0.412	0.532	6.0%
0503	12	5.295	0.266	50	0.226	0.279	0.345	0.446	5.0%
0504	12	4.825	0.259	50	0.220	0.272	0.337	0.435	5.4%
0507	12	4.380	0.223	50	0.189	0.234	0.290	0.374	5.1%
0508	12	4.444	0.239	50	0.203	0.250	0.311	0.401	5.4%
0503	23	5.511	0.187	50	0.159	0.196	0.243	0.314	3.4%
0504	23	5.658	0.258	50	0.219	0.270	0.335	0.432	4.6%
0507	23	5.222	0.242	50	0.205	0.253	0.314	0.405	4.6%
0508	23	4.765	0.202	50	0.172	0.212	0.263	0.339	4.2%
0503	24	4.900	0.265	50	0.225	0.278	0.344	0.444	5.4%
0504	24	4.551	0.263	50	0.223	0.275	0.341	0.441	5.8%
0507	24	4.233	0.245	50	0.208	0.257	0.318	0.411	5.8%
0508	24	5.473	0.237	50	0.201	0.249	0.308	0.398	4.3%
0503	30	3.962	0.303	50	0.258	0.318	0.394	0.508	7.7%
0504	30	3.607	0.264	50	0.224	0.277	0.343	0.442	7.3%
0507	30	3.755	0.275	50	0.234	0.288	0.358	0.461	7.3%
0508	30	3.495	0.324	49	0.275	0.340	0.422	0.544	9.3%
0503	34	4.505	0.310	50	0.263	0.324	0.402	0.519	6.9%
0504	34	4.660	0.274	50	0.233	0.287	0.356	0.459	5.9%
0507	34	2.220	0.225	50	0.191	0.235	0.292	0.376	10.1%
0508	34	4.862	0.461	50	0.391	0.483	0.599	0.773	9.5%
0503	35	5.315	0.229	50	0.195	0.240	0.298	0.385	4.3%
0504	35	5.220	0.269	50	0.228	0.282	0.349	0.450	5.1%
0507	35	6.438	0.245	50	0.208	0.257	0.319	0.411	3.8%
0508	35	6.002	0.234	50	0.199	0.246	0.305	0.393	3.9%
0503	40	4.416	0.356	50	0.302	0.373	0.463	0.597	8.1%
0504	40	4.685	0.483	50	0.410	0.506	0.627	0.809	10.3%
0507	40	6.398	0.431	50	0.366	0.451	0.559	0.722	6.7%
0508	40	5.735	0.487	50	0.413	0.510	0.632	0.816	8.5%
A503	48	4.851	0.251	49	0.213	0.263	0.326	0.421	5.2%
A504	48	4.815	0.315	50	0.267	0.330	0.409	0.528	6.5%
A507	48	4.705	0.271	50	0.230	0.284	0.352	0.454	5.8%
A508	48	4.556	0.272	50	0.231	0.284	0.353	0.455	6.0%
Averages		**4.839**	**0.299**		**0.254**	**0.314**	**0.389**	**0.502**	**6.3%**

Table A6 – Section by section thickness summary statistics from all available SPS-6 projects (rigid pavements with AC overlays).

SHRP ID#	State Code	Section Average (in.)	Section Standard Deviation (in.)	Degrees of Freedom	80th Percentile Deviation from Mean (in.)	85th Percentile Deviation from Mean (in.)	90th percentile deviation from mean (in.)	95th Percentile Deviation from Mean (in.)	Coefficient of Variation
Nominal 4" Overlays:									
0603	17	4.064	0.302	54	0.256	0.316	0.392	0.505	7.4%
0603	29	3.871	0.188	54	0.160	0.197	0.244	0.315	4.9%
0603	40	3.920	0.201	54	0.171	0.211	0.261	0.337	5.1%
0603	42	3.795	0.221	54	0.187	0.231	0.286	0.369	5.8%
0604	17	4.205	0.391	54	0.332	0.409	0.507	0.654	9.3%
0604	29	3.816	0.303	49	0.257	0.318	0.394	0.508	7.9%
0604	40	4.018	0.219	54	0.185	0.229	0.284	0.366	5.4%
0604	42	3.956	0.974	54	0.826	1.019	1.264	1.630	24.6%
0606	17	3.682	0.424	54	0.360	0.443	0.550	0.709	11.5%
0606	29	3.569	0.268	54	0.227	0.280	0.348	0.448	7.5%
0606	40	4.082	0.244	54	0.207	0.255	0.316	0.407	6.0%
0606	42	4.309	0.309	54	0.262	0.324	0.401	0.517	7.2%
0607	17	3.607	0.399	54	0.339	0.418	0.518	0.668	11.1%
0607	40	4.516	0.469	54	0.398	0.491	0.608	0.785	10.4%
0607	42	4.149	0.401	54	0.340	0.419	0.520	0.671	9.7%
A603	5	4.825	0.581	54	0.493	0.608	0.754	0.972	12.0%
A604	5	4.825	0.581	54	0.493	0.608	0.754	0.972	12.0%
A606	5	5.071	0.323	54	0.274	0.338	0.419	0.540	6.4%
A607	5	4.858	0.458	54	0.388	0.479	0.594	0.766	9.4%
Averages		**4.165**	**0.382**		**0.324**	**0.400**	**0.495**	**0.639**	**9.1%**
Nominal 8" Overlays:									
0608	17	7.185	0.439	54	0.372	0.459	0.569	0.734	6.1%
0608	29	7.898	0.591	49	0.502	0.619	0.768	0.991	7.5%
0608	40	7.864	0.577	54	0.489	0.604	0.748	0.965	7.3%
0608	42	8.495	0.434	54	0.368	0.454	0.563	0.726	5.1%
A608	5	9.427	0.407	54	0.345	0.425	0.527	0.680	4.3%
Averages		**8.174**	**0.489**		**0.415**	**0.512**	**0.635**	**0.819**	**6.1%**

Table A7 – Reported variation in layer thicknesses from the 2-point GPS data for flexible pavements.

Standard Deviation (inches)	Count	Nominal or Design Thickness (inches)
AC Thicknesses (GPS Flexible):		
0.17	68	Nominal < 3"
0.22	*123*	*Nominal 3"- 5" incl.*
0.31	61	Nominal 5"- 7" excl.
0.30	*73*	*Nominal 7"- 9" incl.*
0.36	20	Nominal 9"- 10" excl.
0.43	*56*	*Nominal 10"- 14" incl.*
0.42	10	Nominal > 14"
0.28	411	Overall AC
GB & GS Thicknesses (GPS Flexible):		
0.74	*29*	*Nominal 3"- 5" incl.*
0.69	33	Nominal 5"- 7" excl.
1.22	*55*	*Nominal 7"- 9" incl.*
3.41	4	Nominal 9"- 10" excl.
0.77	*90*	*Nominal 10"- 14" incl.*
1.85	52	Nominal 14"- 22" excl.
2.14	44	Nominal > 22" incl.
1.25	307	Overall GB & GS
TB & TS Thicknesses (GPS Flexible):		
n/a	1	Nominal < 3"
0.26	*20*	*Nominal 3"- 5" incl.*
0.41	53	Nominal 5"- 7" excl.
0.50	*29*	*Nominal 7"- 9" incl.*
0.62	5	Nominal 9"- 10" excl.
0.63	*21*	*Nominal 10"- 14" incl.*
0.81	5	Nominal > 14"
0.46	134	Overall TB & TS

Overall nationwide averages and outlier table:

GPS Pavement Layer	Sta.0+00 Avg. Measured Thickness	Sta.5+00 Avg. Measured Thickness	Mean (in.)	Standard .Deviation (in)	Avg. State Reported Thickness (in)	Diff., Mean Meas. vs. State Rprd.	Rejected Data from Tables?
AC	6.17	6.11	6.14	0.28	6.20	-0.06	{No outliers}
GB & GS	13.17	13.19	13.18	1.25	12.93	0.25	{14 outliers}
TB & TS	7.38	7.40	7.39	0.46	7.69	-0.30	{13 outliers}

Table A8 – Reported variation in layer thicknesses from the 2-point GPS data for rigid pavements.

Standard Deviation (inches)	Count	Nominal or Design Thickness (inches)
PCC Thicknesses (GPS Rigid):		
0.30	10	Nominal < 7"
0.18	*227*	*Nominal 7" - 9" incl.*
0.15	19	Nominal 9" - 10" excl.
0.21	*98*	*Nominal 10" - 12" incl.*
0.65	6	Nominal > 12"
0.20	360	Overall PCC
GB & GS Thicknesses (GPS Rigid):		
0.47	5	Nominal < 3"
0.72	*74*	*Nominal 3"- 5" incl.*
1.13	*69*	*Nominal 5"- 7" excl.*
1.01	35	Nominal 7"- 10" incl.
1.09	37	Nominal > 10"
0.95	220	Overall GB & GS
TB & TS Thicknesses (GPS Rigid):		
0.16	6	Nominal < 3"
0.24	*102*	*Nominal 3"- 5" incl.*
0.33	*48*	*Nominal 5"- 7" excl.*
0.48	19	Nominal 7"- 10" incl.
0.86	19	Nominal > 10"
0.35	194	Overall GB & GS

Overall nationwide averages and outlier table:

GPS Pavement Layer	Sta.0+00 Avg. Measured Thickness	Sta.5+00 Avg. Measured Thickness	Mean (in.)	Standard Deviation (in)	Avg. State Reported Thickness (in)	Diff., Mean Meas. vs. State Rprd.	Rejected Data from Tables?
PCC	9.11	9.16	9.14	0.20	8.98	0.15	{No outliers}
GB & GS	7.730	7.771	7.75	0.949	7.64	0.12	*{20 outliers}
TB & TS	5.753	5.823	5.79	0.347	5.81	-0.02	{3 outliers}

* Plus 6 GPS 7000 series data sets that contain identical (default?) values for both the measured and state-reported thicknesses.

APPENDIX B – PCC=1 RELATED FIGURES AND DATA TABLES

Appendix B contains the following tables and figures related to PCC pavement design input parameters:

Table

Figure

Table B1 – GPS Compressive Strength – All Data

SHRP_ID	STATE_CODE	CONSTRUCTION_NO	LAYER_NO	COMP_STRENGTH_AGE	COMP_STRENGTH_MEAN	COMP_STRENGTH_MAX	COMP_STRENGTH_MIN	NO_COMP_STRENGTH_TESTS	COMP_STRENGTH_STD_DEV	COEF. OF VARIATION, %	RECORD_STATUS
3804	12	1	3	28	4393	5540	3110	90	574	13.1	E
3811	12	1	4	28	3949	5093	2900	78	501	12.7	E
4057	12	1	3	28	5063	5550	4700	26	209	4.1	E
4059	12	1	6	28	5852	6475	4570	16	530	9.1	E
4109	12	1	3	28	5852	6475	4570	16	530	9.1	E
5025	16	1	4	28	4130	4550	3500	5	386	9.3	E
0600	19	1	4	28	4596	6015	3720	120	553	12.0	E
3006	19	1	4	28	4295	5560	3380	84	470	10.9	E
3009	19	1	4	28	5291	6225	3875	99	522	9.9	E
3028	19	1	4	28	4276	5155	3620	29	411	9.6	E
3033	19	1	4	28	4000	8305	3130	46	715	17.9	E
3055	19	1	4	28	5130	6220	4275	38	420	8.2	E
5042	19	1	4	28	4831	5260	3850	46	301	6.2	E
5046	19	1	4	28	4626	5980	3200	99	718	15.5	E
9116	19	1	4	28	4803	5925	3510	67	600	12.5	E
9126	19	1	4	28	5757	7040	4060	29	732	12.7	E
3013	23	1	4	28	4365	4569	4239	3	157	3.6	E
3014	23	1	4	28	3857	4304	3225	6	422	10.9	E
4033	27	1	3	60	5682	7120	4310	50	591	10.4	E
4082	27	1	3	60	4926	5700	3620	89	404	8.2	E
3018	28	1	3	200	5693	6862	5361	5	921	16.2	E
3019	28	1	3	197	6062	7148	5361	5	78	1.3	E
3099	28	1	5	28	6208	7146	5775	7	486	7.8	E
5805	28	1	4	28	5139	5975	3498	20	622	12.1	E
5503	29	1	3	28	5078	5500	4680	6	363	7.1	E
3018	31	1	3	28	5980	6540	5160	17	389	6.5	E
3023	31	1	3	28	5592	6510	4000	45	597	10.7	E
3028	31	1	3	28	5120	5750	4100	4	745	14.6	E
3033	31	1	3	39	5107	5910	4280	39	448	8.8	E
3008	37	1	3	14	5146	5977	4266	10	285	5.5	E
3011	37	1	3	28	3838	6032	1854	28	909	23.7	E
3807	37	1	3	28	4343	4782	3947	8	83	1.9	E
3816	37	1	3	7	4178	4740	3680	8	115	2.8	E
5037	37	1	3	14	4626	5340	3910	8	270	5.8	E
3006	38	1	3	28	3902	4510	3220	4	566	14.5	E
0600	40	1	4	28	5245	6510	3340	50	706	13.5	E
5005	41	1	3	28	5690	7000	4770	25	610	10.7	E
5006	41	1	4	28	4340	5220	3255	99	425	9.8	E
5008	41	1	4	28	4739	6245	3765	179	470	9.9	E
5021	41	1	3	28	4639	7000	3395	99	857	18.5	E
5022	41	1	3	28	4520	5960	3425	85	460	10.2	E
7081	41	1	4	28	4934	6170	4190	36	461	9.3	E
3044	42	1	3	7	3537	3608	3431	4	80	2.3	E
3012	46	1	3	28	5416	6650	4550	5	798	14.7	E
3013	46	1	4	28	5833	6380	5320	3	529	9.1	E
3052	46	1	3	28	4956	5520	4280	5	501	10.1	E
5020	46	1	3	28	6188	6720	5530	5	524	8.5	E
5040	46	1	4	28	5771	6995	4735	4	581	10.1	E
5323	48	1	4	7	3939	4400	3478	8	461	11.7	E
5335	48	1	4	7	3939	4400	3478	8	461	11.7	E
7086	49	1	8	28	4550	5210	3650	3	4	0.1	E
1682	50	1	4		5592	5651	5492	3	37	0.7	E
3008	55	1	3	7	3110	3953	2970	20	517	16.6	E
3014	55	1	3	7	3420	4240	2550	26	431	12.6	E
3027	56	1	3	28	4587	5784	3643	32	484	10.6	E
3802	83	1	4	28	4899	5955	3843	52	539	11.0	E
3015	89	1	4	28	4289	4931	3582	30	263	6.1	E
3016	89	1	4	28	5352	5801	4511	6	441	8.2	E
	All Ages		Mean		4848	5795	3959	35	470	10	
	All Ages		min		3110	3608	1854	3	4	0	
	All Ages		max		6208	8305	5775	179	921	24	
	All Ages		stdev		749	945	787	38	208	5	
	All Ages		count		58	58	58	58	58	58	

Table B2 – GPS Compressive Strength – By Age

SHRP_ID	STATE_CODE	CONSTRUCTION_NO	LAYER_NO	COMP_STRENGTH_AGE	COMP_STRENGTH_MEAN	COMP_STRENGTH_MAX	COMP_STRENGTH_MIN	NO_COMP_STRENGTH_TESTS	COMP_STRENGTH_STD_DEV	COEF. OF VARIATION, %	RECORD_STATUS
3816	37	1	3	7	4178	4740	3680	8	115	2.8	E
3044	42	1	3	7	3537	3608	3431	4	80	2.3	E
5323	48	1	4	7	3939	4400	3478	8	461	11.7	E
5335	48	1	4	7	3939	4400	3478	8	461	11.7	E
3008	55	1	3	7	3110	3953	2970	20	517	16.6	E
3014	55	1	3	7	3420	4240	2550	26	431	12.6	E
			Mean		3687	4224	3265	12	344	10	
			min		3110	3608	2550	4	80	2	
			max		4178	4740	3680	26	517	17	
			stdev		399	395	422	9	193	6	
			count		6	6	6	6	6	6	
3804	12	1	3	28	4393	5540	3110	90	574	13.1	E
3811	12	1	4	28	3949	5093	2900	78	501	12.7	E
4057	12	1	3	28	5063	5550	4700	26	209	4.1	E
4059	12	1	6	28	5852	6475	4570	16	530	9.1	E
4109	12	1	3	28	5852	6475	4570	16	530	9.1	E
5025	16	1	4	28	4130	4550	3500	5	386	9.3	E
0600	19	1	4	28	4596	6015	3720	120	553	12.0	E
3006	19	1	4	28	4295	5560	3380	84	470	10.9	E
3009	19	1	4	28	5291	6225	3875	99	522	9.9	E
3028	19	1	4	28	4276	5155	3620	29	411	9.6	E
3033	19	1	4	28	4000	8305	3130	46	715	17.9	E
3055	19	1	4	28	5130	6220	4275	38	420	8.2	E
5042	19	1	4	28	4831	5260	3850	46	301	6.2	E
5046	19	1	4	28	4626	5980	3200	99	718	15.5	E
9116	19	1	4	28	4803	5925	3510	67	600	12.5	E
9126	19	1	4	28	5757	7040	4060	29	732	12.7	E
3013	23	1	4	28	4365	4569	4239	3	157	3.6	E
3014	23	1	4	28	3857	4304	3225	6	422	10.9	E
3099	28	1	5	28	6208	7146	5775	7	486	7.8	E
5805	28	1	4	28	5139	5975	3498	20	622	12.1	E
5503	29	1	3	28	5078	5500	4680	6	363	7.1	E
3018	31	1	3	28	5980	6540	5160	17	389	6.5	E
3023	31	1	3	28	5592	6510	4000	45	597	10.7	E
3028	31	1	3	28	5120	5750	4100	4	745	14.6	E
3011	37	1	3	28	3838	6032	1854	28	909	23.7	E
3807	37	1	3	28	4343	4782	3947	8	83	1.9	E
3006	38	1	3	28	3902	4510	3220	4	566	14.5	E
0600	40	1	4	28	5245	6510	3340	50	706	13.5	E
5005	41	1	3	28	5690	7000	4770	25	610	10.7	E
5006	41	1	4	28	4340	5220	3255	99	425	9.8	E
5008	41	1	4	28	4739	6245	3765	179	470	9.9	E
5021	41	1	3	28	4639	7000	3395	99	857	18.5	E
5022	41	1	3	28	4520	5960	3425	85	460	10.2	E
7081	41	1	4	28	4934	6170	4190	36	461	9.3	E
3012	46	1	3	28	5416	6650	4550	5	798	14.7	E
3013	46	1	4	28	5833	6380	5320	3	529	9.1	E
3052	46	1	3	28	4956	5520	4280	5	501	10.1	E
5020	46	1	3	28	6188	6720	5530	5	524	8.5	E
5040	46	1	4	28	5771	6995	4735	4	581	10.1	E
7086	49	1	8	28	4550	5210	3650	3	4	0.1	E
3027	56	1	3	28	4587	5784	3643	32	484	10.6	E
3802	83	1	4	28	4899	5955	3843	52	539	11.0	E
3015	89	1	4	28	4289	4931	3582	30	263	6.1	E
3016	89	1	4	28	5352	5801	4511	6	441	8.2	E
			Mean		4914	5933	3942	40	504	10	
			min		3838	4304	1854	3	4	0	
			max		6208	8305	5775	179	909	24	
			stdev		669	828	759	40	187	4	
			count		44	44	44	44	44	44	

Table B3 – GPS Compressive Strength – By Strength Level

SHRP_ID	STATE_CODE	CONSTRUCTION_NO	LAYER_NO	COMP_STRENGTH_AGE	COMP_STRENGTH_MEAN	COMP_STRENGTH_MAX	COMP_STRENGTH_MIN	NO_COMP_STRENGTH_TESTS	COMP_STRENGTH_STD_DEV	COEF. OF VARIATION, %	RECORD_STATUS
3008	55	1	3	7	3110	3953	2970	20	517	16.6	E
3014	55	1	3	7	3420	4240	2550	26	431	12.6	E
3044	42	1	3	7	3537	3608	3431	4	80	2.3	E
3011	37	1	3	28	3838	6032	1854	28	909	23.7	E
3014	23	1	4	28	3857	4304	3225	6	422	10.9	E
3006	38	1	3	28	3902	4510	3220	4	566	14.5	E
5323	48	1	4	7	3939	4400	3478	8	461	11.7	E
5335	48	1	4	7	3939	4400	3478	8	461	11.7	E
3811	12	1	4	28	3949	5093	2900	78	501	12.7	E
3033	19	1	4	28	4000	8305	3130	46	715	17.9	E
	All Ages		Mean		3749	4885	3024	23	506	13	
	All Ages		min		3110	3608	1854	4	80	2	
	All Ages		max		4000	8305	3478	78	909	24	
	All Ages		stdev		294	1371	503	24	213	6	
	All Ages		count		10	10	10	10	10	10	
4057	12	1	3	28	5063	5550	4700	26	209	4.1	E
5503	29	1	3	28	5078	5500	4680	6	363	7.1	E
3033	31	1	3	39	5107	5910	4280	39	448	8.8	E
3028	31	1	3	28	5120	5750	4100	4	745	14.6	E
3055	19	1	4	28	5130	6220	4275	38	420	8.2	E
5805	28	1	4	28	5139	5975	3498	20	622	12.1	E
3008	37	1	3	14	5146	5977	4266	10	285	5.5	E
0600	40	1	4	28	5245	6510	3340	50	706	13.5	E
3009	19	1	4	28	5291	6225	3875	99	522	9.9	E
3016	89	1	4	28	5352	5801	4511	6	441	8.2	E
3012	46	1	3	28	5416	6650	4550	5	798	14.7	E
3023	31	1	3	28	5592	6510	4000	45	597	10.7	E
1682	50	1	4		5592	5651	5492	3	37	0.7	E
4033	27	1	3	60	5682	7120	4310	50	591	10.4	E
5005	41	1	3	28	5690	7000	4770	25	610	10.7	E
3018	28	1	3	200	5693	6862	5361	5	921	16.2	E
9126	19	1	4	28	5757	7040	4060	29	732	12.7	E
5040	46	1	4	28	5771	6995	4735	4	581	10.1	E
3013	46	1	4	28	5833	6380	5320	3	529	9.1	E
4059	12	1	6	28	5852	6475	4570	16	530	9.1	E
4109	12	1	3	28	5852	6475	4570	16	530	9.1	E
3018	31	1	3	28	5980	6540	5160	17	389	6.5	E
3019	28	1	3	197	6062	7148	5361	5	78	1.3	E
5020	46	1	3	28	6188	6720	5530	5	524	8.5	E
3099	28	1	5	28	6208	7146	5775	7	486	7.8	E
	All Ages		Mean		5554	6405	4604	21	508	9	
	All Ages		min		5063	5500	3340	3	37	1	
	All Ages		max		6208	7148	5775	99	921	16	
	All Ages		stdev		369	529	637	22	208	4	
	All Ages		count		25	25	25	25	25	25	

Table B4 – GPS Compressive Strength – By State

SHRP_ID	STATE_CODE	CONSTRUCTION_NO	LAYER_NO	COMP_STRENGTH_AGE	COMP_STRENGTH_MEAN	COMP_STRENGTH_MAX	COMP_STRENGTH_MIN	NO_COMP_STRENGTH_TESTS	COMP_STRENGTH_STD_DEV	COEF. OF VARIATION, %	RECORD_STATUS
3804	12	1	3	28	4393	5540	3110	90	574	13.1	E
3811	12	1	4	28	3949	5093	2900	78	501	12.7	E
4057	12	1	3	28	5063	5550	4700	26	209	4.1	E
4059	12	1	6	28	5852	6475	4570	16	530	9.1	E
4109	12	1	3	28	5852	6475	4570	16	530	9.1	E
	All Ages		Mean		5022	5827	3970	45	469	10	
	All Ages		min		3949	5093	2900	16	209	4	
	All Ages		max		5852	6475	4700	90	574	13	
	All Ages		stdev		855	620	886	36	148	4	
	All Ages		count		5	5	5	5	5	5	
0600	19	1	4	28	4596	6015	3720	120	553	12.0	E
3006	19	1	4	28	4295	5560	3380	84	470	10.9	E
3009	19	1	4	28	5291	6225	3875	99	522	9.9	E
3028	19	1	4	28	4276	5155	3620	29	411	9.6	E
3033	19	1	4	28	4000	8305	3130	46	715	17.9	E
3055	19	1	4	28	5130	6220	4275	38	420	8.2	E
5042	19	1	4	28	4831	5260	3850	46	301	6.2	E
5046	19	1	4	28	4626	5980	3200	99	718	15.5	E
9116	19	1	4	28	4803	5925	3510	67	600	12.5	E
9126	19	1	4	28	5757	7040	4060	29	732	12.7	E
	All Ages		Mean		4761	6169	3662	66	544	12	
	All Ages		min		4000	5155	3130	29	301	6	
	All Ages		max		5757	8305	4275	120	732	18	
	All Ages		stdev		526	923	368	33	148	3	
	All Ages		count		10	10	10	10	10	10	
3008	37	1	3	14	5146	5977	4266	10	285	5.5	E
3011	37	1	3	28	3838	6032	1854	28	909	23.7	E
3807	37	1	3	28	4343	4782	3947	8	83	1.9	E
3816	37	1	3	7	4178	4740	3680	8	115	2.8	E
5037	37	1	3	14	4626	5340	3910	8	270	5.8	E
	All Ages		Mean		4426	5374	3531	12	332	8	
	All Ages		min		3838	4740	1854	8	83	2	
	All Ages		max		5146	6032	4266	28	909	24	
	All Ages		stdev		493	623	961	9	335	9	
	All Ages		count		5	5	5	5	5	5	
5005	41	1	3	28	5690	7000	4770	25	610	10.7	E
5006	41	1	4	28	4340	5220	3255	99	425	9.8	E
5008	41	1	4	28	4739	6245	3765	179	470	9.9	E
5021	41	1	3	28	4639	7000	3395	99	857	18.5	E
5022	41	1	3	28	4520	5960	3425	85	460	10.2	E
7081	41	1	4	28	4934	6170	4190	36	461	9.3	E
	All Ages		Mean		4810	6266	3800	87	547	11	
	All Ages		min		4340	5220	3255	25	425	9	
	All Ages		max		5690	7000	4770	179	857	18	
	All Ages		stdev		475	675	582	55	165	3	
	All Ages		count		6	6	6	6	6	6	
3012	46	1	3	28	5416	6650	4550	5	798	14.7	E
3013	46	1	4	28	5833	6380	5320	3	529	9.1	E
3052	46	1	3	28	4956	5520	4280	5	501	10.1	E
5020	46	1	3	28	6188	6720	5530	5	524	8.5	E
5040	46	1	4	28	5771	6995	4735	4	581	10.1	E
	All Ages		Mean		5633	6453	4883	4	587	10	
	All Ages		min		4956	5520	4280	3	501	8	
	All Ages		max		6188	6995	5530	5	798	15	
	All Ages		stdev		467	566	526	1	122	2	
	All Ages		count		5	5	5	5	5	5	

Table B5 – SPS Cylinder Compressive Strength

a) All Data

State	Design 14-day Flexural Strength, psi	14-Day Average Compressive Strength, psi	14-Day Standard Deviation, psi	14-Day Coef. Of Variation, %	28-Day Average Compressive Strength, psi	28-Day Standard Deviation, psi	28-Day Coef. Of Variation, %	1-Year Average Compressive Strength, psi	1-Year Standard Deviation, psi	1-Year Coef. Of Variation, %
4	550	3680	243	6.6	4473	176	3.9	6290	288	4.6
4	900	6197	134	2.2	6610	131	2.0	7637	451	5.9
5	550	-	-		-	-		6348	1912	30.1
5	900	-	-		-	-		10859	713	6.6
8	550	2568	469	18.3	3034	525	17.3	4412	521	11.8
8	900	5665	721	12.7	6406	323	5.0	8267	632	7.6
10	550	3730	175	4.7	4053	215	5.3	4783	1005	21.0
10	900	4740	877	18.5	5580	1513	27.1	6158	1269	20.6
19	550	2840	303	10.7	3530	427	12.1	---	---	---
19	900	6003	549	9.1	6803	560	8.2	8453	959	11.3
20	550	4251	592	13.9	5030	640	12.7	6442	684	10.6
20	900	6430	891	13.9	7168	1113	15.5	8625	1495	17.3
26	550	-	-	-	4223	154	3.6	5267	368	7.0
26	900	-	-	-	-	-	-	8900	672	7.6
32	550	3367	710	21.1	3973	675	17.0	5257	942	17.9
32	900	5757	559	9.7	6583	263	4.0	9670	265	2.7
37	550	-	-	-	-	-	-	6510	1125	17.3
37	900	5734	1141	19.9	7368	618	8.4	9590	729	7.6
38	550	-	-	-	-	-	-	5274	530	10.0
38	900	-	-	-	-	-	-	6610	169	2.6
39	550	5030	350	7.0	5786	719	12.4	7226	884	12.2
39	900	-	-	-	7611	445	5.8	-	-	-
53	550	3399	472	13.9	4219	647	15.3	4633	410	8.8
53	900	6182	408	6.6	7086	432	6.1	5617	676	12.0
55	550	-	-	-	-	-	-	-	-	-
55	900	7020	180	2.6	-	-	-	-	-	-
	Average		516	11.3		532	10.1		759	11.5
	Min		134	2.2		131	2.0		169	2.6
	Max		1141	21.1		1513	27.1		1912	30.1
	ST Deviation		284	5.9		348	6.6		422	6.8
	VC, %		55.0	52.9		65.5	65.0		55.6	58.9
	Records		17	17.0		18.0	18.0		22.0	22.0

b) Sorted by Design Strength

State	Design 14-day Flex Str, psi	14-Day Average Comp Str, psi	14-Day Std Dev., psi	14-Day Coef. of Var., %	28-Day Avg Comp Str, psi	28-Day Std Dev, psi	28-Day Coef. of Var, %	1-Year Avg Comp Str, psi	1-Year Std Dev., psi	1-Year Coef. of Var, %
4	550	3680	243	6.6	4473	176	3.9	6290	288	4.6
5	550	-	-		-	-		6348	1912	30.1
8	550	2568	469	18.3	3034	525	17.3	4412	521	11.8
10	550	3730	175	4.7	4053	215	5.3	4783	1005	21.0
19	550	2840	303	10.7	3530	427	12.1	---	---	---
20	550	4251	592	13.9	5030	640	12.7	6442	684	10.6
26	550	-	-	-	4223	154	3.6	5267	368	7.0
32	550	3367	710	21.1	3973	675	17.0	5257	942	17.9
37	550	-	-	-	-	-	-	6510	1125	17.3
38	550	-	-	-	-	-	-	5274	530	10.0
39	550	5030	350	7.0	5786	719	12.4	7226	884	12.2
53	550	3399	472	13.9	4219	647	15.3	4633	410	8.8
55	550	-	-	-	-	-	-	-	-	-
	Average	3608	414	12.0	4258	464	11.1	5677	788	13.8
	Min	2568	175	4.7	3034	154	3.6	4412	288	4.6
	Max	5030	710	21.1	5786	719	17.3	7226	1912	30.1
	ST Deviation	777	181	5.8	801	229	5.4	922	466	7.3
	VC, %	21.5	43.6	48.6	18.8	49.4	49.1	16.2	59.2	53.1
	Records	8.0	8.0	8.0	9.0	9.0	9.0	11.0	11.0	11.0
4	900	6197	134	2.2	6610	131	2.0	7637	451	5.9
5	900	-	-		-	-		10859	713	6.6
8	900	5665	721	12.7	6406	323	5.0	8267	632	7.6
10	900	4740	877	18.5	5580	1513	27.1	6158	1269	20.6
19	900	6003	549	9.1	6803	560	8.2	8453	959	11.3
20	900	6430	891	13.9	7168	1113	15.5	8625	1495	17.3
26	900	-	-	-	-	-	-	8900	672	7.6
32	900	5757	559	9.7	6583	263	4.0	9670	265	2.7
37	900	5734	1141	19.9	7368	618	8.4	9590	729	7.6
38	900	-	-	-	-	-	-	6610	169	2.6
39	900	-	-	-	7611	445	5.8	-	-	-
53	900	6182	408	6.6	7086	432	6.1	5617	676	12.0
55	900	7020	180	2.6	-	-	-	-	-	-
	Average	5970	607	10.6	6802	600	9.1	8217	730	9.3
	Min	4740	134	2.2	5580	131	2.0	5617	169	2.6
	Max	7020	1141	19.9	7611	1513	27.1	10859	1495	20.6
	ST Deviation	624	336	6.3	605	442	7.7	1600	394	5.7
	VC, %	10.5	55.4	59.7	8.9	73.7	84.8	19.5	54.0	61.1
	Records	9.0	9.0	9.0	9.0	9.0	9.0	11.0	11.0	11.0

Table B6 – SPS Core Compressive Strength

a) All Data

State	Design 14-day Flexural Strength, psi	14-Day Average Compressive Strength, psi	14-Day Standard Deviation, psi	14-Day Coef. Of Variation, %	28-Day Average Compressive Strength, psi	28-Day Standard Deviation, psi	28-Day Coef. Of Variation, %	1-Year Average Compressive Strength, psi	1-Year Standard Deviation, psi	1-Year Coef. Of Variation, %
4	550	3947	346	8.8	4363	415	9.5	6093	724	11.9
4	900	6268	315	5.0	6760	309	4.6	7868	713	9.1
5	550	-	-	-	-	-	-	5759	1043	18.1
5	900	-	-	-	-	-	-	10773	714	6.6
8	550	2759	400	14.5	3230	288	8.9	4890	439	9.0
8	900	5061	579	11.4	6168	881	14.3	7792	451	5.8
10	550	4316	356	8.3	4540	639	14.1	5570	897	16.1
10	900	4741	899	19.0	5003	679	13.6	6148	1510	24.6
19	550	3055	496	16.2	3160	301	9.5	4475	578	12.9
19	900	5242	398	7.6	5764	330	5.7	6298	775	12.3
20	550	-	-	-	-	-	-	-	-	-
20	900	-	-	-	-	-	-	-	-	-
26	550	4513	773	17.1	4438	1017	22.9	6263	851	13.6
26	900	6073	74	1.2	5815	329	5.7	9023	310	3.4
32	550	2783	486	17.5	3267	492	15.1	4862	211	4.3
32	900	3328	181	5.4	4102	147	3.6	7746	793	10.2
37	550	3240	761	23.5	4006	361	9.0	6060	1038	17.1
37	900	5026	1274	25.3	6218	360	5.8	8192	1827	22.3
38	550	3000	469	15.6	3600	436	12.1	5210	641	12.3
38	900	5770	346	6.0	-	-	-	7962	450	5.6
39	550	5227	986	18.9	5569	757	13.6	7897	877	11.1
39	900	7028	624	8.9	7314	1048	14.3	-	-	-
53	550	2849	307	10.8	3358	215	6.4	4532	438	9.7
53	900	6598	472	7.2	7253	529	7.3	8150	399	4.9
55	550	4117	513	12.5	4442	620	13.9	-	-	-
55	900	5907	626	10.6	6403	667	10.4	-	-	-
	Average		531	12.3		515	10.5		747	11.5
	Min		74	1.2		147	3.6		211	3.4
	Max		1274	25.3		1048	22.9		1827	24.6
	ST Deviation		276	6.3		255	4.7		386	5.7
	VC, %		52.0	50.7		49.6	44.5		51.7	50.0
	Records		22.0	22.0		21.0	21.0		21.0	21.0

b) Sorted by Design Strength

State	Design 14-day Flex Str, psi	14-Day Average Comp Str, psi	14-Day Std Dev., psi	14-Day Coef. of Var., %	28-Day Avg Comp Str, psi	28-Day Std Dev, psi	28-Day Coef. of Var, %	1-Year Avg Comp Str, psi	1-Year Std Dev., psi	1-Year Coef. of Var, %
4	550	3947	346	8.8	4363	415	9.5	6093	724	11.9
5	550	-	-	-	-	-	-	5759	1043	18.1
8	550	2759	400	14.5	3230	288	8.9	4890	439	9.0
10	550	4316	356	8.3	4540	639	14.1	5570	897	16.1
19	550	3055	496	16.2	3160	301	9.5	4475	578	12.9
20	550	-	-	-	-	-	-	-	-	-
26	550	4513	773	17.1	4438	1017	22.9	6263	851	13.6
32	550	2783	486	17.5	3267	492	15.1	4862	211	4.3
37	550	3240	761	23.5	4006	361	9.0	6060	1038	17.1
38	550	3000	469	15.6	3600	436	12.1	5210	641	12.3
39	550	5227	986	18.9	5569	757	13.6	7897	877	11.1
53	550	2849	307	10.8	3358	215	6.4	4532	438	9.7
55	550	4117	513	12.5	4442	620	13.9	-	-	-
	Average	3619	536	14.9	3998	504	12.3	5601	703	12.4
	Min	2759	307	8.3	3160	215	6.4	4475	211	4.3
	Max	5227	986	23.5	5569	1017	22.9	7897	1043	18.1
	ST Deviation	843	214	4.6	754	237	4.5	990	268	4.0
	VC, %	23.3	39.9	30.7	18.9	47.1	36.5	17.7	38.1	32.0
	Records	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0
4	900	6268	315	5.0	6760	309	4.6	7868	713	9.1
5	900	-	-	-	-	-	-	10773	714	6.6
8	900	5061	579	11.4	6168	881	14.3	7792	451	5.8
10	900	4741	899	19.0	5003	679	13.6	6148	1510	24.6
19	900	5242	398	7.6	5764	330	5.7	6298	775	12.3
20	900	-	-	-	-	-	-	-	-	-
26	900	6073	74	1.2	5815	329	5.7	9023	310	3.4
32	900	3328	181	5.4	4102	147	3.6	7746	793	10.2
37	900	5026	1274	25.3	6218	360	5.8	8192	1827	22.3
38	900	5770	346	6.0	-	-	-	7962	450	5.6
39	900	7028	624	8.9	7314	1048	14.3	-	-	-
53	900	6598	472	7.2	7253	529	7.3	8150	399	4.9
55	900	5907	626	10.6	6403	667	10.4	-	-	-
	Average	5549	526	9.8	6080	528	8.5	7995	794	10.5
	Min	3328	74	1.2	4102	147	3.6	6148	310	3.4
	Max	7028	1274	25.3	7314	1048	14.3	10773	1827	24.6
	ST Deviation	1022	338	6.9	985	286	4.2	1302	497	7.3
	VC, %	18.4	64.2	70.2	16.2	54.2	49.6	16.3	62.5	70.0
	Records	11.0	11.0	11.0	10.0	10.0	10.0	10.0	10.0	10.0

Table B7 – GPS Flexural Strength – All Data

SHRP_ID	STATE_CODE	CONSTRUCTION_NO	LAYER_NO	FLEXURAL_STRENGTH_AGE	FLEXURAL_STRENGTH_MEAN	FLEXURAL_STRENGTH_MIN	FLEXURAL_STRENGTH_MAX	NO_FLEXURAL_STRENGTH_TESTS	FLEXURAL_STRENGTH_STD_DEV	COEF. OF VARIATION, %	RECORD_STATUS
3010	6	1	4	14	565	505	590	6	35	6.2	E
4008	9	1	3	9	645	600	680	4	37	5.7	E
5001	9	1	3	14	678	630	720	8	32	4.7	E
4059	12	1	6	28	853	700	1050	10	125	14.7	E
4109	12	1	3	28	853	700	1050	10	125	14.7	E
3017	16	1	5	28	681	650	735	7	32	4.7	E
5025	16	1	4	14	625	585	675	6	38	6.1	E
5908	17	1	3	14	910	755	1000	25	77	8.5	E
0600	19	1	4	14	649	485	868	118	78	12.0	E
3006	19	1	4	14	783	652	902	23	66	8.4	E
3009	19	1	4	14	834	709	978	23	76	9.1	E
3028	19	1	4	14	706	580	784	19	50	7.1	E
3033	19	1	4	14	762	635	849	29	54	7.1	E
3055	19	1	4	14	700	579	803	12	68	9.7	E
5042	19	1	4	7	686	561	772	8	58	8.5	E
5046	19	1	4	7	613	517	664	16	39	6.4	E
9116	19	1	4	14	804	661	905	19	60	7.5	E
9126	19	1	4	14	716	530	935	59	90	12.6	E
3013	20	1	4	6	612	540	696	20	48	7.8	E
3015	20	1	3	7	644	490	837	58	83	12.9	E
3060	20	1	4	5	610	559	708	14	49	8.0	E
3013	23	1	4	28	588	549	638	3	44	7.5	E
3014	23	1	4	28	605	579	646	4	29	4.8	E
3018	28	1	3	28	645	507	754	10	87	13.5	E
3019	28	1	3	28	645	507	754	10	87	13.5	E
3099	28	1	5	7	679	610	771	15	41	6.0	E
4024	28	1	4	28	712	595	785	19	50	7.0	E
5006	28	1	4	28	524	325	666	3	178	34.0	E
5025	28	1	4	28	608	527	680	11	51	8.4	E
5803	28	1	3	28	595	495	715	5	96	16.1	E
5805	28	1	4	28	710	533	970	101	82	11.5	E
3010	32	1	4	13	545	313	725	41	81	14.9	E
3013	32	1	4	7	651	482	810	77	77	11.8	E
3008	37	1	3	14	608	487	703	99	9	1.5	E
3011	37	1	3	14	609	559	666	77	6	1.0	E
3807	37	1	3	14	602	541	716	10	29	4.8	E
3816	37	1	3	14	645	558	718	10	27	4.2	E
5037	37	1	3	14	488	375	600	10	56	11.5	E
5005	41	1	3	28	584	530	700	15	45	7.7	E
5008	41	1	4	28	611	545	685	6	61	10.0	E
5283	48	1	5	7	717	638	795	144	59	8.2	E
5284	48	1	5	7	722	591	878	59	85	11.8	E
5301	48	1	4	7	768	590	945	241	86	11.2	E
5310	48	1	4	7	795	664	925	72	114	14.3	E
5317	48	1	4	7	706	604	807	326	22	3.1	E
5328	48	1	4	7	769	475	1010	179	102	13.3	E
5334	48	1	4	7	619	563	666	28	28	4.5	E
1682	50	1	4	28	851	628	1135	43	129	15.2	E
3011	53	1	4	14	713	650	781	5	57	8.0	E
3019	53	1	3	14	812	594	937	4	155	19.1	E
3027	56	1	3	28	787	716	900	8	68.5	8.7	E
	All Ages		Mean		683	568	798	42	66	10	
	All Ages		min		488	313	590	3	6	1	
	All Ages		max		910	755	1135	326	178	34	
	All Ages		stdev		94	89	130	63	35	5	
	All Ages		count		51	51	51	51	51	51	

Table B8 – GPS Flexural Strength – By Age

SHRP_ID	STATE_CODE	CONSTRUCTION_NO	LAYER_NO	FLEXURAL_STRENGTH_AGE	FLEXURAL_STRENGTH_MEAN	FLEXURAL_STRENGTH_MIN	FLEXURAL_STRENGTH_MAX	NO_FLEXURAL_STRENGTH_TESTS	FLEXURAL_STRENGTH_STD_DEV	COEF. OF VARIATION, %	RECORD_STATUS
5042	19	1	4	7	686	561	772	8	58	8.5	E
5046	19	1	4	7	613	517	664	16	39	6.4	E
3015	20	1	3	7	644	490	837	58	83	12.9	E
3099	28	1	5	7	679	610	771	15	41	6.0	E
3013	32	1	4	7	651	482	810	77	77	11.8	E
5283	48	1	5	7	717	638	795	144	59	8.2	E
5284	48	1	5	7	722	591	878	59	85	11.8	E
5301	48	1	4	7	768	590	945	241	86	11.2	E
5310	48	1	4	7	795	664	925	72	114	14.3	E
5317	48	1	4	7	706	604	807	326	22	3.1	E
5328	48	1	4	7	769	475	1010	179	102	13.3	E
5334	48	1	4	7	619	563	666	28	28	4.5	E
			Mean		697	565	823	102	66	9	
			min		613	475	664	8	22	3	
			max		795	664	1010	326	114	14	
			stdev		60	62	104	101	30	4	
			count		12	12	12	12	12	12	
3010	6	1	4	14	565	505	590	6	35	6.2	E
5001	9	1	3	14	678	630	720	8	32	4.7	E
5025	16	1	4	14	625	585	675	6	38	6.1	E
5908	17	1	3	14	910	755	1000	25	77	8.5	E
0600	19	1	4	14	649	485	868	118	78	12.0	E
3006	19	1	4	14	783	652	902	23	66	8.4	E
3009	19	1	4	14	834	709	978	23	76	9.1	E
3028	19	1	4	14	706	580	784	19	50	7.1	E
3033	19	1	4	14	762	635	849	29	54	7.1	E
3055	19	1	4	14	700	579	803	12	68	9.7	E
9116	19	1	4	14	804	661	905	19	60	7.5	E
9126	19	1	4	14	716	530	935	59	90	12.6	E
3008	37	1	3	14	608	487	703	99	9	1.5	E
3011	37	1	3	14	609	559	666	77	6	1.0	E
3807	37	1	3	14	602	541	716	10	29	4.8	E
3816	37	1	3	14	645	558	718	10	27	4.2	E
5037	37	1	3	14	488	375	600	10	56	11.5	E
3011	53	1	4	14	713	650	781	5	57	8.0	E
3019	53	1	3	14	812	594	937	4	155	19.1	E
			Mean		695	583	796	30	56	8	
			min		488	375	590	4	6	1	
			max		910	755	1000	118	155	19	
			stdev		105	88	126	34	34	4	
			count		19	19	19	19	19	19	
4059	12	1	6	28	853	700	1050	10	125	14.7	E
4109	12	1	3	28	853	700	1050	10	125	14.7	E
3017	16	1	5	28	681	650	735	7	32	4.7	E
3013	23	1	4	28	588	549	638	3	44	7.5	E
3014	23	1	4	28	605	579	646	4	29	4.8	E
3018	28	1	3	28	645	507	754	10	87	13.5	E
3019	28	1	3	28	645	507	754	10	87	13.5	E
4024	28	1	4	28	712	595	785	19	50	7.0	E
5006	28	1	4	28	524	325	666	3	178	34.0	E
5025	28	1	4	28	608	527	680	11	51	8.4	E
5803	28	1	3	28	595	495	715	5	96	16.1	E
5805	28	1	4	28	710	533	970	101	82	11.5	E
5005	41	1	3	28	584	530	700	15	45	7.7	E
5008	41	1	4	28	611	545	685	6	61	10.0	E
1682	50	1	4	28	851	628	1135	43	129	15.2	E
3027	56	1	3	28	787	716	900	8	68.5	8.7	E
			Mean		671	558	798	17	81	12	
			min		524	325	638	3	29	5	
			max		853	700	1135	101	178	34	
			stdev		106	93	166	25	43	7	
			count		15	15	15	15	15	15	

Table B9 – GPS Flexural Strength – By Strength Level

SHRP_ID	STATE_CODE	CONSTRUCTION_NO	LAYER_NO	FLEXURAL_STRENGTH_AGE	FLEXURAL_STRENGTH_MEAN	FLEXURAL_STRENGTH_MIN	FLEXURAL_STRENGTH_MAX	NO_FLEXURAL_STRENGTH_TESTS	FLEXURAL_STRENGTH_STD_DEV	COEF. OF VARIATION, %	RECORD_STATUS
5037	37	1	3	14	488	375	600	10	56	11.5	E
5006	28	1	4	28	524	325	666	3	178	34.0	E
3010	32	1	4	13	545	313	725	41	81	14.9	E
3010	6	1	4	14	565	505	590	6	35	6.2	E
5005	41	1	3	28	584	530	700	15	45	7.7	E
3013	23	1	4	28	588	549	638	3	44	7.5	E
5803	28	1	3	28	595	495	715	5	96	16.1	E
3807	37	1	3	14	602	541	716	10	29	4.8	E
3014	23	1	4	28	605	579	646	4	29	4.8	E
5025	28	1	4	28	608	527	680	11	51	8.4	E
3008	37	1	3	14	608	487	703	99	9	1.5	E
3011	37	1	3	14	609	559	666	77	6	1.0	E
3060	20	1	4	5	610	559	708	14	49	8.0	E
5008	41	1	4	28	611	545	685	6	61	10.0	E
3013	20	1	4	6	612	540	696	20	48	7.8	E
5046	19	1	4	7	613	517	664	16	39	6.4	E
5334	48	1	4	7	619	563	666	28	28	4.5	E
5025	16	1	4	14	625	585	675	6	38	6.1	E
3015	20	1	3	7	644	490	837	58	83	12.9	E
4008	9	1	3	9	645	600	680	4	37	5.7	E
3018	28	1	3	28	645	507	754	10	87	13.5	E
3019	28	1	3	28	645	507	754	10	87	13.5	E
3816	37	1	3	14	645	558	718	10	27	4.2	E
0600	19	1	4	14	649	485	868	118	78	12.0	E
			Mean		604	510	698	24	55	9	
			min		488	313	590	3	6	1	
			max		649	600	868	118	178	34	
			stdev		40	74	62	32	36	7	
			count		24	24	24	24	24	24	
3028	19	1	4	14	706	580	784	19	50	7.1	E
5317	48	1	4	7	706	604	807	326	22	3.1	E
5805	28	1	4	28	710	533	970	101	82	11.5	E
4024	28	1	4	28	712	595	785	19	50	7.0	E
3011	53	1	4	14	713	650	781	5	57	8.0	E
9126	19	1	4	14	716	530	935	59	90	12.6	E
5283	48	1	5	7	717	638	795	144	59	8.2	E
5284	48	1	5	7	722	591	878	59	85	11.8	E
3033	19	1	4	14	762	635	849	29	54	7.1	E
5301	48	1	4	7	768	590	945	241	86	11.2	E
5328	48	1	4	7	769	475	1010	179	102	13.3	E
3006	19	1	4	14	783	652	902	23	66	8.4	E
3027	56	1	3	28	787	716	900	8	68.5	8.7	E
5310	48	1	4	7	795	664	925	72	114	14.3	E
9116	19	1	4	14	804	661	905	19	60	7.5	E
3019	53	1	3	14	812	594	937	4	155	19.1	E
3009	19	1	4	14	834	709	978	23	76	9.1	E
1682	50	1	4	28	851	628	1135	43	129	15.2	E
4059	12	1	6	28	853	700	1050	10	125	14.7	E
4109	12	1	3	28	853	700	1050	10	125	14.7	E
5908	17	1	3	14	910	755	1000	25	77	8.5	E
	All Ages		Mean		775	629	920	68	83	11	
	All Ages		min		706	475	781	4	22	3	
	All Ages		max		910	755	1135	326	155	19	
	All Ages		stdev		61	69	98	86	33	4	
	All Ages		count		21	21	21	21	21	21	

Table B10 – GPS Flexural Strength – By State

SHRP_ID	STATE_CODE	CONSTRUCTION_NO	LAYER_NO	FLEXURAL_STRENGTH_AGE	FLEXURAL_STRENGTH_MEAN	FLEXURAL_STRENGTH_MIN	FLEXURAL_STRENGTH_MAX	NO_FLEXURAL_STRENGTH_TESTS	FLEXURAL_STRENGTH_STD_DEV	COEF. OF VARIATION, %	RECORD_STATUS
0600	19	1	4	14	649	485	868	118	78	12.0	E
3006	19	1	4	14	783	652	902	23	66	8.4	E
3009	19	1	4	14	834	709	978	23	76	9.1	E
3028	19	1	4	14	706	580	784	19	50	7.1	E
3033	19	1	4	14	762	635	849	29	54	7.1	E
3055	19	1	4	14	700	579	803	12	68	9.7	E
5042	19	1	4	7	686	561	772	8	58	8.5	E
5046	19	1	4	7	613	517	664	16	39	6.4	E
9116	19	1	4	14	804	661	905	19	60	7.5	E
9126	19	1	4	14	716	530	935	59	90	12.6	E
	All Ages		Mean		725	591	846	33	64	9	
	All Ages		min		613	485	664	8	39	6	
	All Ages		max		834	709	978	118	90	13	
	All Ages		stdev		70	72	92	33	15	2	
	All Ages		count		10	10	10	10	10	10	
3018	28	1	3	28	645	507	754	10	87	13.5	E
3019	28	1	3	28	645	507	754	10	87	13.5	E
3099	28	1	5	7	679	610	771	15	41	6.0	E
4024	28	1	4	28	712	595	785	19	50	7.0	E
5006	28	1	4	28	524	325	666	3	178	34.0	E
5025	28	1	4	28	608	527	680	11	51	8.4	E
5803	28	1	3	28	595	495	715	5	96	16.1	E
5805	28	1	4	28	710	533	970	101	82	11.5	E
	All Ages		Mean		640	512	762	22	84	14	
	All Ages		min		524	325	666	3	41	6	
	All Ages		max		712	610	970	101	178	34	
	All Ages		stdev		63	87	94	32	43	9	
	All Ages		count		8	8	8	8	8	8	
3008	37	1	3	14	608	487	703	99	9	1.5	E
3011	37	1	3	14	609	559	666	77	6	1.0	E
3807	37	1	3	14	602	541	716	10	29	4.8	E
3816	37	1	3	14	645	558	718	10	27	4.2	E
5037	37	1	3	14	488	375	600	10	56	11.5	E
			Mean		590	504	681	41	25	5	
			min		488	375	600	10	6	1	
			max		645	559	718	99	56	11	
			stdev		60	78	50	43	20	4	
			count		5	5	5	5	5	5	
5283	48	1	5	7	717	638	795	144	59	8.2	E
5284	48	1	5	7	722	591	878	59	85	11.8	E
5301	48	1	4	7	768	590	945	241	86	11.2	E
5310	48	1	4	7	795	664	925	72	114	14.3	E
5317	48	1	4	7	706	604	807	326	22	3.1	E
5328	48	1	4	7	769	475	1010	179	102	13.3	E
5334	48	1	4	7	619	563	666	28	28	4.5	E
			Mean		728	589	861	150	71	9	
			min		619	475	666	28	22	3	
			max		795	664	1010	326	114	14	
			stdev		58	60	115	108	36	4	
			count		7	7	7	7	7	7	

Table B11 – SPS Flexural Strength – All Data

State	Design 14-day Flexural Strength, psi	14-Day COUNT Flexural Strength, psi	14-Day Standard Deviation, psi	14-Day Coef. Of Variation, %	28-Day COUNT Flexural Strength, psi	28-Day Standard Deviation, psi	14-Day Coef. Of Variation, %	1-Year COUNT Flexural Strength, psi	1-Year Standard Deviation, psi	1-Year Coef. Of Variation, %
4	550	572	10	1.8	665	30	4.6	867	72	8.2
4	900	837	58	6.9	868	55	6.4	966	67	7.0
5	550	545	30	5.4	478	55	11.4	640	54	8.4
5	900	-	-	-	-	-	-	-	-	-
8	550	526	45	8.5	578	54	9.4	668	30	4.5
8	900	906	58	6.4	928	91	9.8	959	76	7.9
10	550	657	101	15.3	767	146	19.0	797	153	19.2
10	900	757	152	20.1	883	266	30.1	837	246	29.4
19	550	467	31	6.6	547	38	6.9	627	47	7.5
19	900	753	47	6.3	747	25	3.4	863	83	9.6
20	550	613	47	7.7	648	40	6.2	720	31	4.3
20	900	843	50	5.9	903	62	6.8	902	73	8.1
26	550	-	-	-	-	-	-	888	46	5.2
26	900	-	-	-	-	-	-	947	65	6.8
32	550	522	33	6.2	562	32	5.7	632	74	11.7
32	900	785	87	11.1	838	53	6.3	872	42	4.8
37	550	-	-	-	-	-	-	-	-	-
37	900	-	-	-	1007	74	7.3	1036	28	2.7
39	550	684	56	8.2	804	92	11.4	904	49	5.4
39	900	614	153	24.9	834	53	6.4	944	13	1.3
53	550	485	55	11.3	617	76	12.3	676	72	10.7
53	900	831	35	4.2	945	85	9.0	808	71	8.8
55	550	633	28	4.5	670	28	4.2	-	-	-
55	900	884	53	5.9	-	-	-	-	-	-
	Average		59	8.8		71	9.3		70	8.6
	Min		10	1.8		25	3.4		13	1.3
	Max		153	24.9		266	30.1		246	29.4
	ST Deviation		39	5.7		55	6.2		51	6.2
	VC, %		65.1	64.8		77.7	66.8		72.9	72.1
	Count		19	19		19	19		20	20

Table B12 – SPS Flexural Strength – Sorted by Design Strength

State	Design 14-day Flex Str, psi	14-Day COUNT Flexural Strength, psi	14-Day Std Dev, psi	14-Day Coef. of Variation, %	28-Day COUNT Flexural Strength, psi	28-Day Standard Deviation, psi	14-Day Coef. Of Variation, %	1-Year COUNT Flexural Strength, psi	1-Year Standard Deviation, psi	1-Year Coef. Of Variation, %
4	550	572	10	1.8	665	30	4.6	867	72	8.2
5	550	545	30	5.4	478	55	11.4	640	54	8.4
8	550	526	45	8.5	578	54	9.4	668	30	4.5
10	550	657	101	15.3	767	146	19.0	797	153	19.2
19	550	467	31	6.6	547	38	6.9	627	47	7.5
20	550	613	47	7.7	648	40	6.2	720	31	4.3
26	550	-	-	-	-	-	-	888	46	5.2
32	550	522	33	6.2	562	32	5.7	632	74	11.7
37	550	-	-	-	-	-	-	-	-	-
39	550	684	56	8.2	804	92	11.4	904	49	5.4
53	550	485	55	11.3	617	76	12.3	676	72	10.7
55	550	633	28	4.5	670	28	4.2	-	-	-
	Average	570	43	7.6	634	59	9.1	742	63	8.5
	Min	467	10	1.8	478	28	4.2	627	30	4.3
	Max	684	101	15.3	804	146	19.0	904	153	19.2
	ST Deviation	74	24	3.7	100	37	4.6	112	35	4.5
	VC, %	12.9	56.2	49.5	15.7	62.5	50.4	15.1	56.5	53.2
	COUNT	10	10	10	10	10	10	10	10	10
4	900	837	58	6.9	868	55	6.4	966	67	7.0
5	900	-	-	-	-	-	-	-	-	-
8	900	906	58	6.4	928	91	9.8	959	76	7.9
10	900	757	152	20.1	883	266	30.1	837	246	29.4
19	900	753	47	6.3	747	25	3.4	863	83	9.6
20	900	843	50	5.9	90l3	62	6.8	902	73	8.1
26	900	-	-	-	-	-	-	947	65	6.8
32	900	785	87	11.1	838	53	6.3	872	42	4.8
37	900	-	-	-	1007	74	7.3	1036	28	2.7
39	900	614	153	24.9	834	53	6.4	944	13	1.3
53	900	831	35	4.2	945	85	9.0	808	71	8.8
55	900	884	53	5.9	-	-	-	-	-	-
	Average	801	77	10.2	884	85	9.5	913	76	8.6
	Min	614	35	4.2	747	25	3.4	808	13	1.3
	Max	906	153	24.9	1007	266	30.1	1036	246	29.4
	ST Deviation	88	45	7.3	75	71	7.9	69	64	7.8
	VC, %	10.9	58.5	71.7	8.5	83.1	83.5	7.6	83.6	89.7
	COUNT	9	9	9	9	9	9	10	10	10

Table B13 – GPS Split Tensile Strength – All Data

SHRP_ID	STATE_CODE	CONSTRUCTION_NO	LAYER_NO	TENSILE_STRENGTH_AGE	TENSILE_STRENGTH_MEAN	TENSILE_STRENGTH_MAX	TENSILE_STRENGTH_MIN	NO_TENSILE_STRENGTH_TESTS	TENSILE_STRENGTH_STD_DEV	COEF. OF VARIATION, %	RECORD_STATUS
4059	12	1	6	28	459	562	385	15	53	11.5	E
3023	16	1	4	14	422	500	305	78			E
5807	24	1	4	28	370	370	370	1			E
3005	27	1	4	360	507	510	505	2			E
3007	27	1	4	360	555	565	545	2			E
3009	27	1	4	360	605	625	585	2			E
3010	27	1	4	28	490	490	490	2			E
3012	27	1	3	28	525	545	505	2			E
5323	48	1	4	7	474	497	450				E
5335	48	1	4	7	474	497	450				E
	All Ages		Mean		488	516	459	13	53		
	All Ages		min		370	370	305	1	53		
	All Ages		max		605	625	585	78	53		
	All Ages		stdev		66	67	85	27			
	All Ages		count		10	10	10	8	1		
Duplicate Data For Another Section on Same Project											
4109	12	1	3	28	459	562	385	15	53		E

Table B14 – SPS Cylinder Split Tensile Strength

a) All Data

State	Design 14-day Flexural Strength, psi	14-Day Average Split Tensile Strength, psi	14-Day Standard Deviation, psi	14-Day Coef. Of Variation, %	28-Day Average Split Tensile Strength, psi	28-Day Standard Deviation, psi	28-Day Coef. Of Variation, %	1-Year Average Split Tensile Strength, psi	1-Year Standard Deviation, psi	1-Year Coef. Of Variation, %
4	550	375	25	6.7	375	10	2.7	490	52	10.6
4	900	487	18	3.6	545	39	7.2	770	90	11.7
5	550	-	-	-	-	-	-	-	-	-
5	900	-	-	-	-	-	-	-	-	-
8	550	332	34	10.3	367	85	23.1	497	63	12.7
8	900	474	76	15.9	559	46	8.2	647	65	10.0
10	550	-	-	-	-	-	-	461	22	4.7
10	900	-	-	-	-	-	-	539	102	18.9
19	550	337	51	15.2	-	-	-	413	55	13.3
19	900	487	40	8.3	537	42	7.8	-	-	-
20	550	481	39	8.1	525	76	14.5	504	55	11.0
20	900	580	55	9.5	600	42	7.0	592	81	13.6
26	550	-	-	-	-	-	-	-	-	-
26	900	-	-	-	-	-	-	-	-	-
32	550	403	68	16.8	432	75	17.4	541	88	16.2
32	900	497	18	3.5	570	46	8.0	857	19	2.3
37	550	376	39	10.4	487	12	2.5	619	60	9.7
37	900	507	43	8.6	547	28	5.0	708	39	5.5
38	550	-	-	-	-	-	-	-	-	-
38	900	-	-	-	-	-	-	-	-	-
39	550	370	34	9.1	-	-	-	561	46	8.2
39	900	491	21	4.3	-	-	-	-	-	-
53	550	399	46	11.5	443	20	4.5	546	36	6.7
53	900	570	34	5.9	642	38	5.9	664	71	10.7
55	550	383	56	14.5	491	58	11.9	-	-	-
55	900	564	59	10.4	522	138	26.4	-	-	-
	Average	451	42	9.6	509	50	10.1	588	59	10.4
	Min	332	18	3.5	367	10	2.5	413	19	2.3
	Max	580	76	16.8	642	138	26.4	857	102	18.9
	ST Deviation	80	17	4	78	33	7.2	118	24	4.3
	VC, %	17.8	39.5	42.4	15.3	65.6	71.4	20.1	40.2	41.4
	Count	18	18	18	15	15	15	16	16	16

b) Sorted by Design Strength

State	Design 14-day Flex Str, psi	14-Day Average S.T. Strength, psi	14-Day Std Deviation, psi	14-Day Coef. of Var, %	28-Day Average S.T. Strength, psi	28-Day Std Deviation, psi	28-Day Coef. of Var, %	1-Year Average S.T. Strength, psi	1-Year Std Deviation, psi	1-Year Coef. of Var, %
4	550	375	25	6.7	375	10	2.7	490	52	10.6
5	550	-	-	-	-	-	-	-	-	-
8	550	332	34	10.3	367	85	23.1	497	63	12.7
10	550	-	-	-	-	-	-	461	22	4.7
19	550	337	51	15.2	-	-	-	413	55	13.3
20	550	481	39	8.1	525	76	14.5	504	55	11.0
26	550	-	-	-	-	-	-	-	-	-
32	550	403	68	16.8	432	75	17.4	541	88	16.2
37	550	376	39	10.4	487	12	2.5	619	60	9.7
38	550	-	-	-	-	-	-	-	-	-
39	550	370	34	9.1	-	-	-	561	46	8.2
53	550	399	46	11.5	443	20	4.5	546	36	6.7
55	550	383	56	14.5	491	58	11.9	-	-	-
	Average	384	44	11.4	446	48	10.9	515	53	10.3
	Min	332	25	6.7	367	10	2.5	413	22	4.7
	Max	481	68	16.8	525	85	23.1	619	88	16.2
	ST Deviation	44	13	3.4	60	33	8.0	60	18	3.5
	VC, %	11.4	30.1	30.2	13.4	68.7	73.4	11.7	34.6	34.3
	Count	9	9	9	7	7	7	9	9	9
4	900	487	18	3.6	545	39	7.2	770	90	11.7
5	900	-	-	-	-	-	-	-	-	-
8	900	474	76	15.9	559	46	8.2	647	65	10.0
10	900	-	-	-	-	-	-	539	102	18.9
19	900	487	40	8.3	537	42	7.8	-	-	-
20	900	580	55	9.5	600	42	7.0	592	81	13.6
26	900	-	-	-	-	-	-	-	-	-
32	900	497	18	3.5	570	46	8.0	857	19	2.3
37	900	507	43	8.6	547	28	5.0	708	39	5.5
38	900	-	-	-	-	-	-	-	-	-
39	900	491	21	4.3	-	-	-	-	-	-
53	900	570	34	5.9	642	38	5.9	664	71	10.7
55	900	564	59	10.4	522	138	26.4	-	-	-
	Average	517	40	7.8	565	52	9.4	682	67	10.4
	Min	474	18	3.5	522	28	5.0	539	19	2.3
	Max	580	76	15.9	642	138	26.4	857	102	18.9
	ST Deviation	42	20	4.0	39	35	7.0	107	29	5.4
	VC, %	8.0	49.9	51.3	6.9	67.3	73.6	15.7	43.4	52.1
	Count	9	9	9	8	8	8	7	7	7

Table B15 – SPS Split Tensile Strength

a) All Data

State	Design 14-day Flexural Strength, psi	14-Day Average Split Tensile Strength, psi	14-Day Standard Deviation, psi	14-Day Coef. Of Variation, %	28-Day Average Split Tensile Strength, psi	28-Day Standard Deviation, psi	28-Day Coef. Of Variation, %	1-Year Average Split Tensile Strength, psi	1-Year Standard Deviation, psi	1-Year Coef. Of Variation, %
4	550	468	65	13.9	413	53	12.9	647	57	8.8
4	900	583	43	7.4	532	59	11.1	655	52	8.0
5	550	-	-	-	440	81	18.3	696	173	24.8
5	900	-	-	-	598	101	16.9	607	127	20.9
8	550	461	101	21.8	495	68	13.7	625	61	9.7
8	900	620	38	6.2	676	151	22.4	806	80	10.0
10	550	599	108	17.9	472	68	14.4	609	99	16.3
10	900	545	59	10.8	525	18	3.5	570	99	17.4
19	550	320	53	16.5	387	55	14.2	430	63	14.6
19	900	455	39	8.5	528	22	4.2	555	25	4.5
20	550	-	-	-	-	-	-	-	-	-
20	900	-	-	-	-	-	-	-	-	-
26	550	510	17	3.3	420	46	10.9	-	-	-
26	900	-	-	-	-	-	-	772	80	10.3
32	550	334	59	17.5	343	20	5.7	473	31	6.4
32	900	491	24	4.9	535	20	3.7	616	63	10.2
37	550	380	71	18.8	423	55	13.0	666	125	18.8
37	900	526	16	3.0	576	44	7.6	682	100	14.7
38	550	438	38	8.7	485	62	12.8	-	-	-
38	900	647	34	5.3	-	-	-	901	102	11.3
39	550	391	13	3.4	507	69	13.6	-	-	-
39	900	540	109	20.2	531	139	26.1	-	-	-
53	550	453	26	5.8	471	38	8.1	600	68	11.3
53	900	755	24	3.2	789	47	5.9	820	65	8.0
55	550	415	38	9.1	470	41	8.7	-	-	-
55	900	495	85	17.3	558	49	8.7	-	-	-
	Average	496	50	10.6	508	59	11.7	652	82	12.6
	Min	320	13	3.0	343	18	3.5	430	25	4.5
	Max	755	109	21.8	789	151	26.1	901	173	24.8
	ST Deviation	106	30	6.4	98	34	5.9	118	36	5.4
	VC, %	21.4	59.2	60.2	19.2	58.1	50.4	18.1	44.6	42.8
	Count	21	21	21	22	22	22	18	18	18

b) Sorted by Design Strength

State	Design 14-day Flex Str, psi	14-Day Average S.T. Strength, psi	14-Day Std Deviation, psi	14-Day Coef. of Var, %	28-Day Average S.T. Strength, psi	28-Day Std Deviation, psi	28-Day Coef. of Var, %	1-Year Average S.T. Strength, psi	1-Year Std Deviation, psi	1-Year Coef. of Var, %
4	550	468	65	13.9	413	53	12.9	647	57	8.8
5	550	-	-	-	440	81	18.3	696	173	24.8
8	550	461	101	21.8	495	68	13.7	625	61	9.7
10	550	599	108	17.9	472	68	14.4	609	99	16.3
19	550	320	53	16.5	387	55	14.2	430	63	14.6
20	550	-	-	-	-	-	-	-	-	-
26	550	510	17	3.3	420	46	10.9	-	-	-
32	550	334	59	17.5	343	20	5.7	473	31	6.4
37	550	380	71	18.8	423	55	13.0	666	125	18.8
38	550	438	38	8.7	485	62	12.8	-	-	-
39	550	391	13	3.4	507	69	13.6	-	-	-
53	550	453	26	5.8	471	38	8.1	600	68	11.3
55	550	415	38	9.1	470	41	8.7	-	-	-
	Average	434	53	12.4	444	55	12.2	593	85	13.9
	Min	320	13	3.3	343	20	5.7	430	31	6.4
	Max	599	108	21.8	507	81	18.3	696	173	24.8
	ST Deviation	80	31	6.6	49	17	3.4	93	46	6.0
	VC, %	18.4	58.7	53.4	10.9	30.5	27.6	15.8	54.2	43.6
	Count	11	11	11	12	12	12	8	8	8
4	900	583	43	7.4	532	59	11.1	655	52	8.0
5	900	-	-	-	598	101	16.9	607	127	20.9
8	900	620	38	6.2	676	151	22.4	806	80	10.0
10	900	545	59	10.8	525	18	3.5	570	99	17.4
19	900	455	39	8.5	528	22	4.2	555	25	4.5
20	900	-	-	-	-	-	-	-	-	-
26	900	-	-	-	-	-	-	772	80	10.3
32	900	491	24	4.9	535	20	3.7	616	63	10.2
37	900	526	16	3.0	576	44	7.6	682	100	14.7
38	900	647	34	5.3	-	-	-	901	102	11.3
39	900	540	109	20.2	531	139	26.1	-	-	-
53	900	755	24	3.2	789	47	5.9	820	65	8.0
55	900	495	85	17.3	558	49	8.7	-	-	-
	Average	566	47	8.7	585	65	11.0	698	79	11.5
	Min	455	16	3.0	525	18	3.5	555	25	4.5
	Max	755	109	20.2	789	151	26.1	901	127	20.9
	ST Deviation	89	29	5.8	86	49	8.1	119	29	4.9
	VC, %	15.7	62.4	67.2	14.6	75.0	73.7	17.0	37.0	42.2
	Count	10	10	10	10	10	10	10	10	10

Table B16 – GPS Modulus of Elasticity – All Data

SHRP_ID	STATE_CODE	CONSTRUCTION_NO	LAYER_NO	ELASTIC_MOD_MEAN	ELASTIC_MOD_MIN	ELASTIC_MOD_MAX	NO_ELASTIC_MOD_TESTS	ELASTIC_MOD_STD_DEV	COEF. OF VARIATION, %	RECORD_STATUS
7614	4	1	3	3652			1			E
3011	5	1	4	4029						E
3059	5	1	3	3612						E
3073	5	1	3	3743						E
3074	5	1	3	3819						E
4019	5	1	4	3832	3828	3835	2			E
4023	5	1	3	4162						E
4046	5	1	3	4288						E
5803	5	1	4	3717						E
5805	5	1	4	3461						E
3032	8	1	3	3802						E
7776	8	1	3	4177			1			E
4059	12	1	6	4399	3580	5200	15	406	9.2	E
4109	12	1	3	4399	3580	5200	15	406	9.2	E
3007	13	1	3	4216						E
3011	13	1	4	3523						E
3015	13	1	4	3872						E
3016	13	1	4	3704						E
3019	13	1	3	3467						E
3020	13	1	3	4331						E
3006	19	1	4	3730	3314	4250	84	203	5.4	E
3009	19	1	4	4136	3497	4497	99	218	5.3	E
3028	19	1	4	3723	3429	4093	29	179	4.8	E
3033	19	1	4	3594	3189	5195	46	279	7.8	E
3055	19	1	4	4079	3727	4495	38	167	4.1	E
5042	19	1	4	3960	3537	4134	46	125	3.2	E
5046	19	1	4	3738	3224	4455	99	301	8.1	E
9116	19	1	4	3950	3370	4388	67	251	6.4	E
9126	19	1	4	4316	3632	4783	29	277	6.4	E
3016	21	1	4	3981						E
3013	23	1	4	3765	3711	3853	3	71	1.9	E
3014	23	1	4	3535	3237	3739	6	197	5.6	E
3005	27	1	4	4778	4746	4810	2			E
3007	27	1	4	4778	4746	4810	2			E
3009	27	1	4	4778			2			E
3010	27	1	4	4778			2			E
3012	27	1	3	4778			2			E
6702	31	1	3	4204						E
3005	38	1	3	3696	3636	3755	2			E
3006	38	1	3	3561	3561	3561	1			E
3010	49	1	5	4030						E
7083	49	1	5	3949						E
3015	55	1	3	4106			1			E
3027	56	1	3	3600			1			E
3802	83	1	4	3989	3533	4398				E
	All Ages		Mean	3994	3636	4392	24	237	6	
	All Ages		min	3461	3189	3561	1	71	2	
	All Ages		max	4778	4746	5200	99	406	9	
	All Ages		stdev	380	429	515	32	98	2	
	All Ages		count	45	19	19	25	13	13	

Table B17 – SPS Modulus of Elasticity – All Data

State	Design 14-day Flexural Strength, psi	28-Day Average Modulus of Elasticity, ksi	28-Day Standard Deviation, ksi	28-Day Coef. Of Variation, %	1-Year Average Modulus of Elasticity, ksi	1-Year Standard Deviation, ksi	1-Year Coef. Of Variation, %
4	550	4.86	0.33	6.8	4.94	0.13	2.6
4	900	4.74	0.12	2.5	4.35	0.46	10.6
8	550	3.09	1.07	34.6	3.93	0.51	13.0
8	900	4.20	0.82	19.5	4.54	0.32	7.0
10	550	-	-		-	-	-
10	900	-	-		4.66	0.63	13.5
19	550	5.84	0.28	4.8	5.33	0.90	16.9
19	900	8.04	1.17	14.6	5.75	0.71	12.3
26	550	4.07	0.45	11.1	-	-	-
26	900	-	-		-	-	-
32	550	2.58	0.25	9.7	2.98	0.19	6.4
32	900	3.36	0.36	10.7	3.52	0.31	8.8
37	550	5.02	0.67	13.3	4.34	0.54	12.4
37	900	5.42	1.17	21.6	4.92	0.77	15.7
38	550	4.46	0.23	5.2	5.43	1.07	19.7
38	900	4.80	0.36	7.5	5.68	0.31	5.5
39	550	3.09	0.85	27.5	5.01	1.37	27.3
39	900	3.67	0.65	17.7	6.12	1.20	19.6
53	550	4.10	0.18	4.4	4.32	0.31	7.2
53	900	4.63	0.16	3.5	5.30	0.22	4.2
55	550	4.06	0.61	15.0	-	-	-
55	900	4.71	0.56	11.9	-	-	-
	Average	4.46	0.54	12.7	4.77	0.59	11.9
	Min	2.58	0.12	2.5	2.98	0.13	2.6
	Max	8.04	1.17	34.6	6.12	1.37	27.3
	ST Deviation	1.20	0.34	8.5	0.82	0.37	6.5
	VC, %	27.0	63.0	67.2	17.3	63.3	54.6
	Count	19.00	19.00	19.00	17.00	17.00	17.00

Table B18 – SPS Modulus of Elasticity – By Strength Level

State	Design 14-day Flex Str, psi	28-Day Average Mod of Elast, ksi	28-Day Std Deviation, ksi	28-Day Coef. Of Variation, %	1-Year Average Mod of Elast, ksi	1-Year Std Deviation, ksi	1-Year Coef. Of Variation, %
4	550	4.86	0.33	6.8	4.94	0.13	2.6
8	550	3.09	1.07	34.6	3.93	0.51	13.0
10	550	-	-		-	-	-
19	550	5.84	0.28	4.8	5.33	0.90	16.9
26	550	4.07	0.45	11.1	-	-	-
32	550	2.58	0.25	9.7	2.98	0.19	6.4
37	550	5.02	0.67	13.3	4.34	0.54	12.4
38	550	4.46	0.23	5.2	5.43	1.07	19.7
39	550	3.09	0.85	27.5	5.01	1.37	27.3
53	550	4.10	0.18	4.4	4.32	0.31	7.2
55	550	4.06	0.61	15.0	-	-	-
	Average	4.12	0.49	13.2	4.54	0.63	13.2
	Min	2.58	0.18	4.4	2.98	0.13	2.6
	Max	5.84	1.07	34.6	5.43	1.37	27.3
	ST Deviation	1.00	0.30	10.2	0.82	0.44	8.0
	VC, %	24.2	60.8	77.1	18.1	70.8	60.9
	Count	10	10	10	8	8	8
4	900	4.74	0.12	2.5	4.35	0.46	10.6
8	900	4.20	0.82	19.5	4.54	0.32	7.0
10	900	-	-		4.66	0.63	13.5
19	900	8.04	1.17	14.6	5.75	0.71	12.3
26	900	-	-		-	-	-
32	900	3.36	0.36	10.7	3.52	0.31	8.8
37	900	5.42	1.17	21.6	4.92	0.77	15.7
38	900	4.80	0.36	7.5	5.68	0.31	5.5
39	900	3.67	0.65	17.7	6.12	1.20	19.6
53	900	4.63	0.16	3.5	5.30	0.22	4.2
55	900	4.71	0.56	11.9	-	-	-
	Average	4.84	0.60	12.2	4.98	0.55	10.8
	Min	3.36	0.12	2.5	3.52	0.22	4.2
	Max	8.04	1.17	21.6	6.12	1.20	19.6
	ST Deviation	1.35	0.39	6.8	0.81	0.31	5.0
	VC, %	27.9	66.0	56.1	16.4	57.3	46.5
	Count	9.00	9.00	9.00	9.00	9.00	9.00

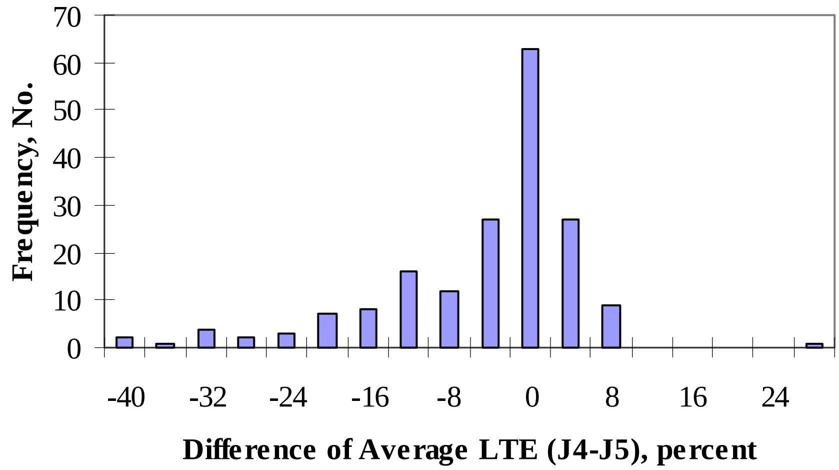


Figure B1. Distribution of Average LTE Difference (J4-J5) for Jointed Pavements with Plain Joints

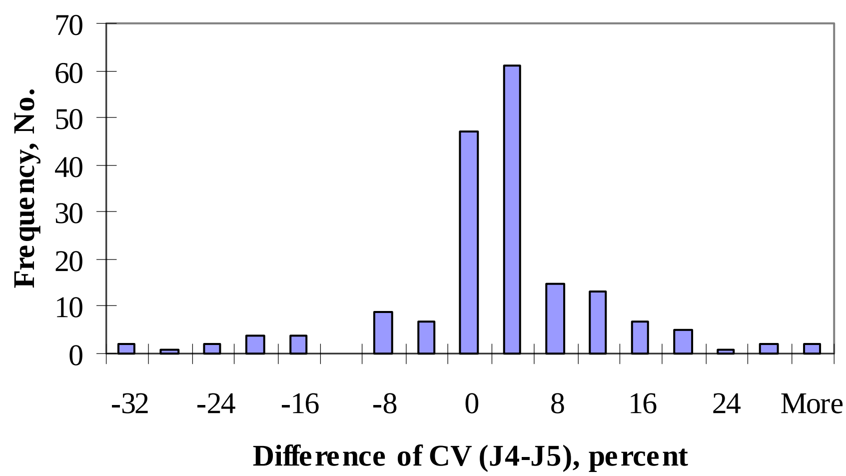


Figure B2. Distribution of CV Difference (J4-J5) for Jointed Pavements with Plain Joints

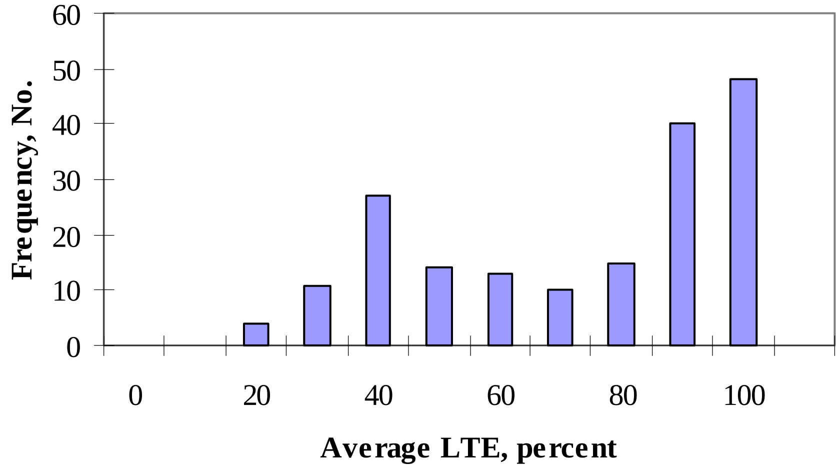


Figure B3. Distribution of Average LTE for Pavements with Plain Joints

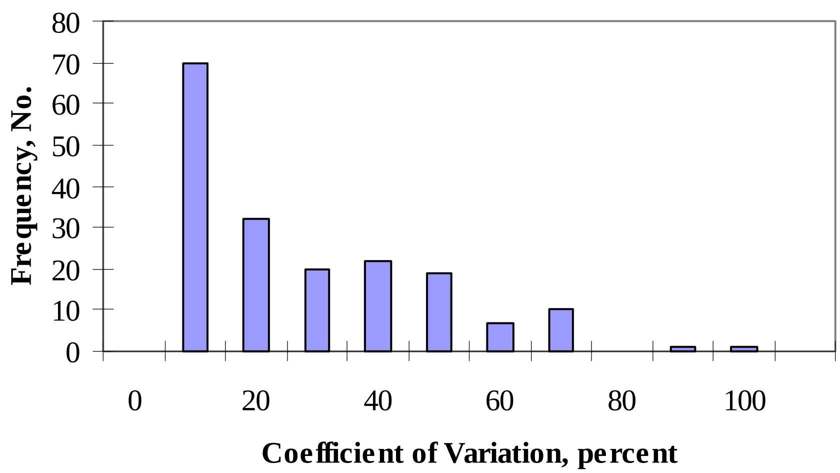


Figure B4. Distribution of CV for Pavements with Plain Joints

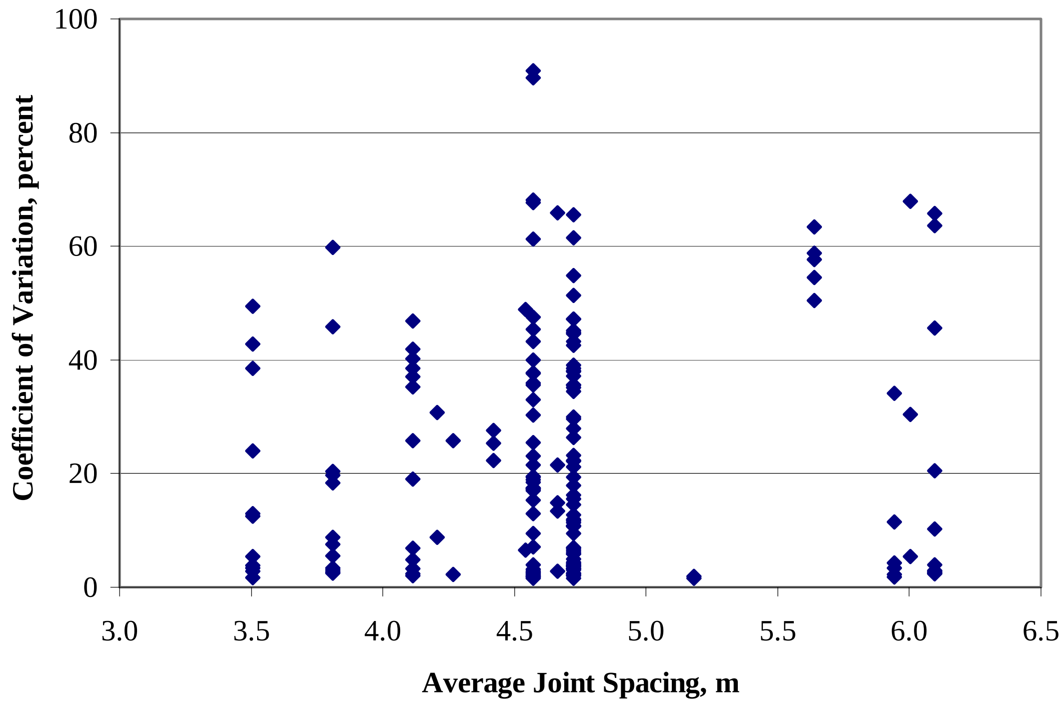


Figure B5. CV vs. Average Joint Spacing for Pavements with Plain Joints

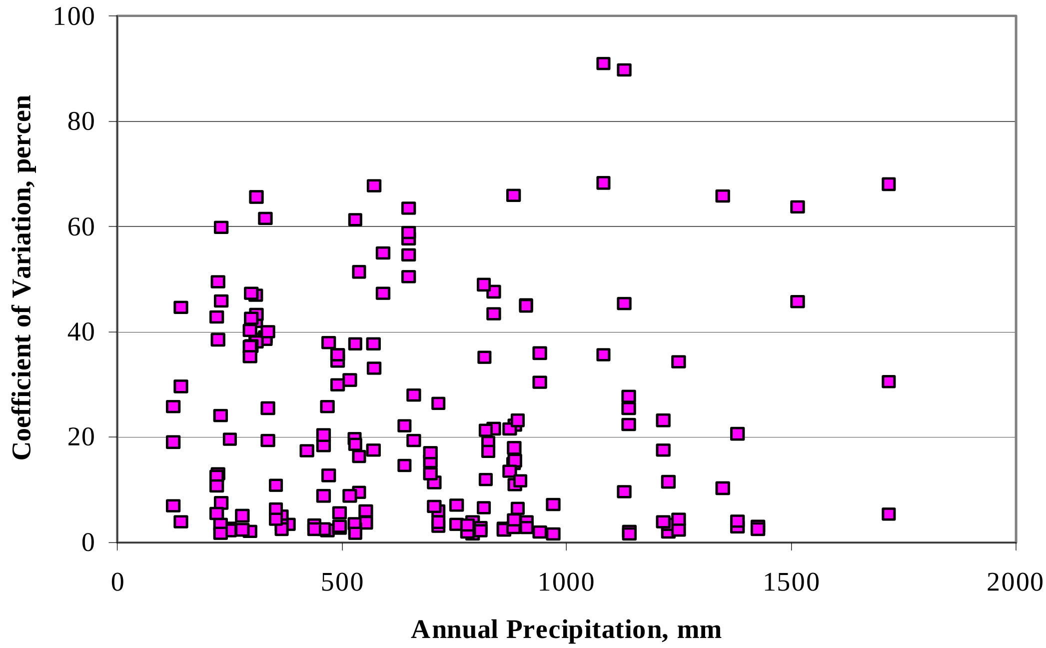


Figure B6. CV vs. Annual Precipitation for Pavements with Plain Joints

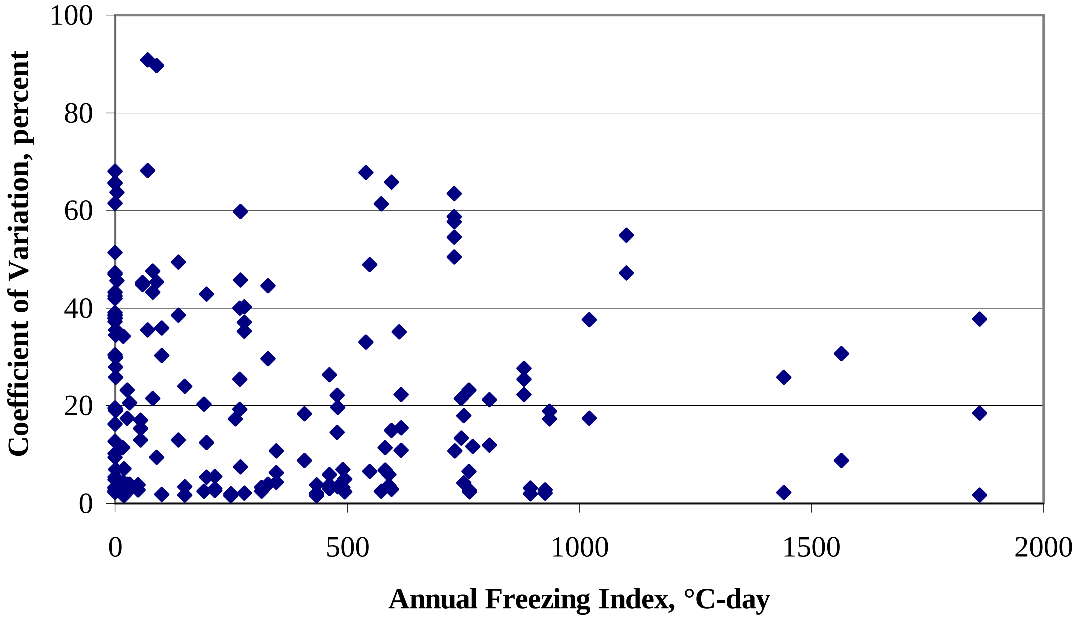


Figure B7. CV vs. Annual Freezing Index for Pavements with Plain Joints

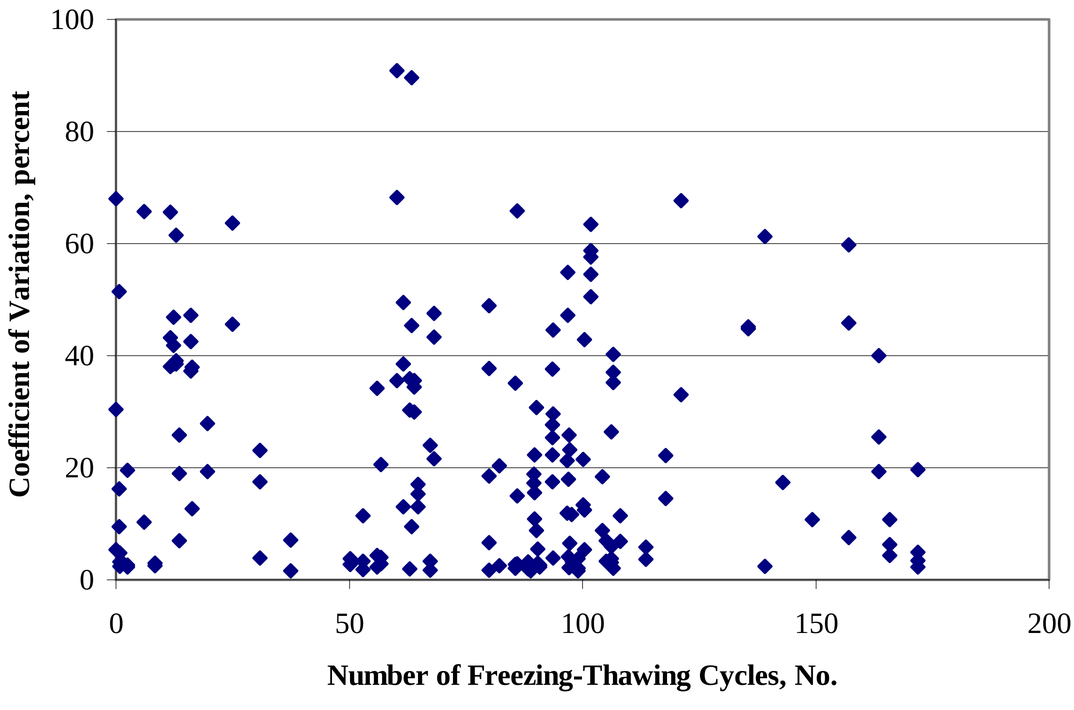


Figure B8. CV vs. Number of Annual Freezing-Thawing Cycles for Pavement with Plain Joints

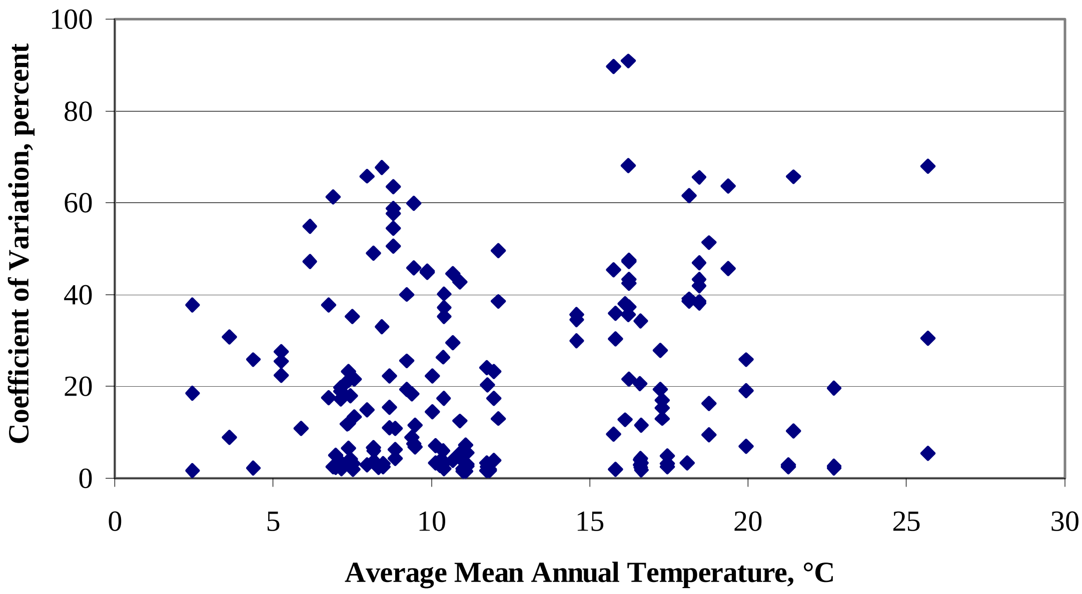


Figure B9. CV vs. Average Mean Annual Temperature for Pavement with Plain Joints

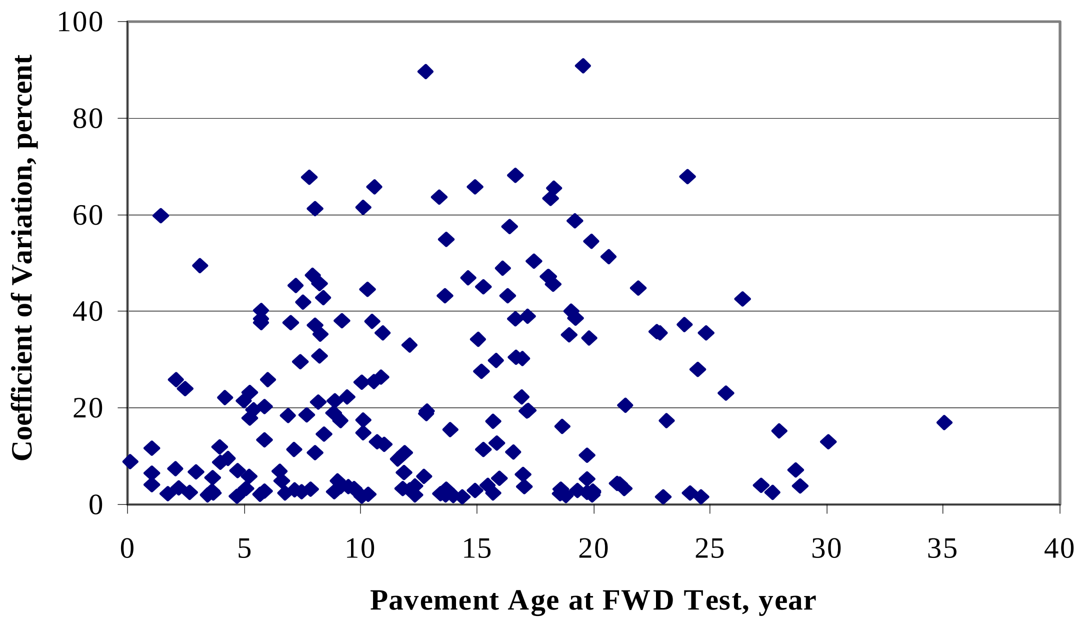


Figure B10. CV vs. Pavement Age for Pavement with Plain Joints

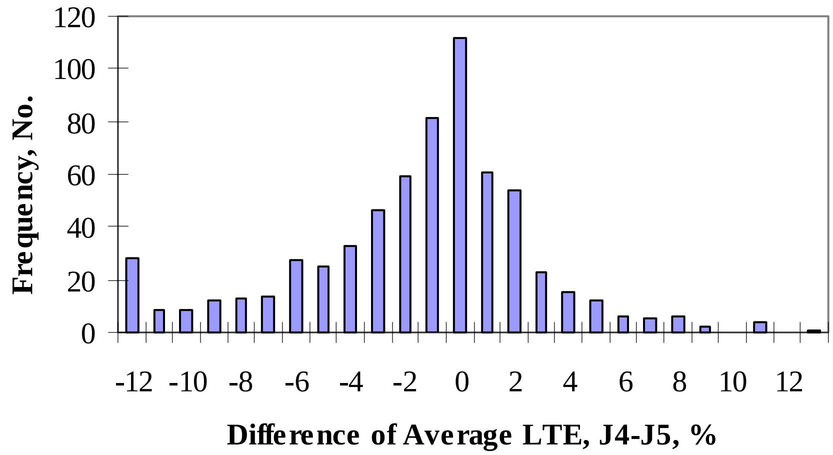


Figure B11. Distribution of Average LTE Difference (J4-J5) for Jointed Pavements with Doweled Joints

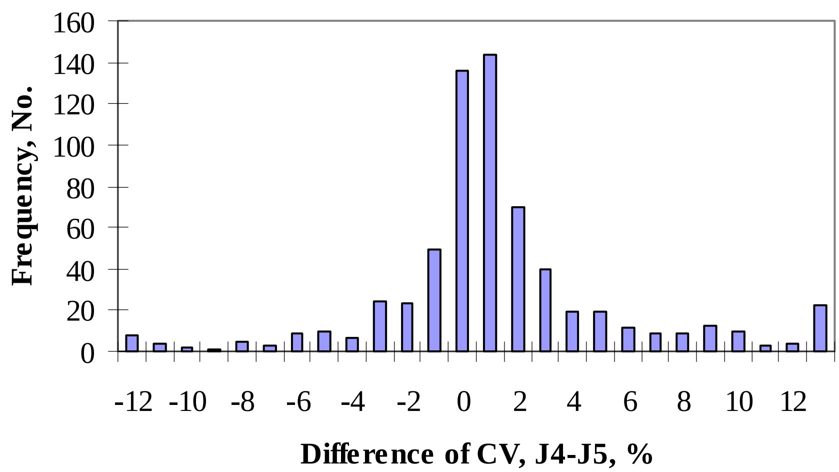


Figure B12. Distribution of CV Difference (J4-J5) for Jointed Pavements with Doweled Joints

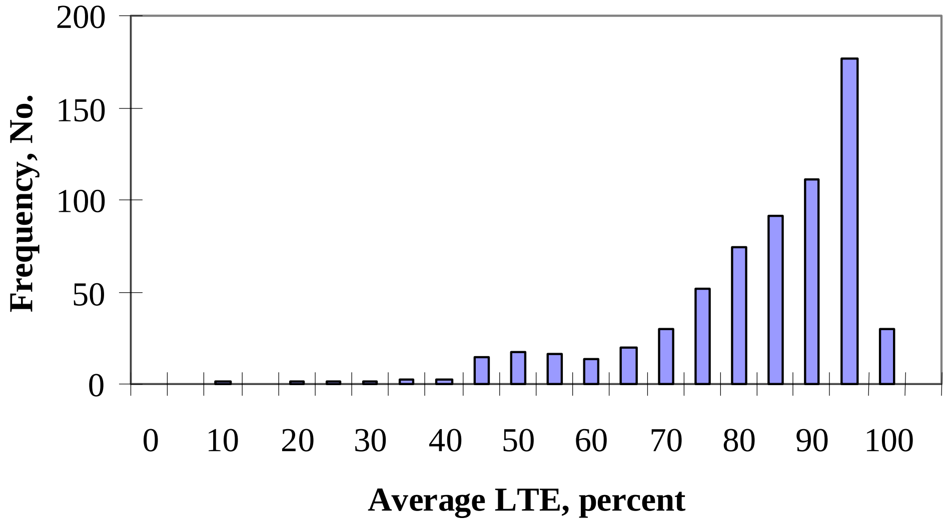


Figure B13. Distribution of Average LTE for Pavements with Doweled Joints

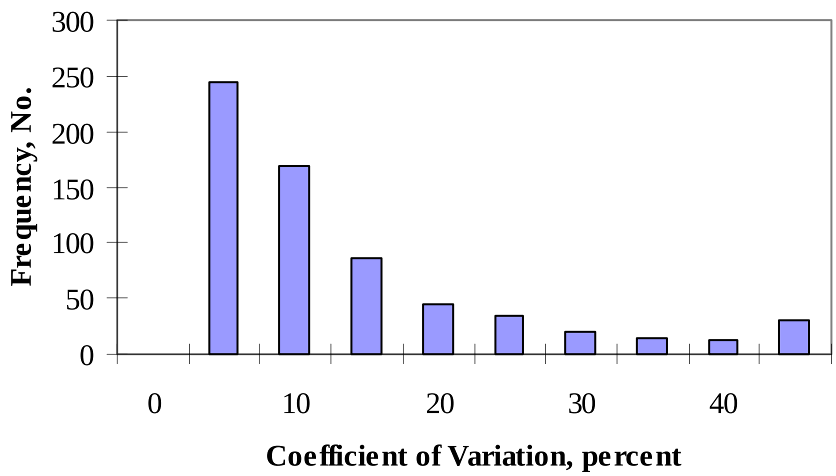


Figure B14. Distribution of CV for Pavements with Doweled Joints

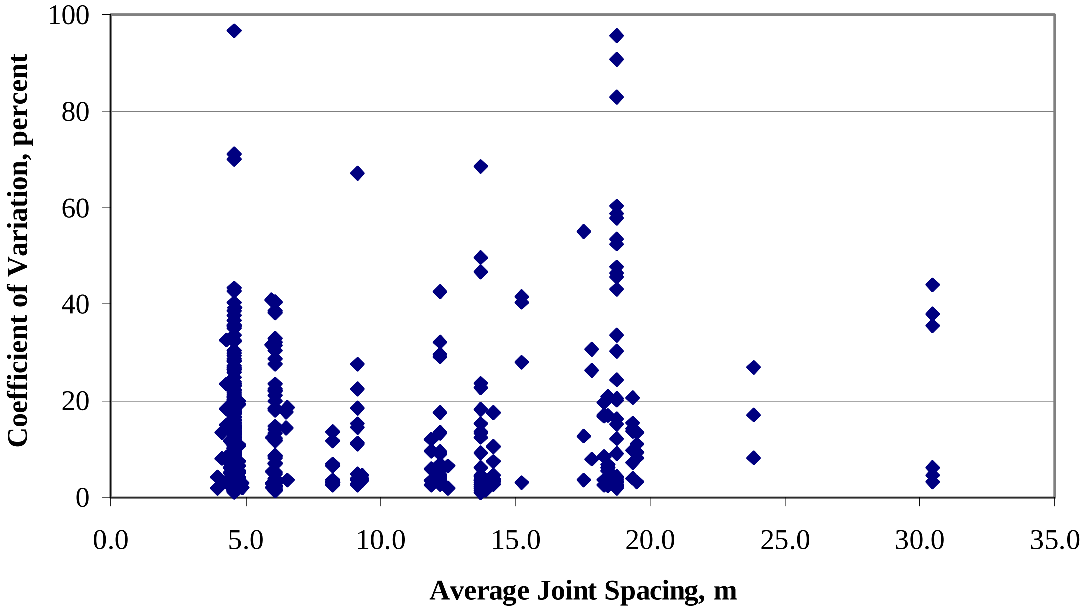


Figure B15. CV vs. Average Joint Spacing for Pavements with Doweled Joints

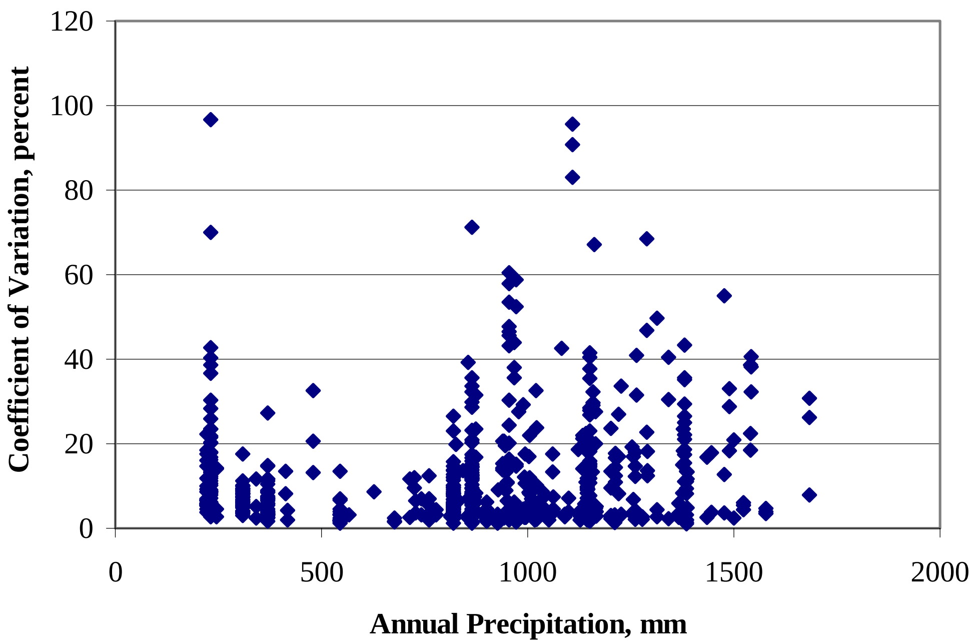


Figure B16. CV vs. Annual Precipitation for Pavements with Doweled Joints

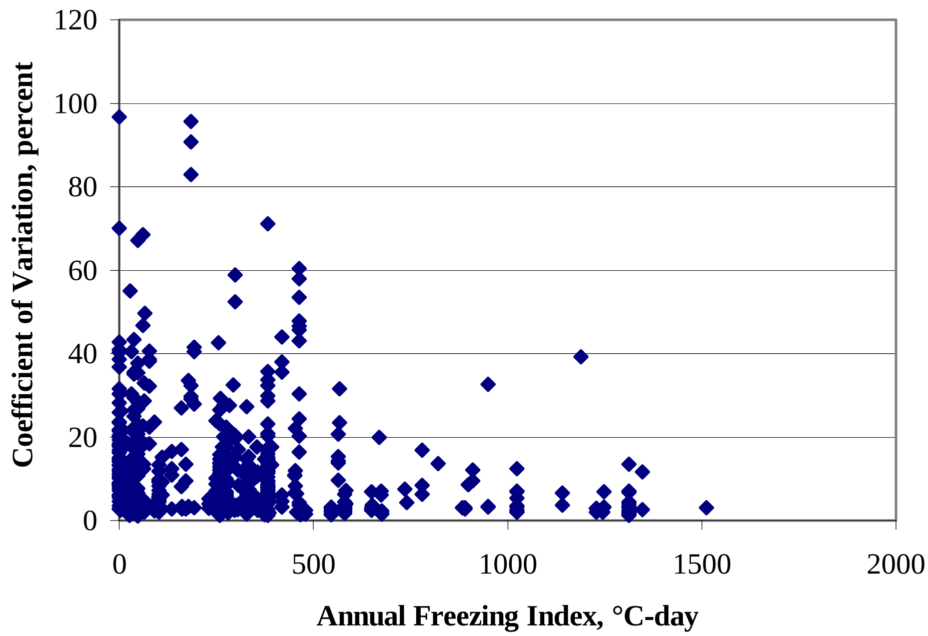


Figure B17. CV vs. Annual Freezing Index for Pavements with Doweled Joints

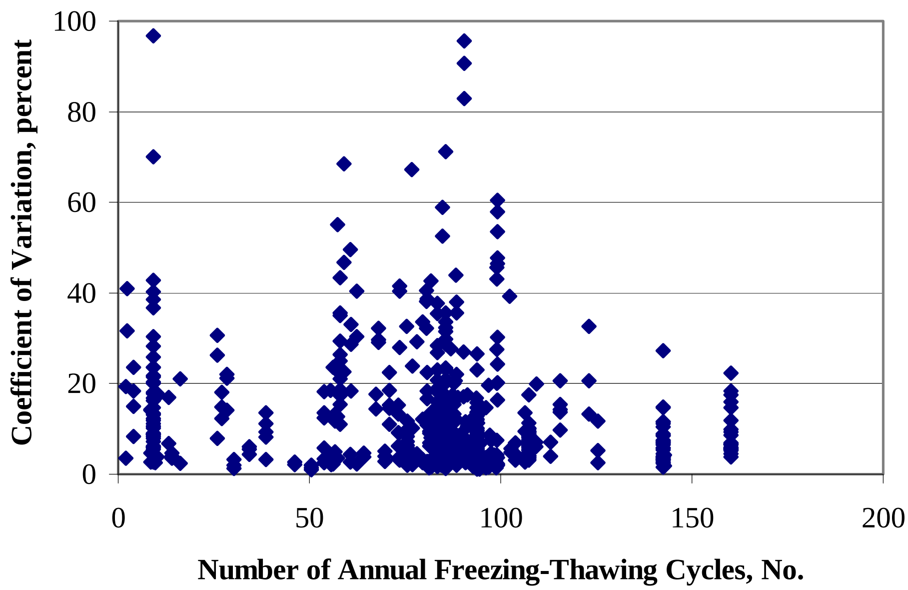


Figure B18. CV vs. Number of Annual Freezing-Thawing Cycles for Pavements with Doweled Joints

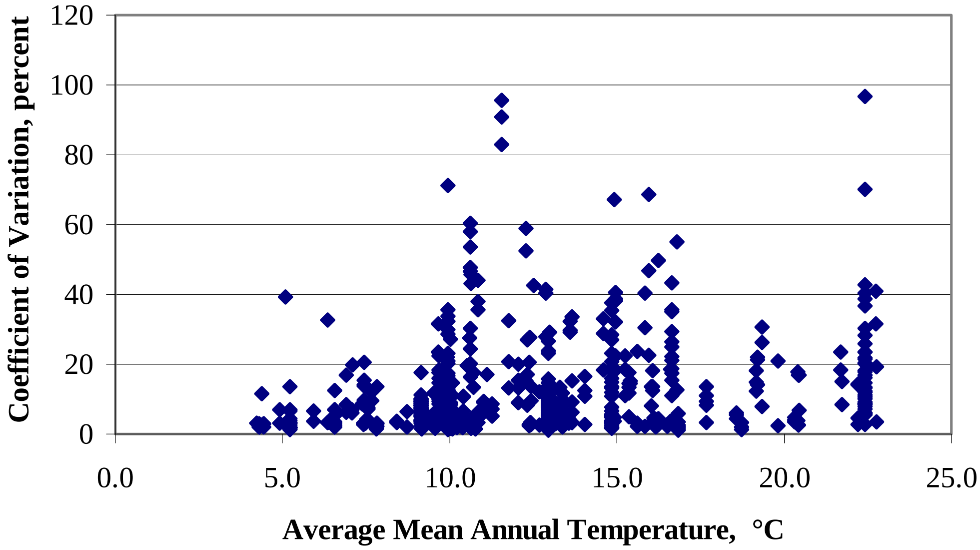


Figure B19. CV vs. Average Mean Annual Temperature for Pavements with Doweled Joints

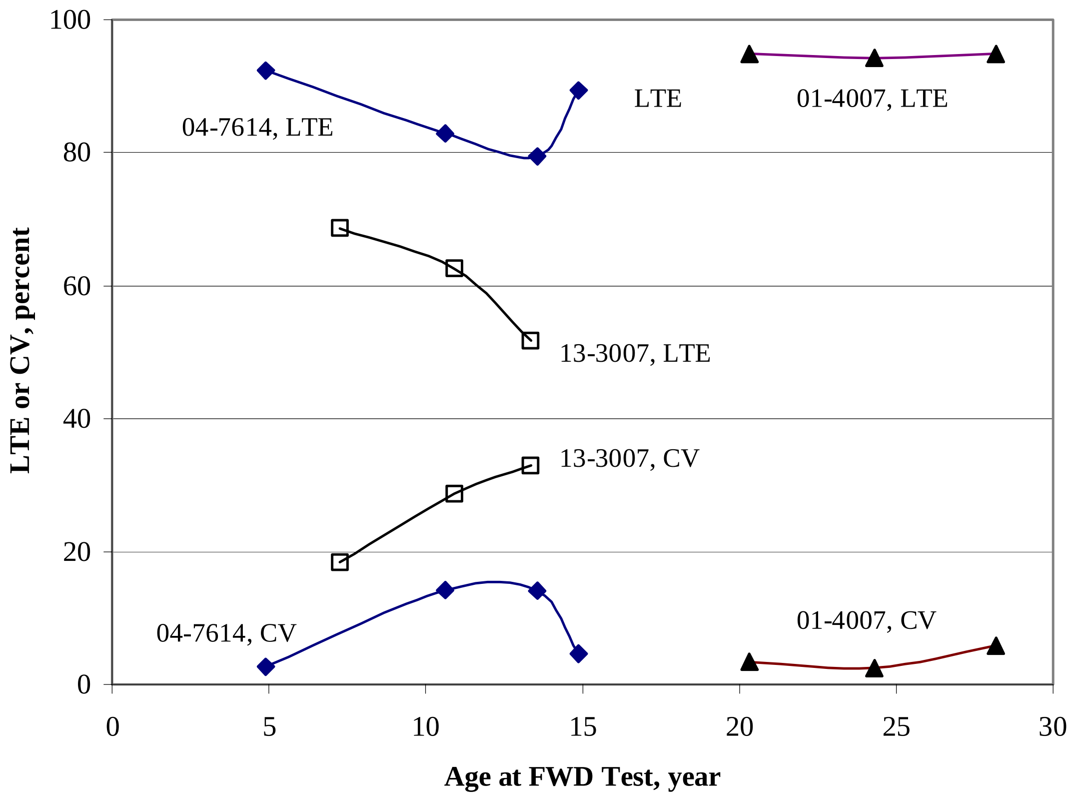


Figure B20. Effects of Age on CV with Respect to the Average LTE for Pavements with Doweled Joints

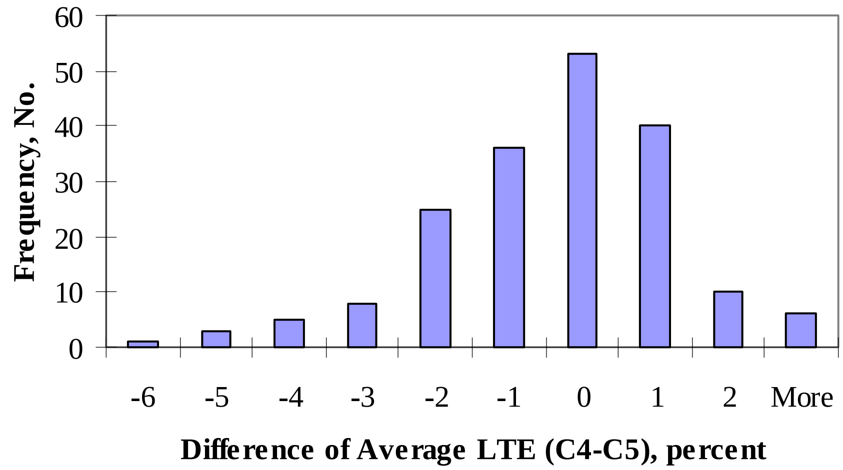


Figure B21. Distribution of Average LTE Difference (C4-C5) for CRC Pavements

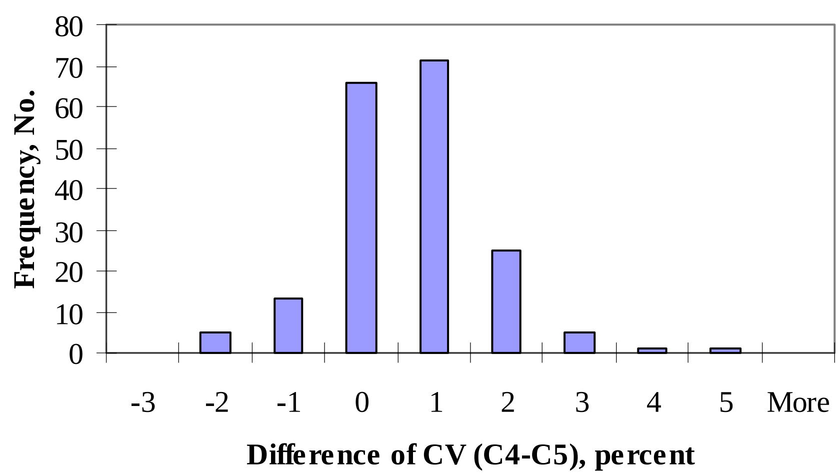


Figure B22. Distribution of CV Difference (C4-C5) for CRC Pavements

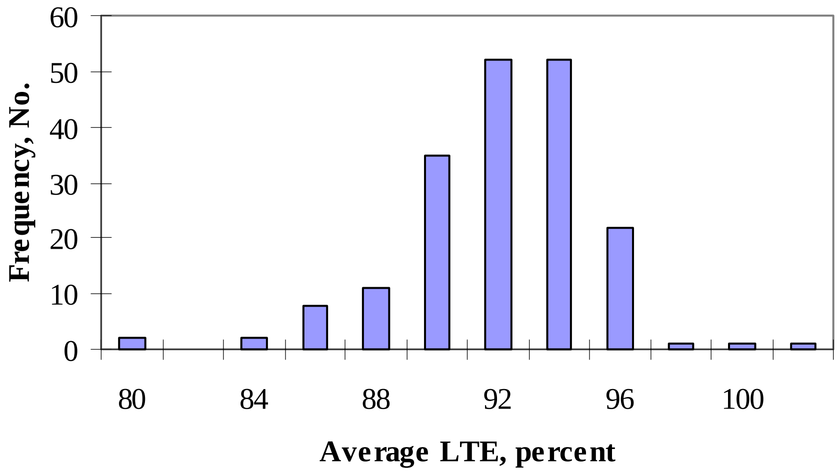


Figure B23. Distribution of Average LTE for CRC Pavements

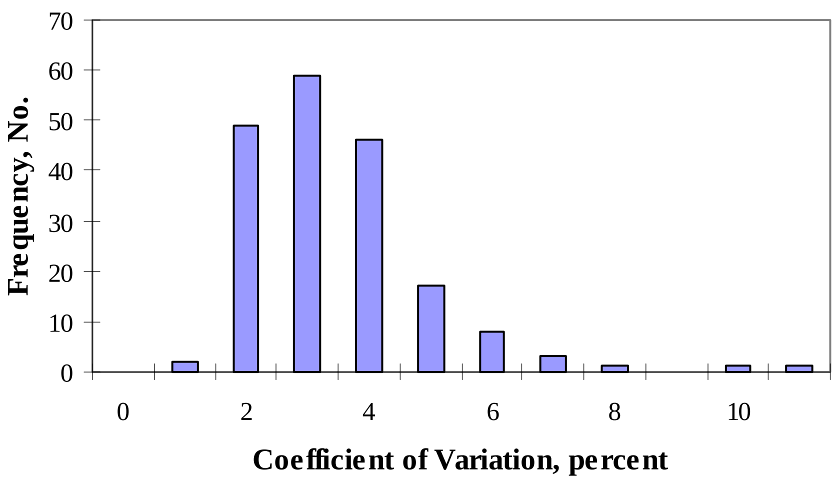


Figure B24. Distribution of CV for CRC Pavements

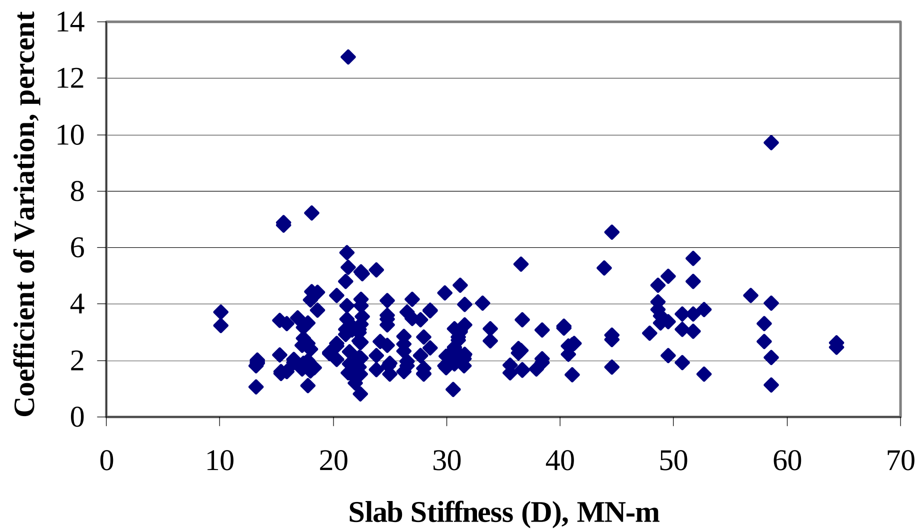


Figure B25. CV vs. Slab Stiffness for CRC Pavements

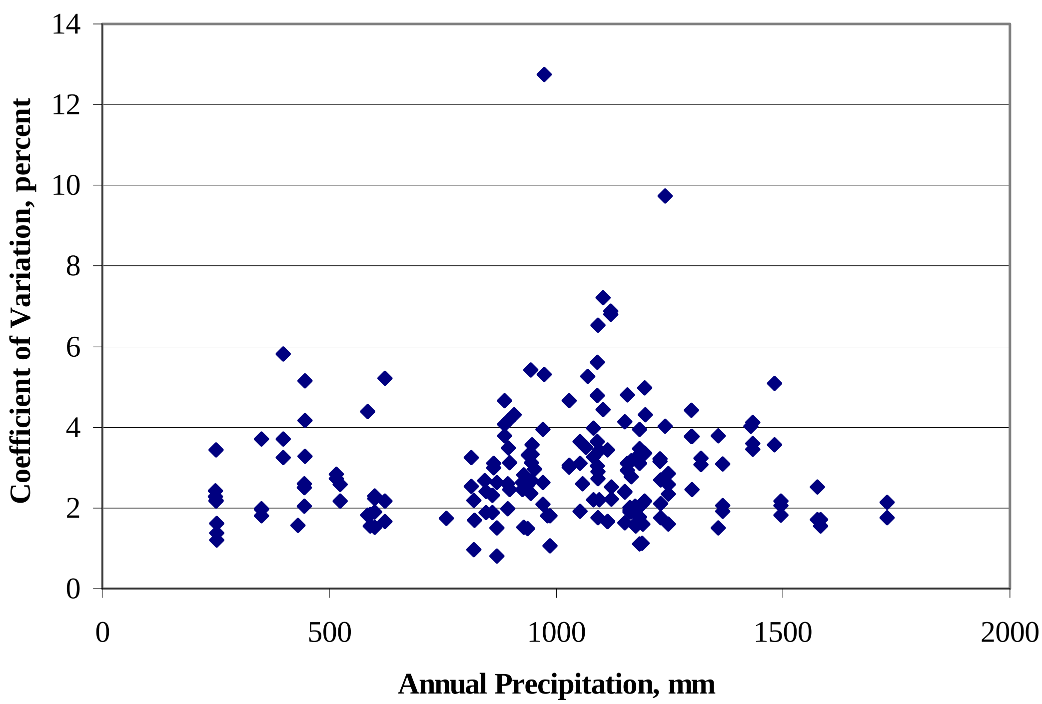


Figure B26. CV vs. Annual Precipitation for CRC Pavements

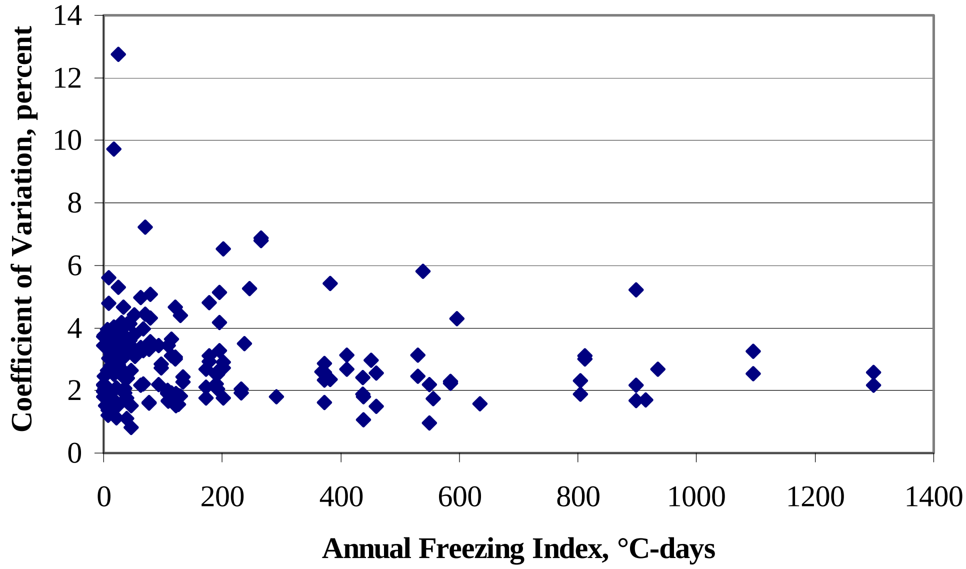


Figure B27. CV vs. Annual Freezing Index for CRC Pavements

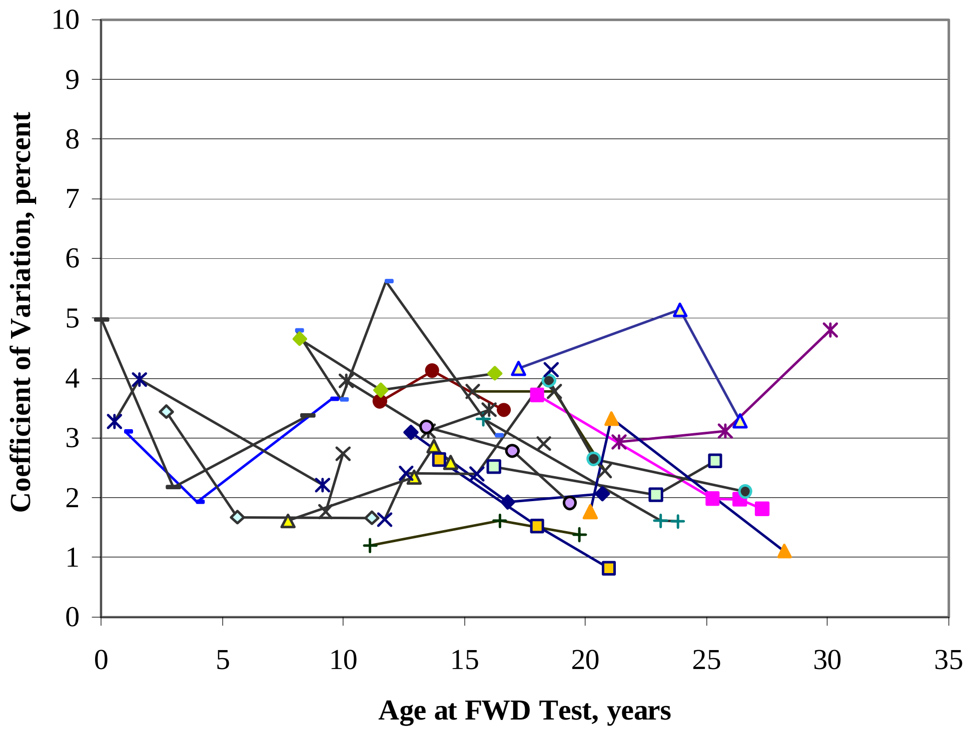


Figure B28. CV vs. Pavement Age for CRC Pavements

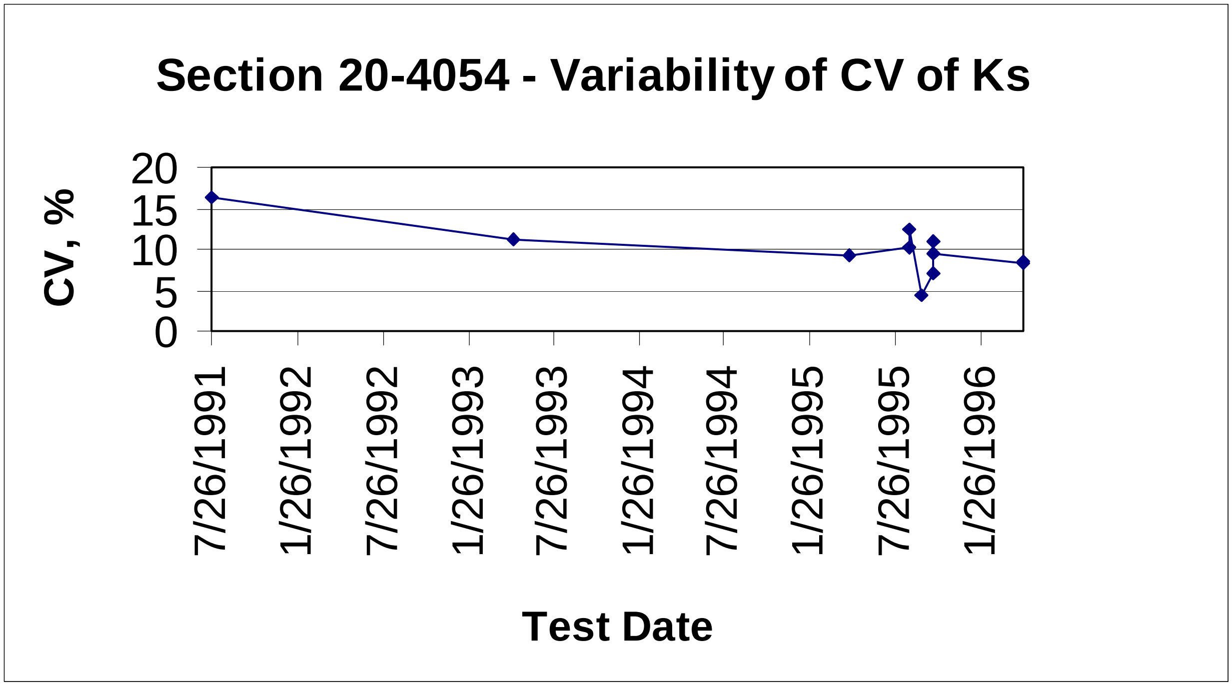


Figure B29 – Variability of CV of Moduli of Subgrade Reaction for Section 20-4054

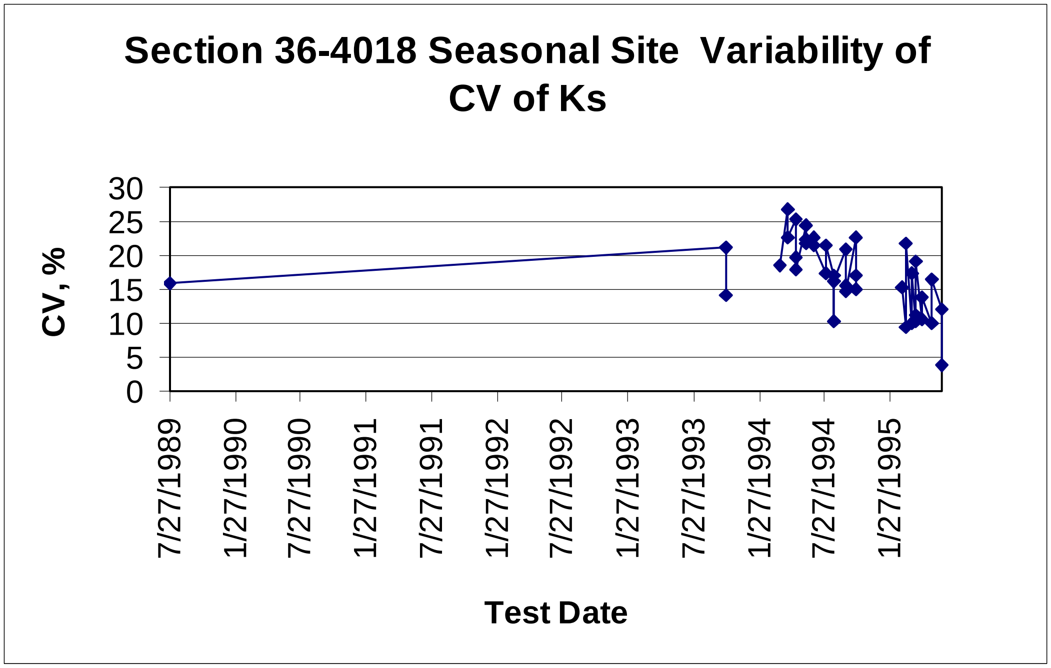


Figure B30 – Variability of CV of Moduli of Subgrade Reaction for Section 36-4018

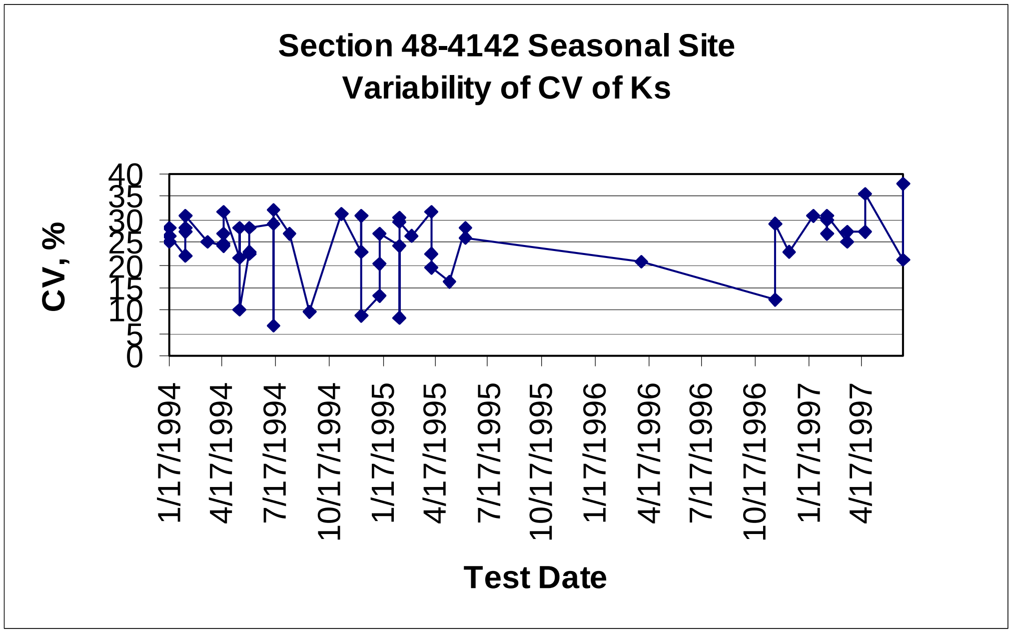


Figure B31 – Variability of CV of Moduli of Subgrade Reaction for Section 48-4142

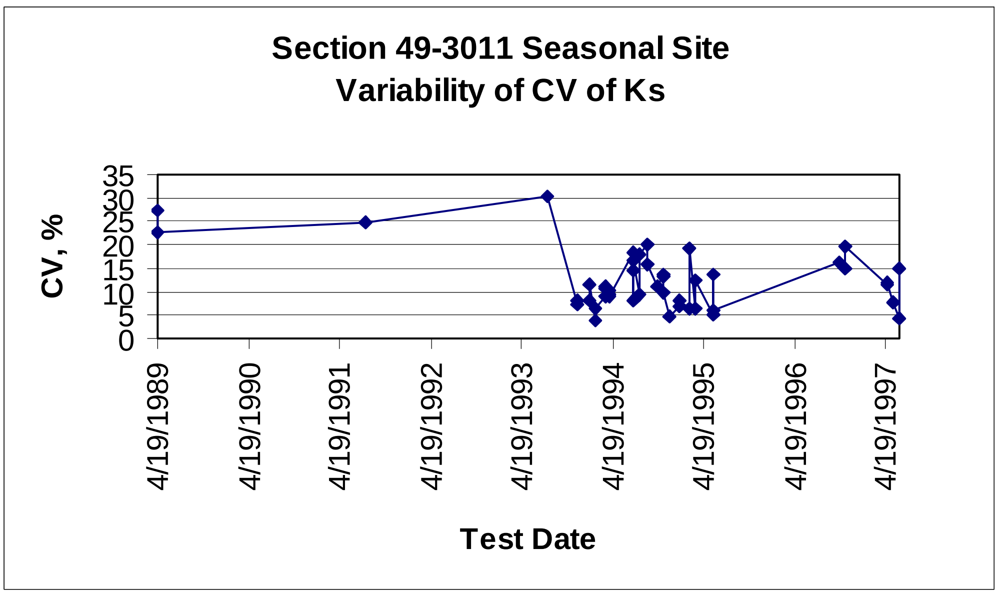


Figure B32 – Variability of CV of Moduli of Subgrade Reaction for Section 49-3011

APPENDIX C – AC-RELATED FIGURES AND DATA TABLES

Appendix C contains the following figures and tables related to asphalt concrete pavement design input parameters:

FIGURES

TABLES

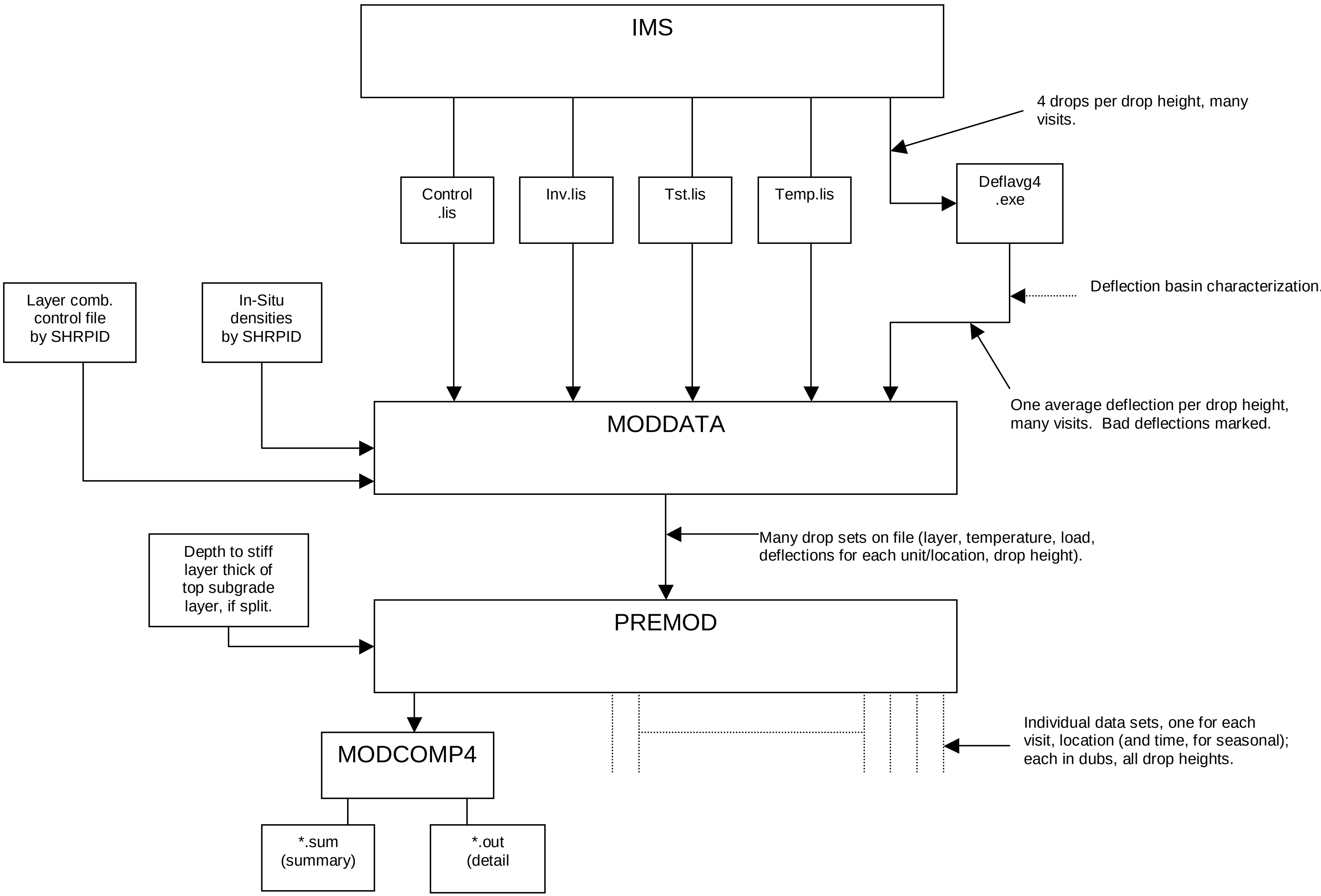


Figure C1. Flowchart Depicting the Backcalculation Process Utilized in Determining Moduli for the LTPP GPS, SPS and SMP Pavement Sections.

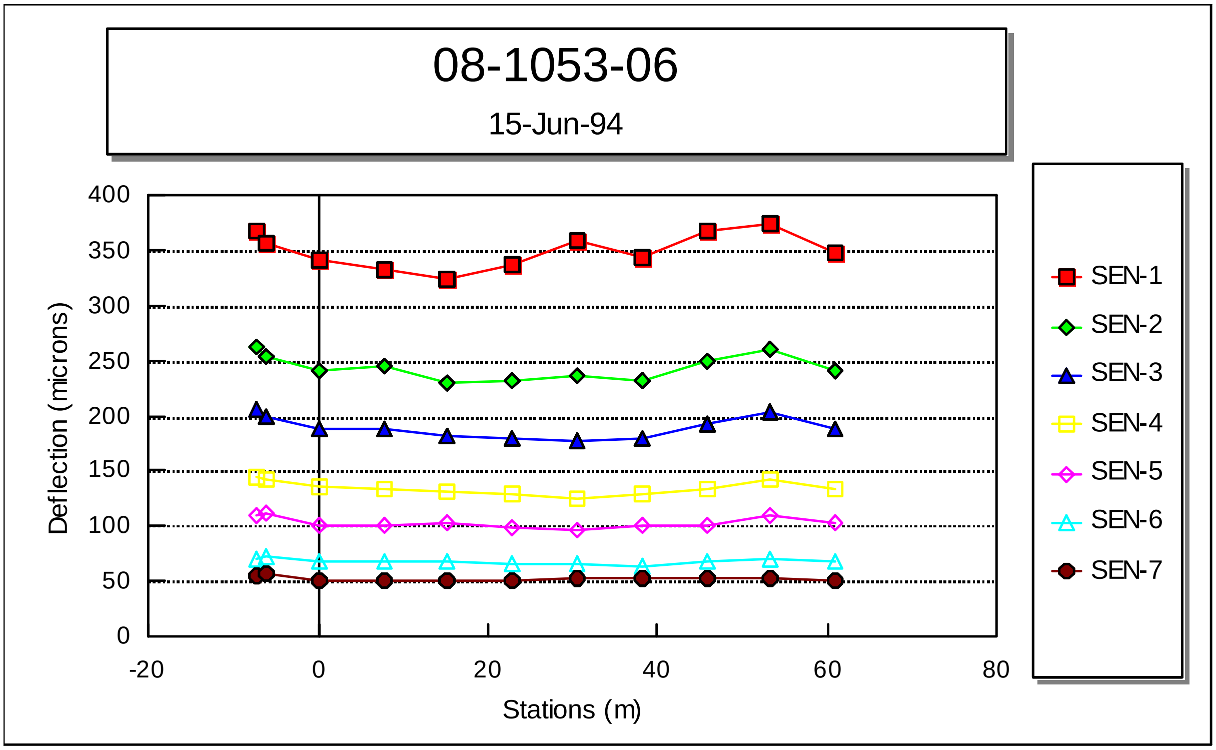


Figure C2. Example of a Pavement Section With Consistent Deflection Measurements.

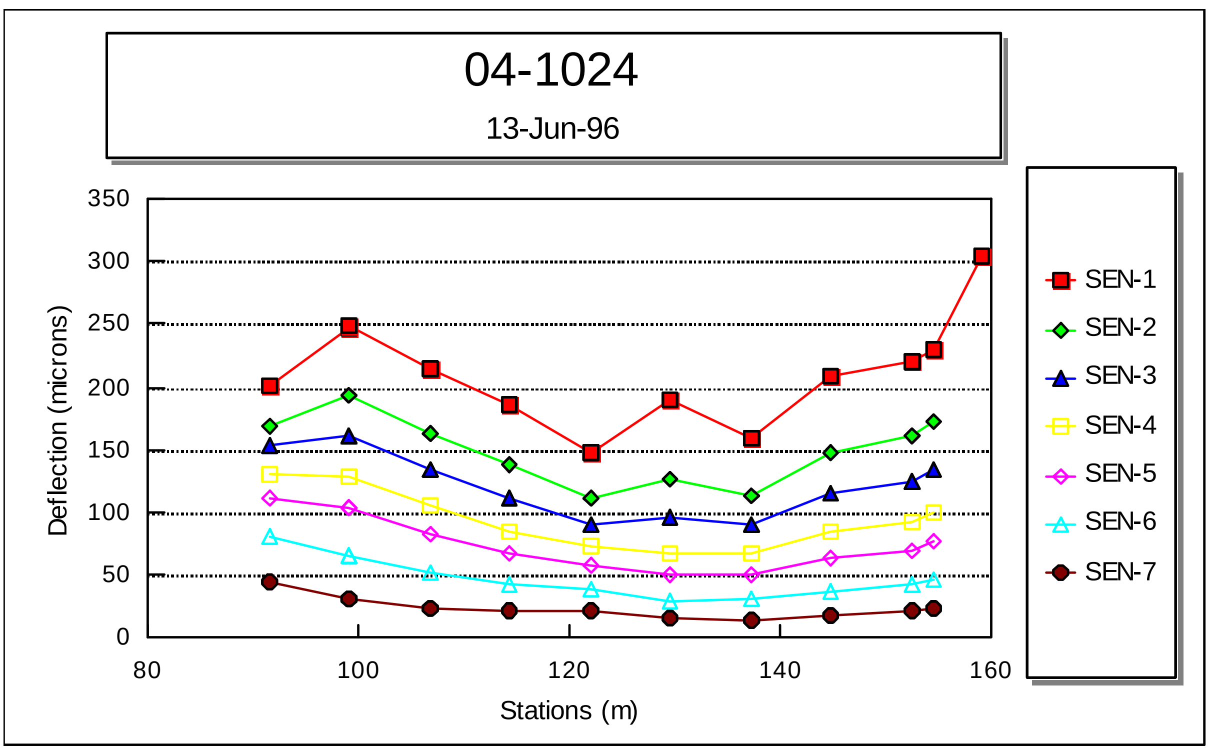


Figure C3. Example of a Pavement Section With Variable Deflection Measurements.

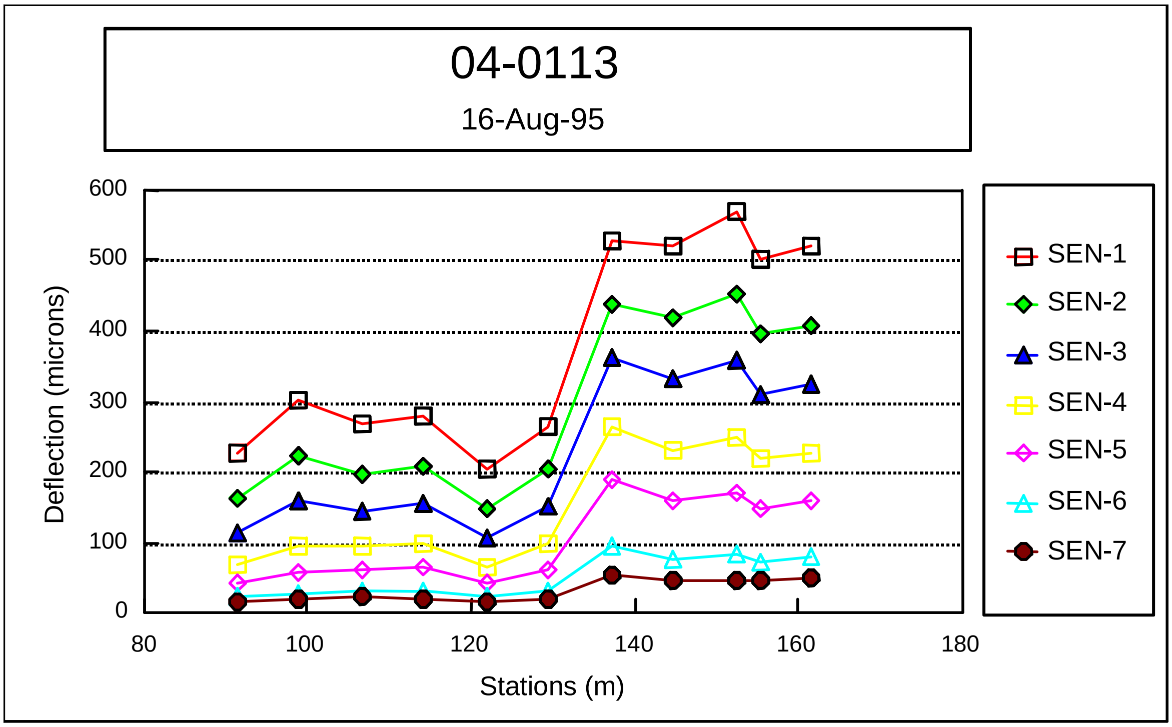


Figure C4. Example of a Pavement Section With Significantly Different Deflection Measurements Between the First Half and Second Half of the Section.

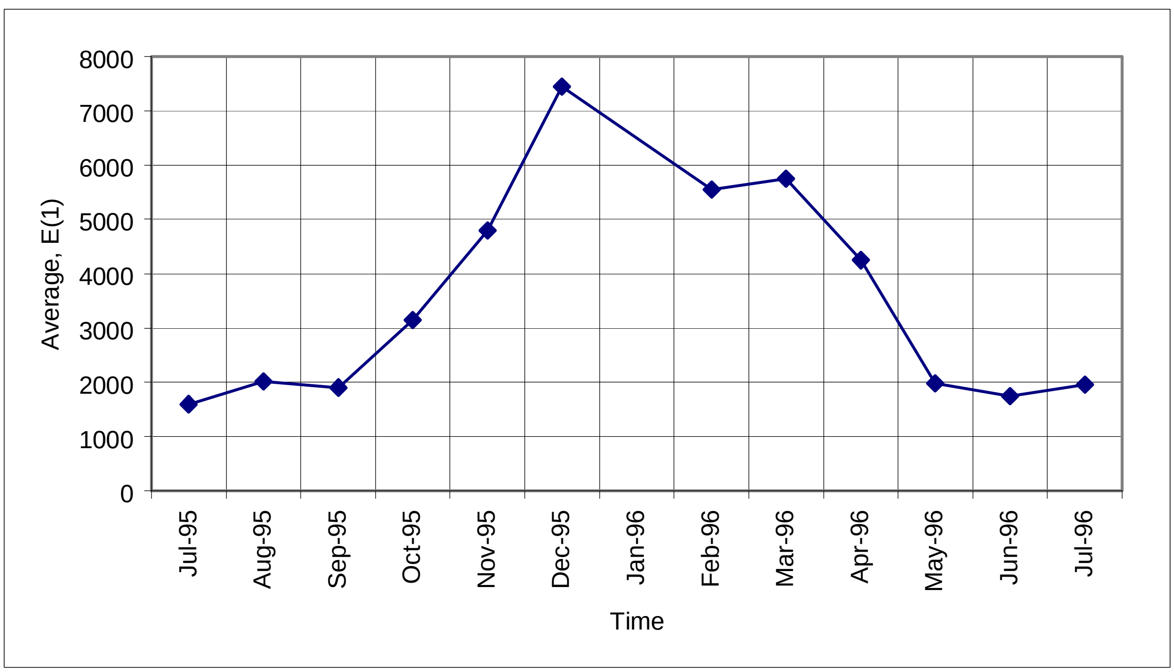
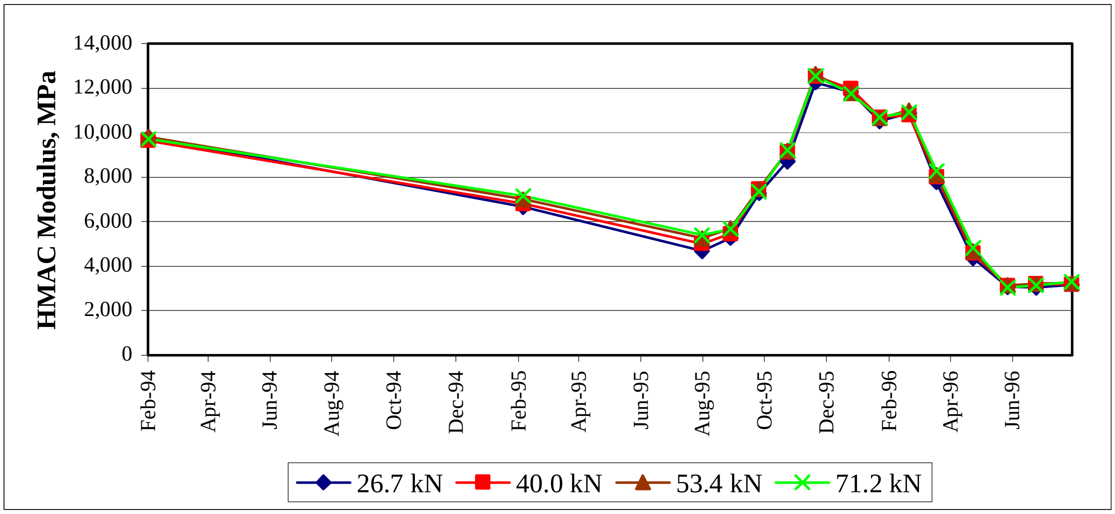
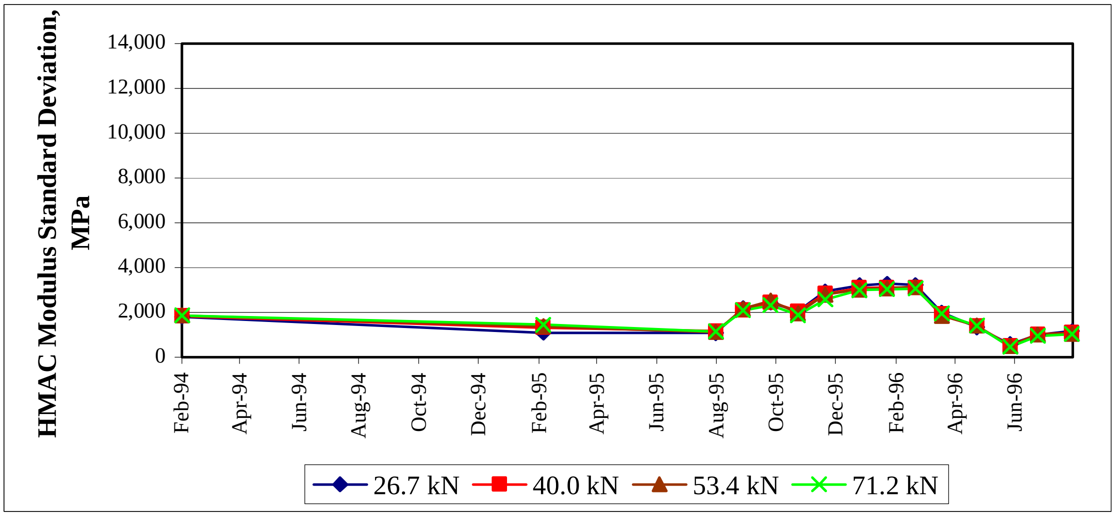


Figure C5. Example of the Change in AC Modulus Over Time During the Year.





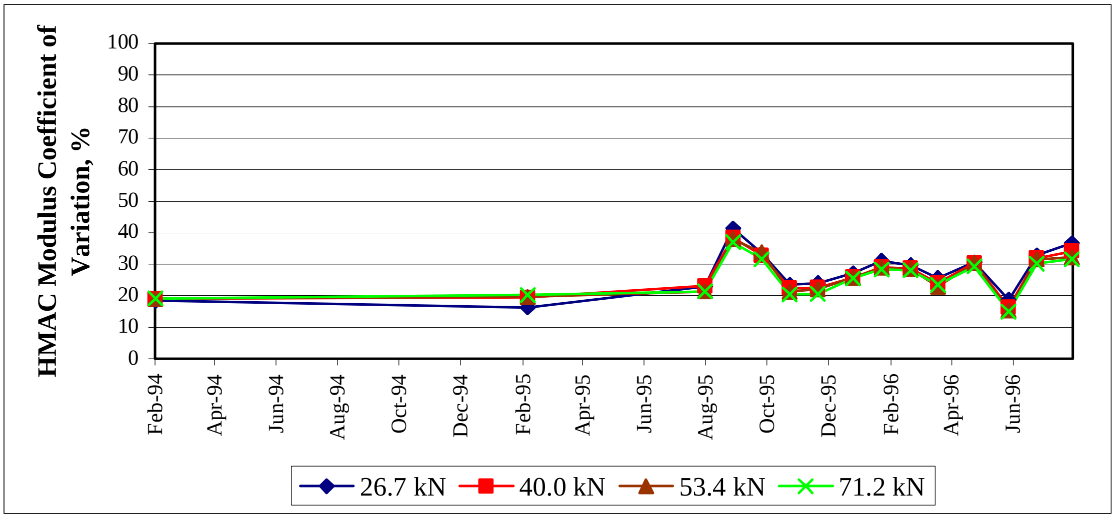
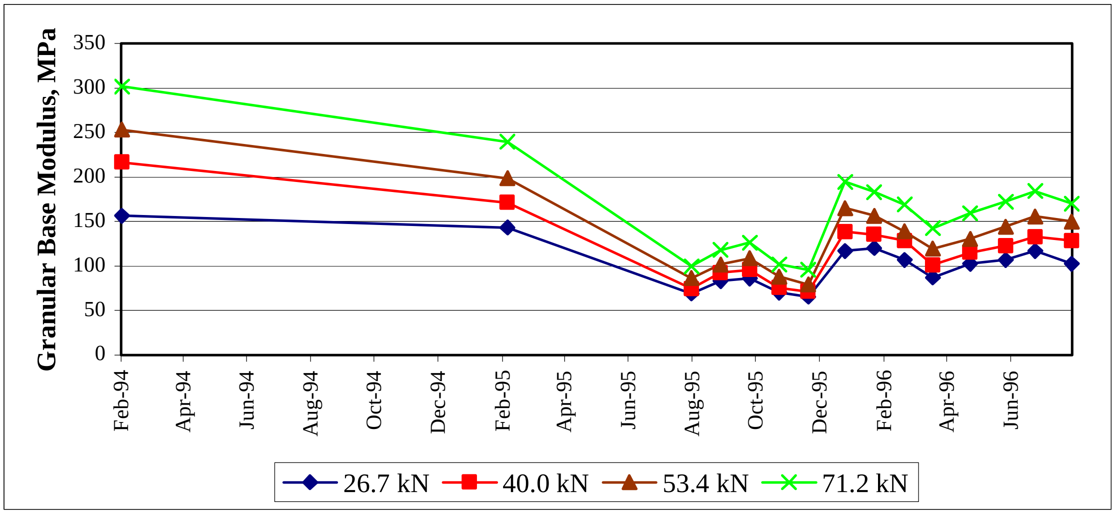
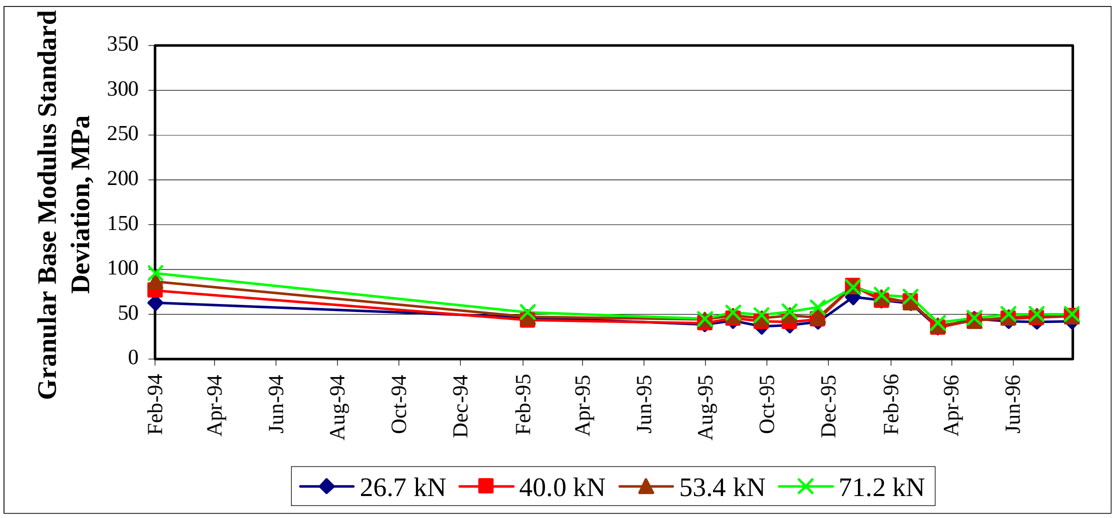


Figure C6. Seasonal Monitoring Site 04-0113 (SPS-1) - HMAC Layer Backcalculated Statistics.





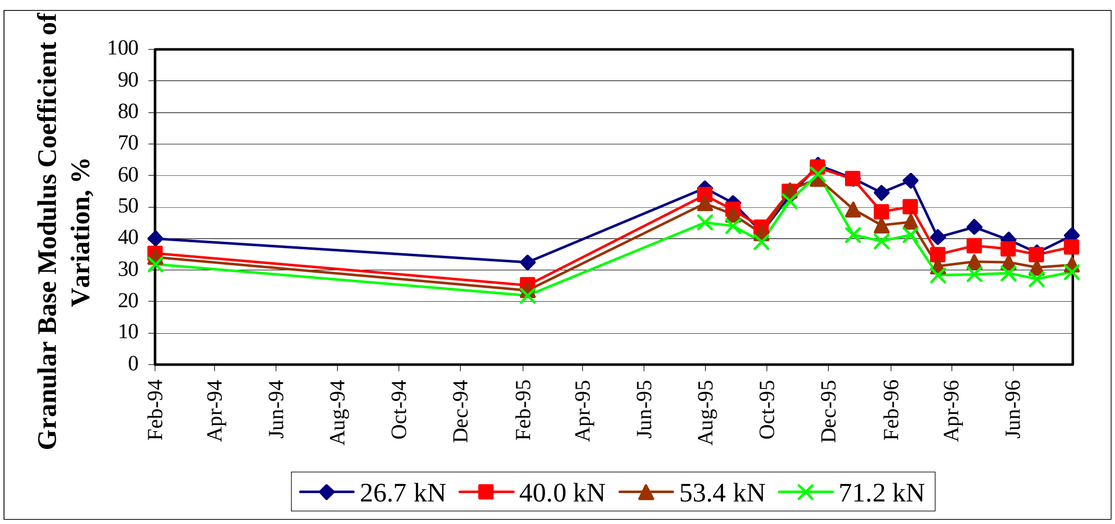


Figure C7. Seasonal Monitoring Site 04-0113 (SPS-1) - Granular Base Layer Backcalculated Statistics.

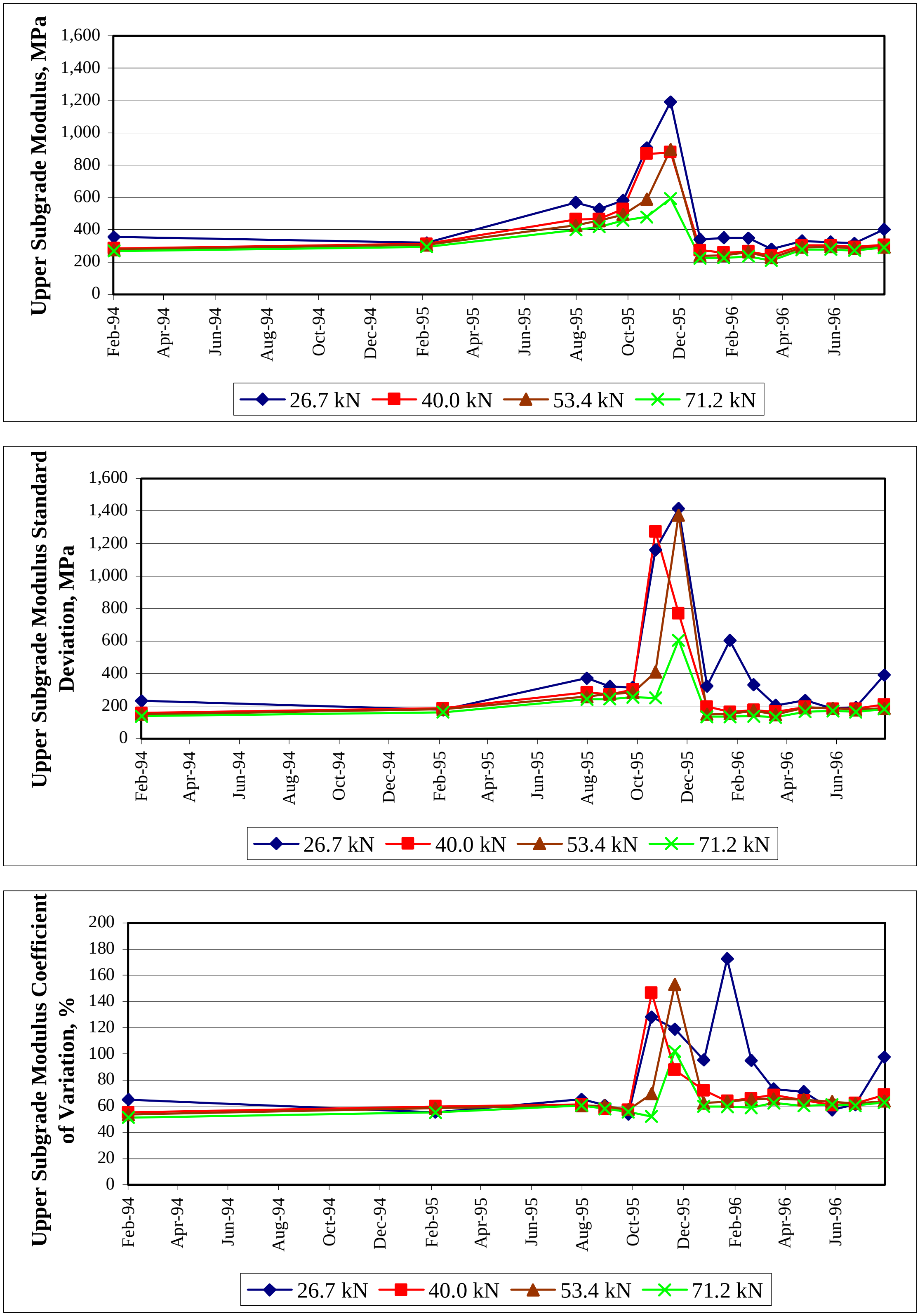
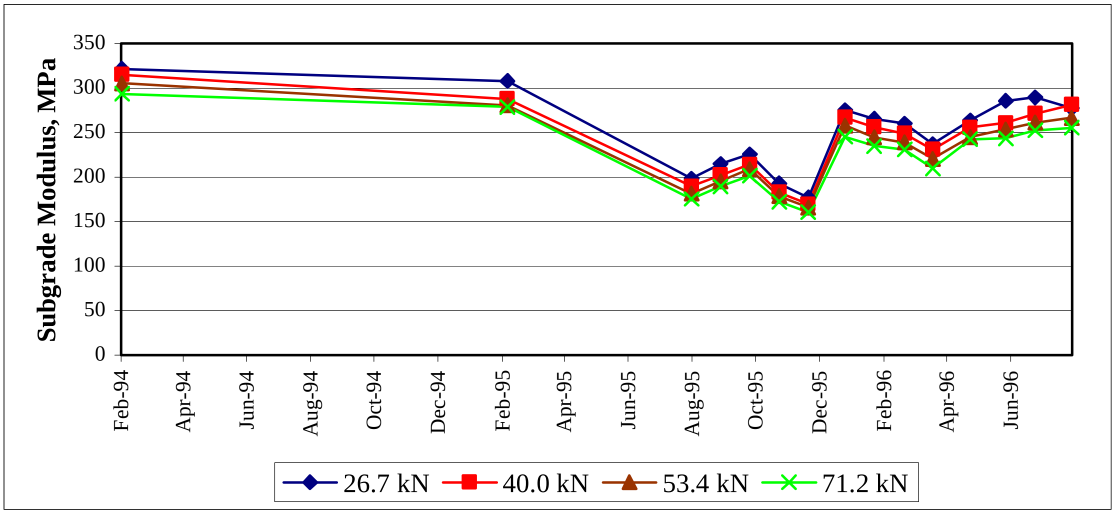
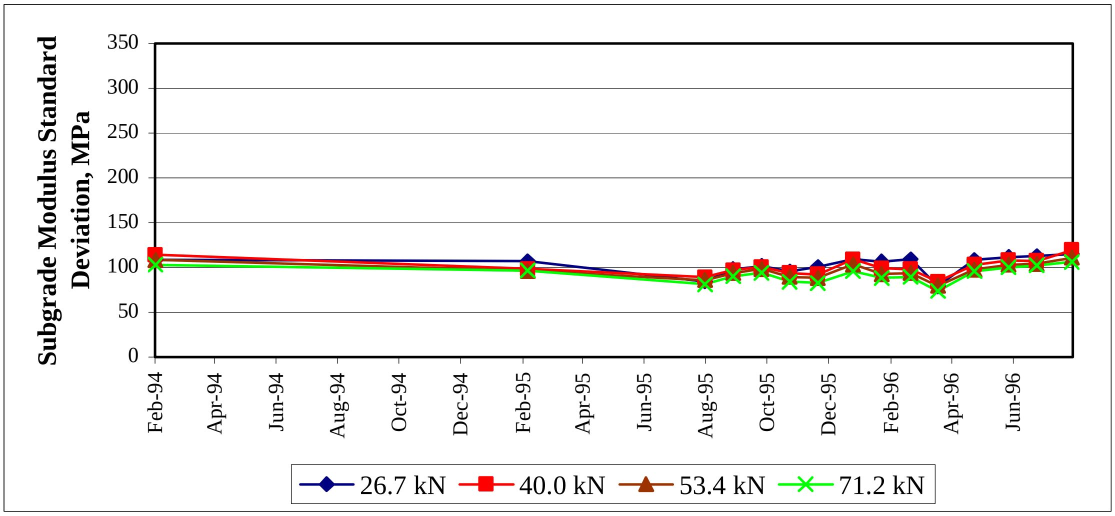


Figure C8. Seasonal Monitoring Site 04-0113 (SPS-1) - Upper Subgrade Backcalculated Statistics.





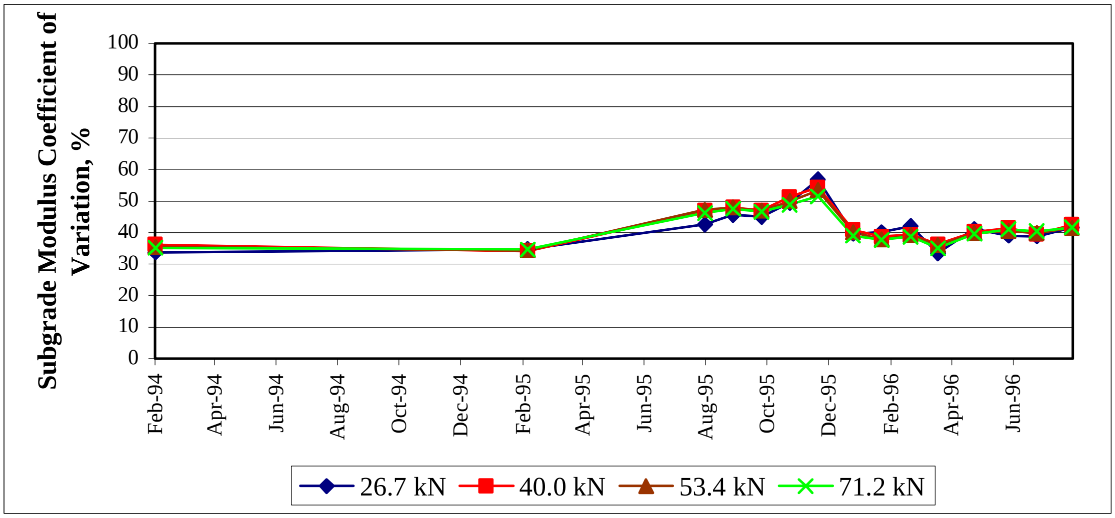


Figure C9. Seasonal Monitoring Site 04-0113 (SPS-1) - Subgrade Layer Backcalculated Statistics.

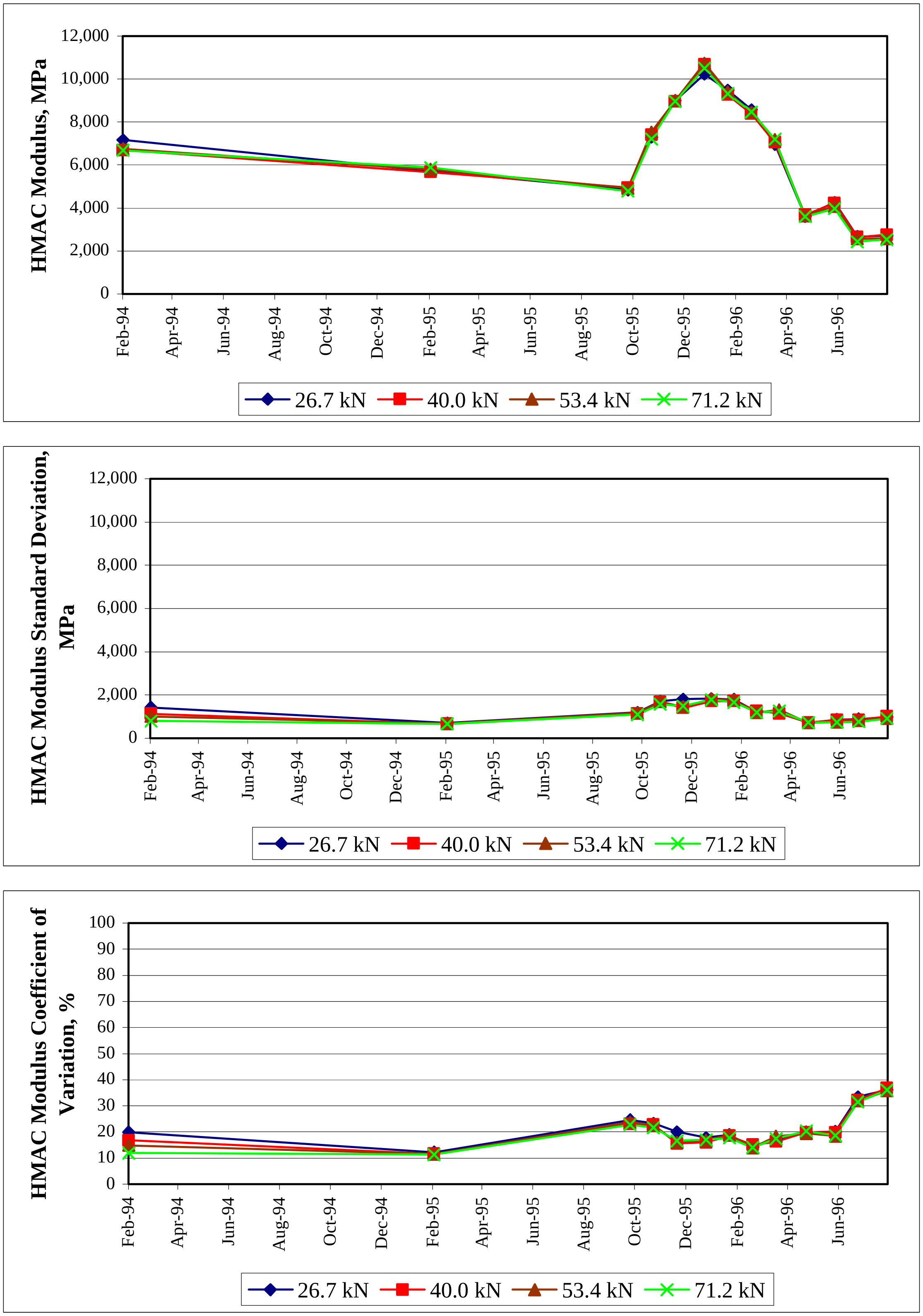
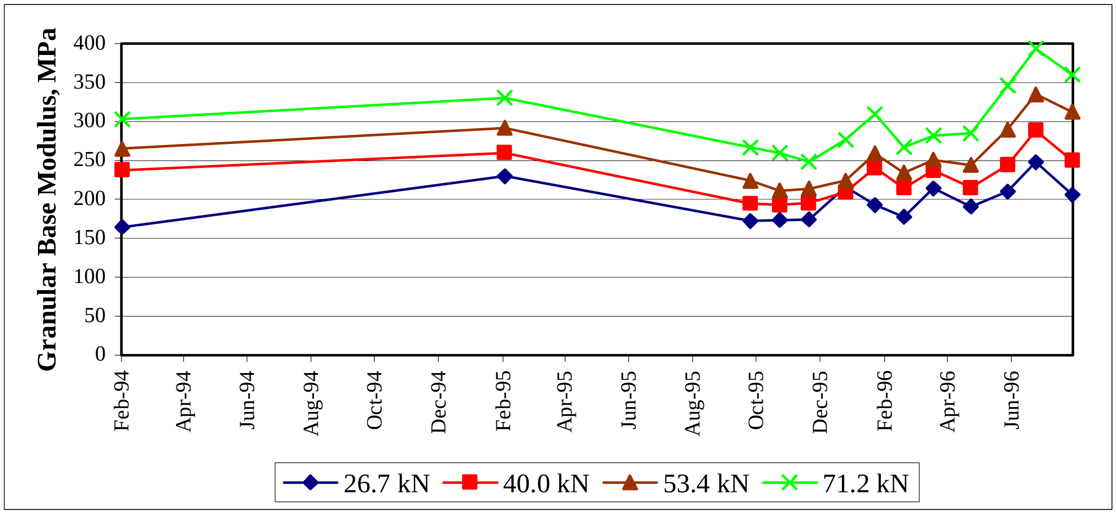
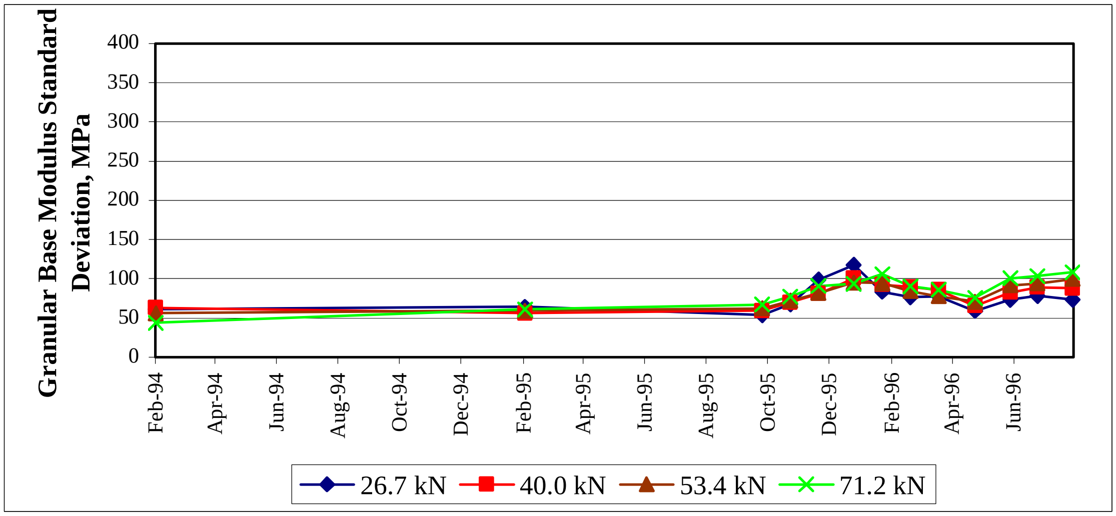


Figure C10. Seasonal Monitoring Site 04-0114 (SPS-1) - HMAC Layer Backcalculated Statistics.





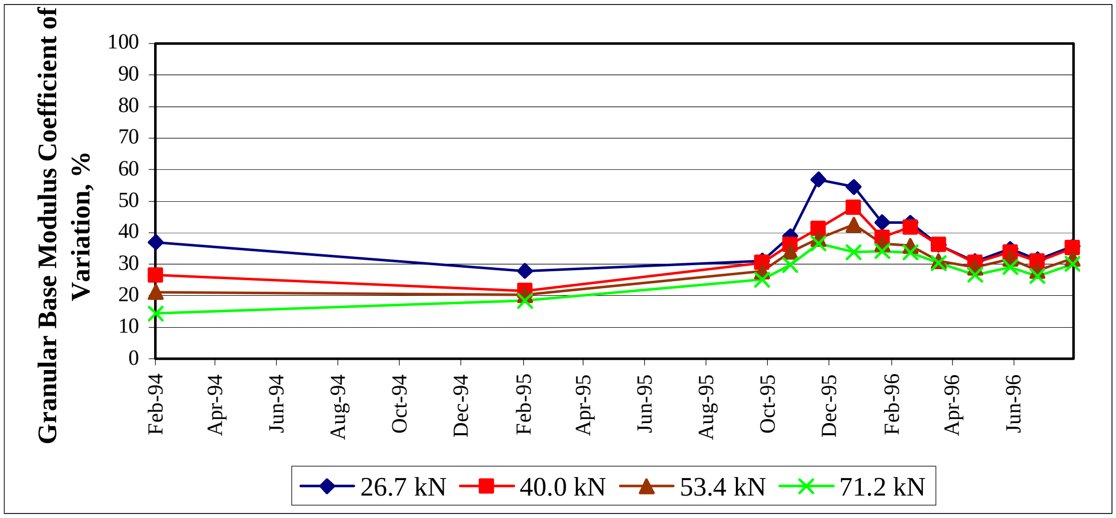
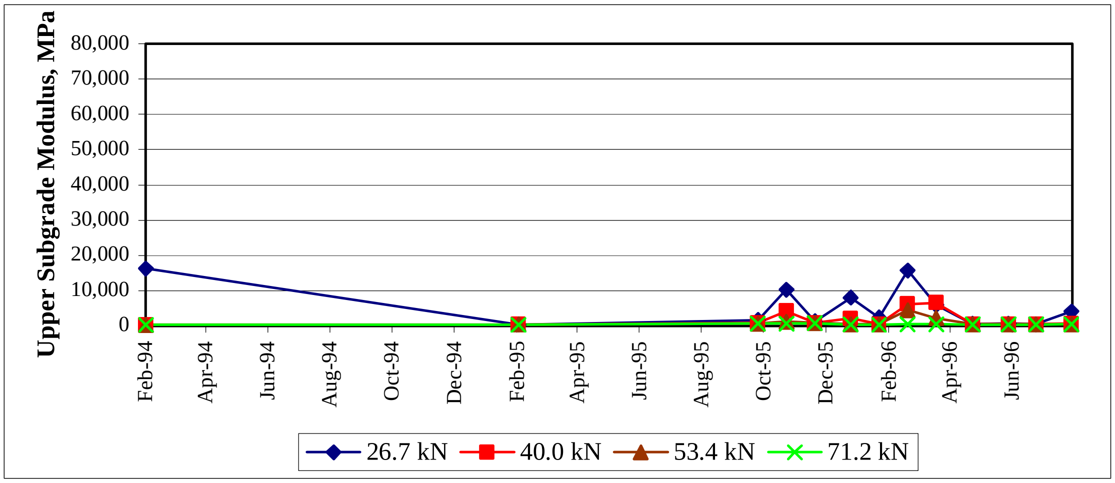
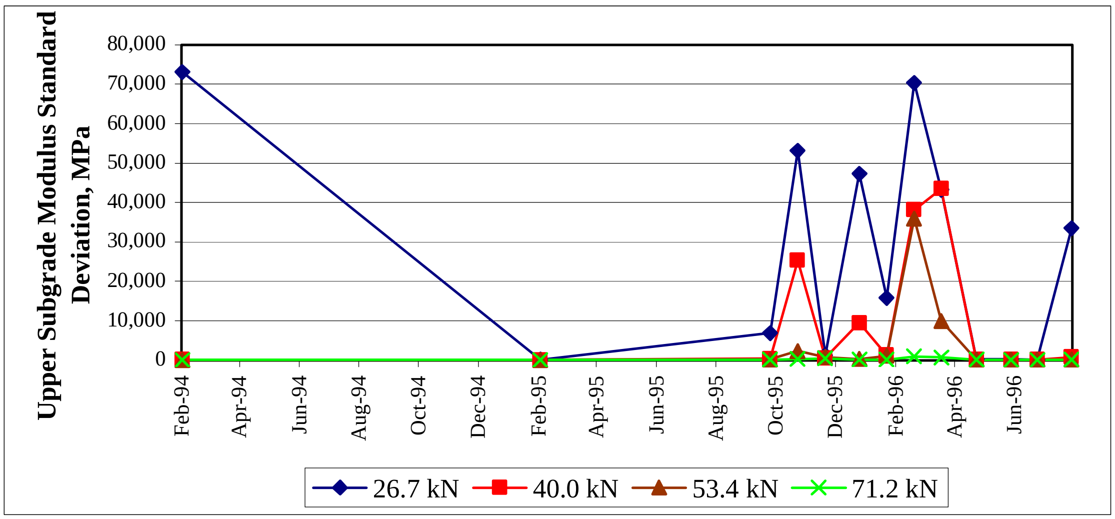


Figure C11. Seasonal Monitoring Site 04-0114 (SPS-1) - Granular Base Layer Backcalculated Statistics.





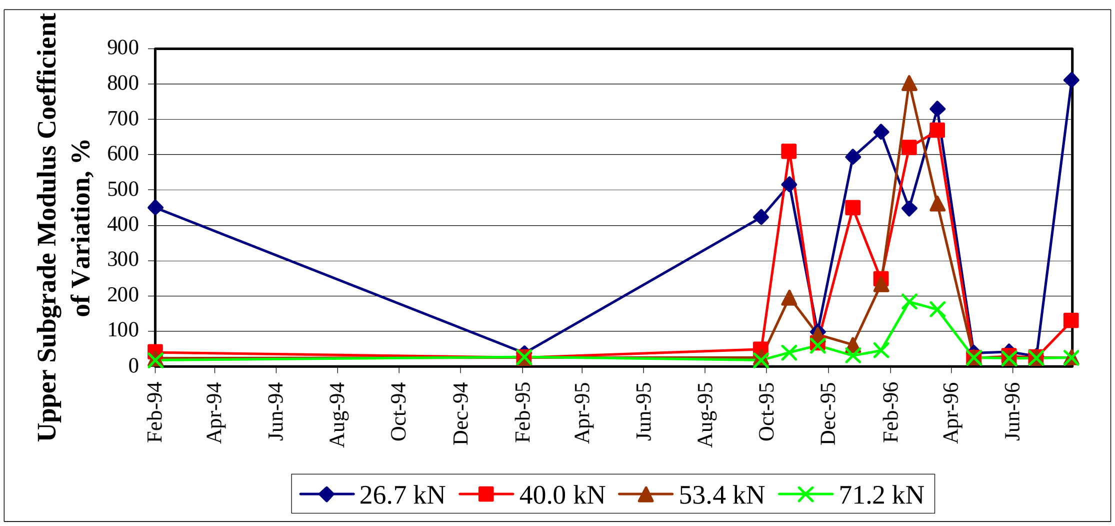
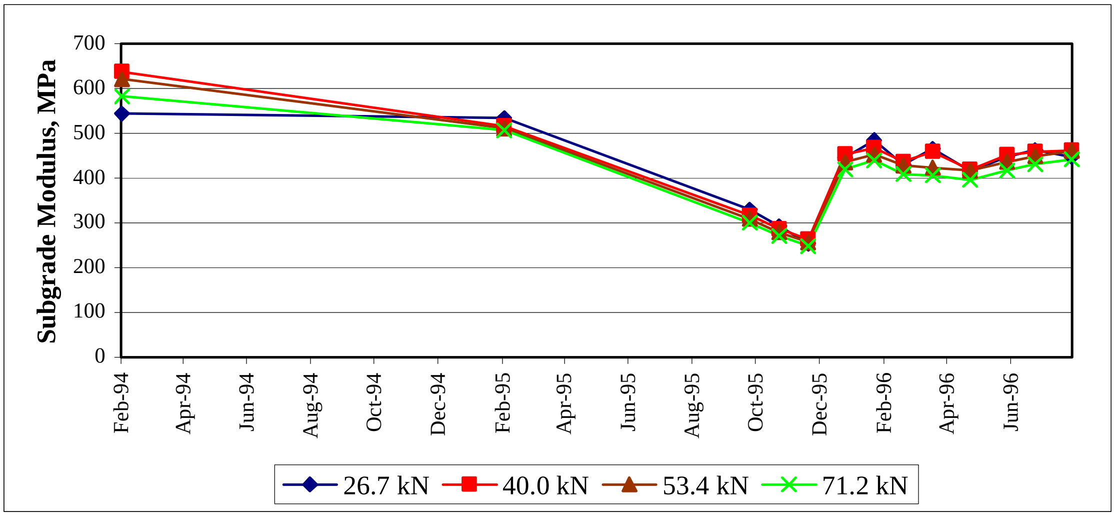
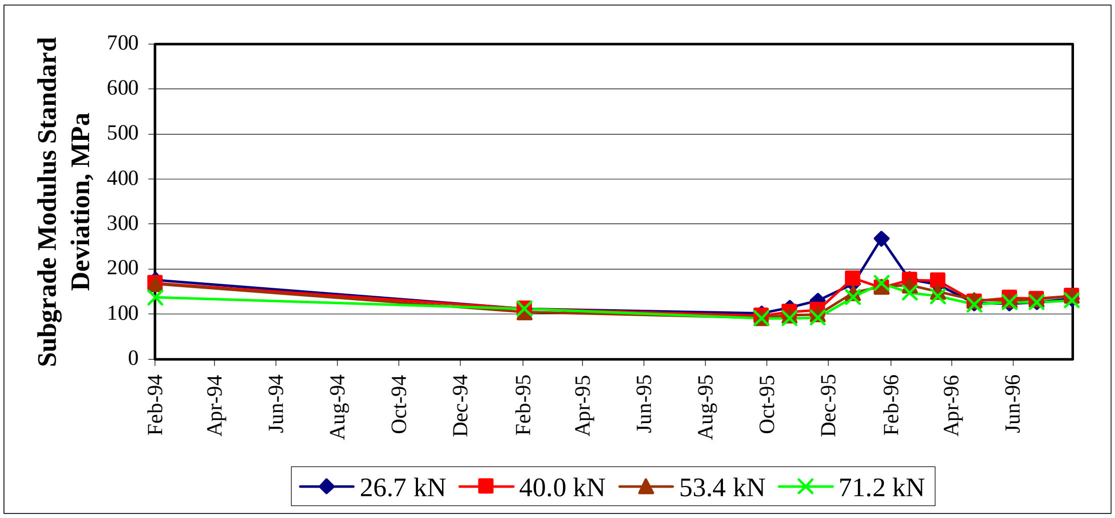


Figure C12. Seasonal Monitoring Site 04-0114 (SPS-1) - Upper Subgrade Backcalculated Statistics.





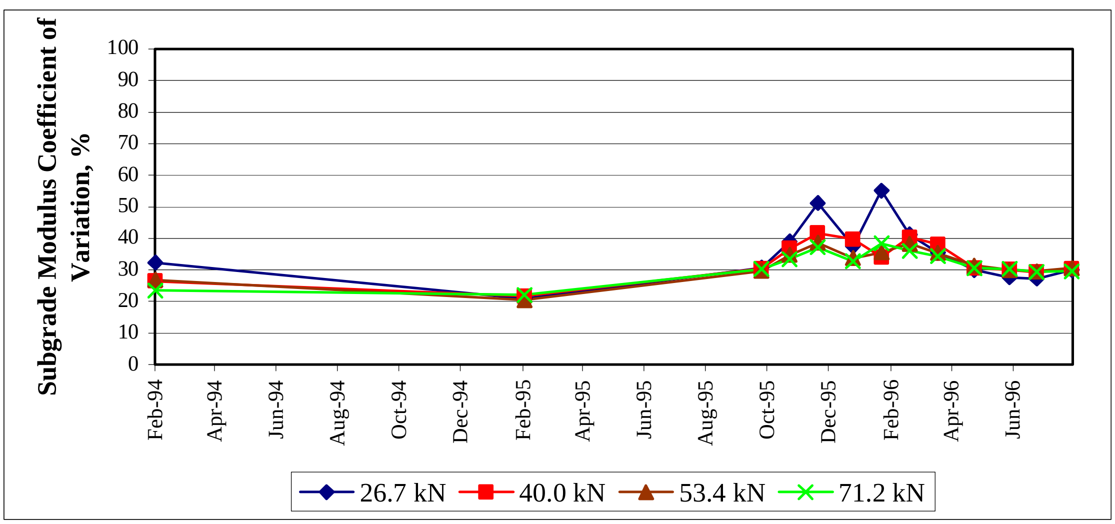
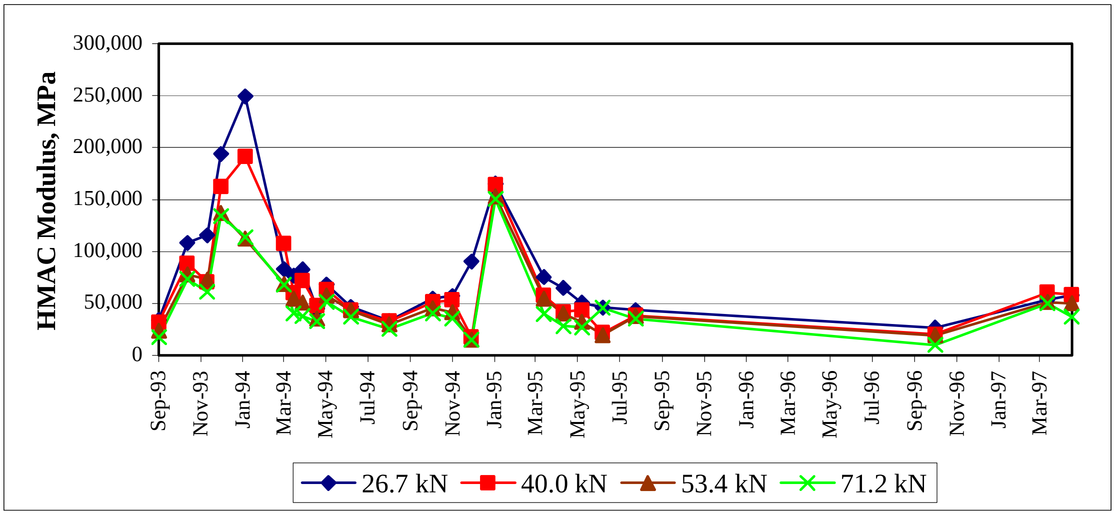
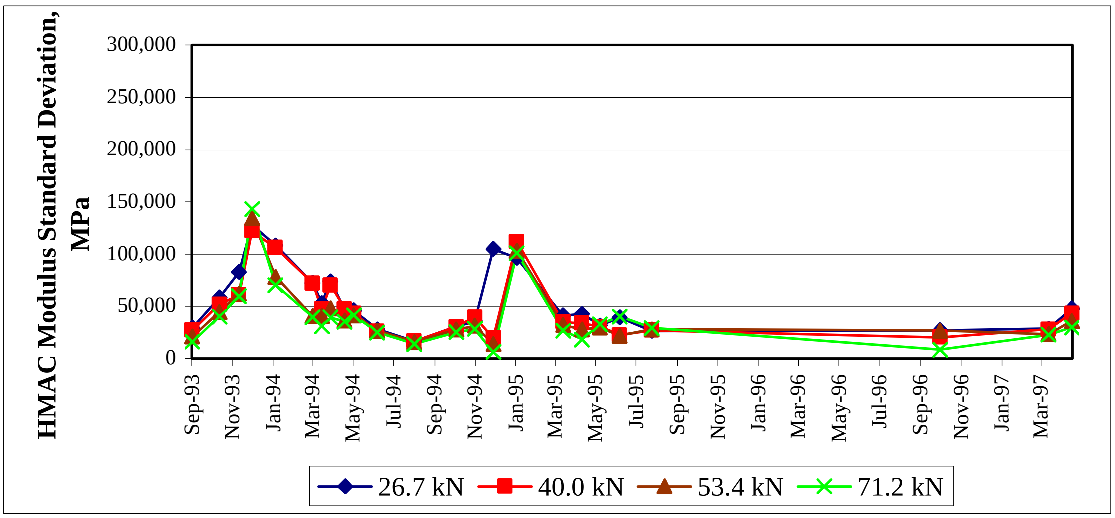


Figure C13. Seasonal Monitoring Site 04-0114 (SPS-1) - Subgrade Layer Backcalculated Statistics.





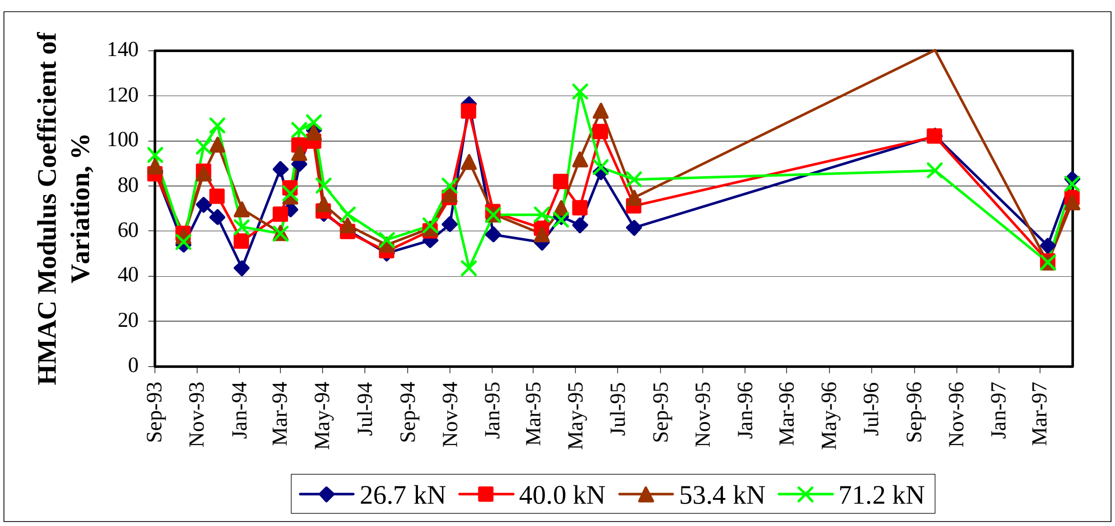
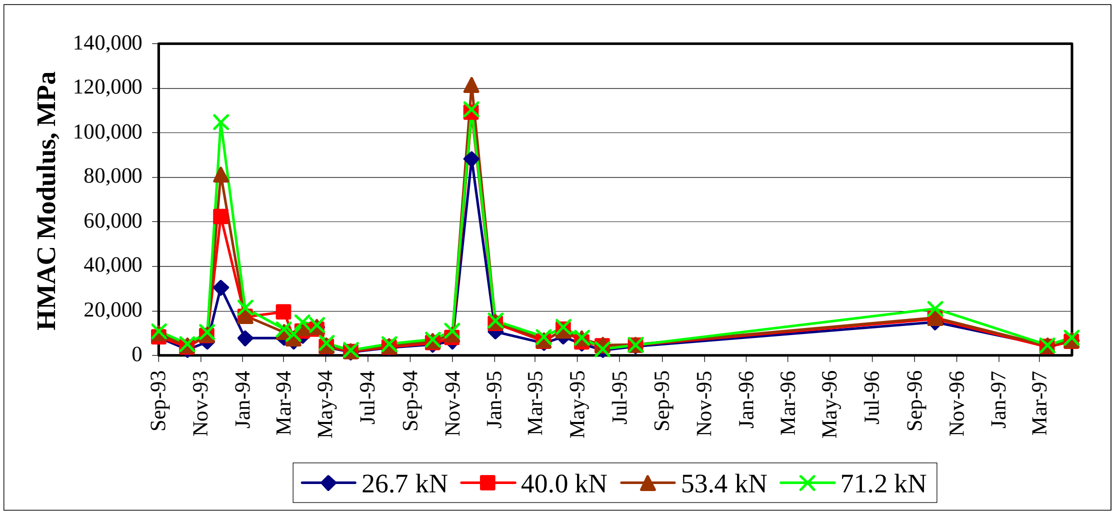
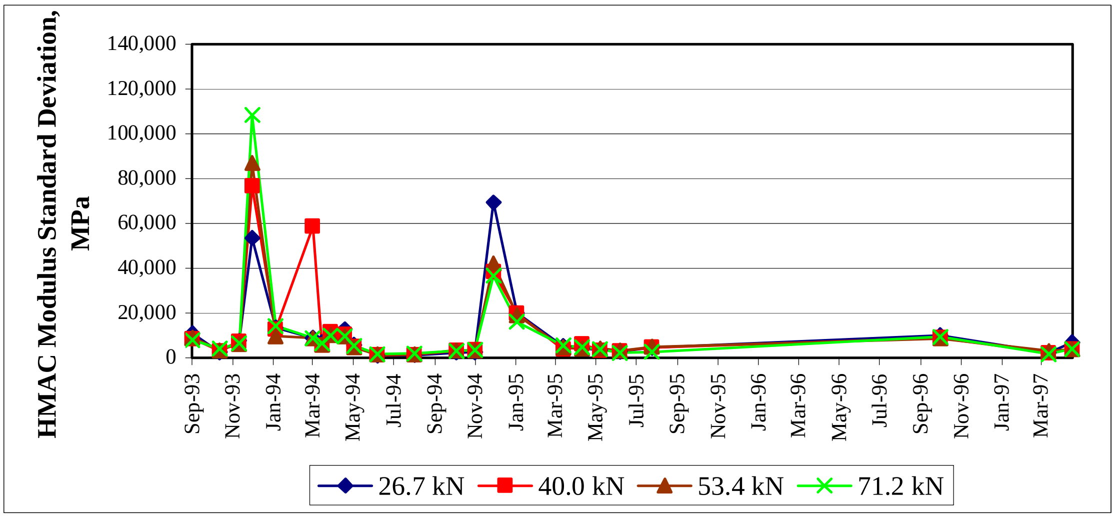


Figure C14. Seasonal Monitoring Site 27-1028 (GPS-1) - HMAC Layer Backcalculated Statistics.





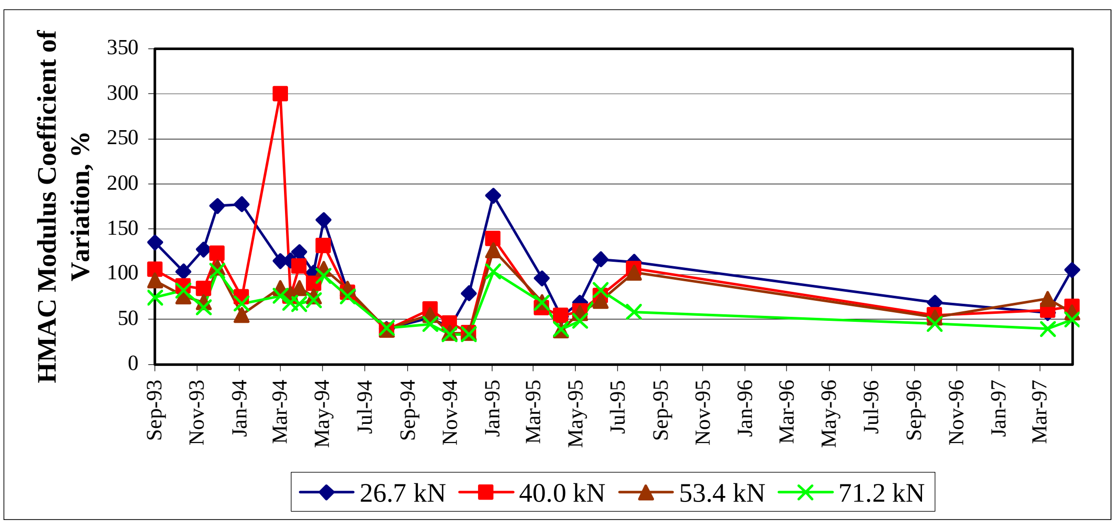
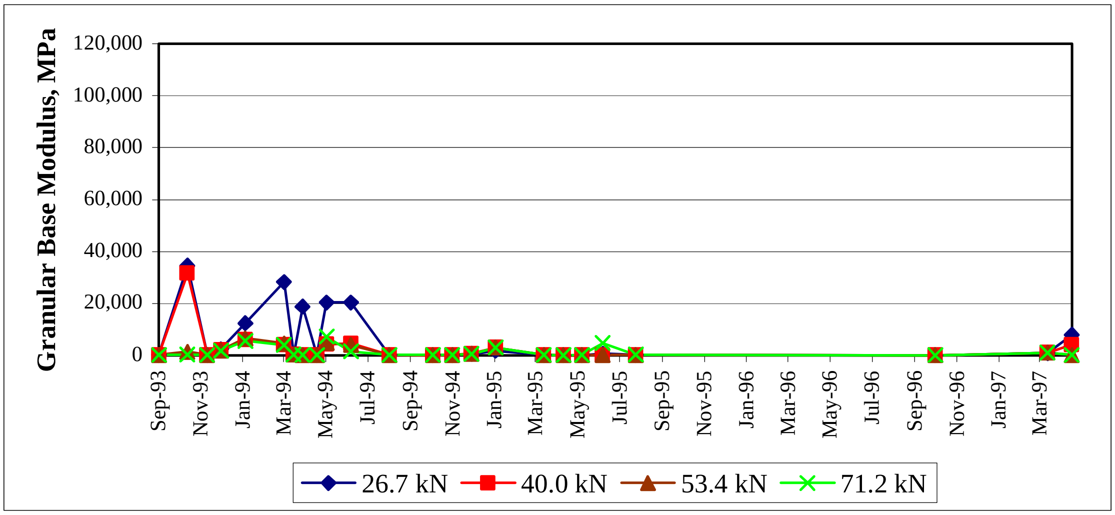
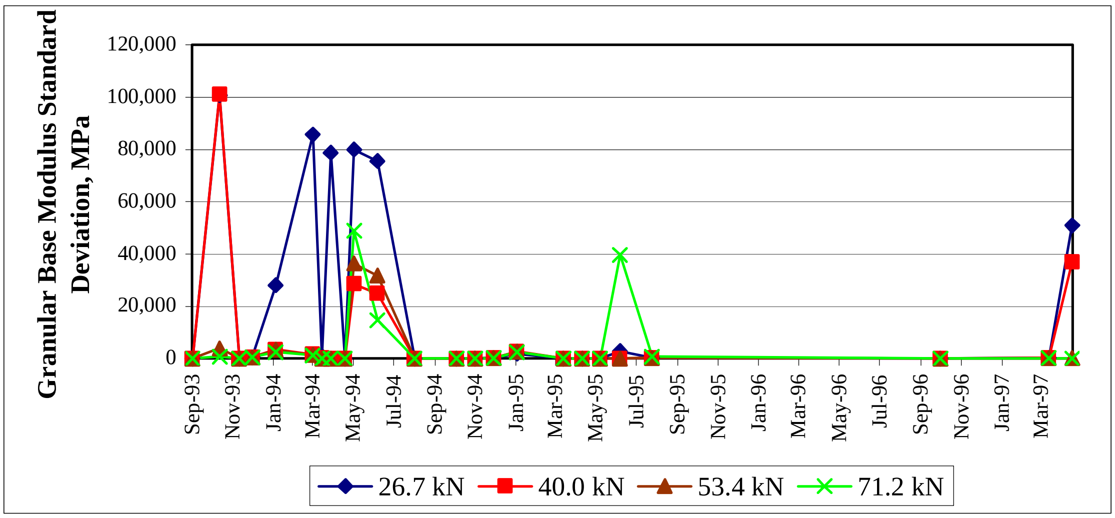


Figure C15. Seasonal Monitoring Site 27-1028 (GPS-1) - HMAC Base Layer Backcalculated Statistics.





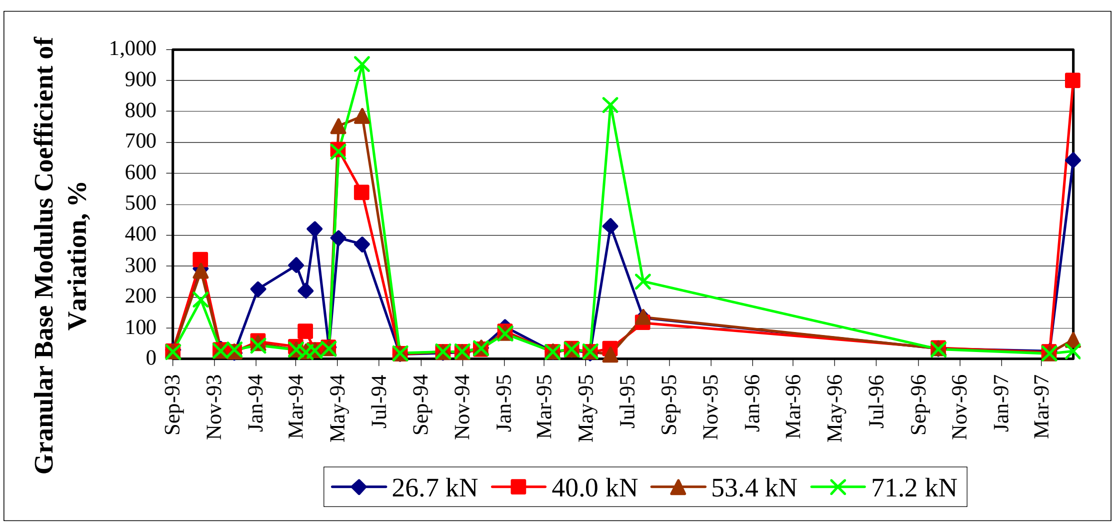
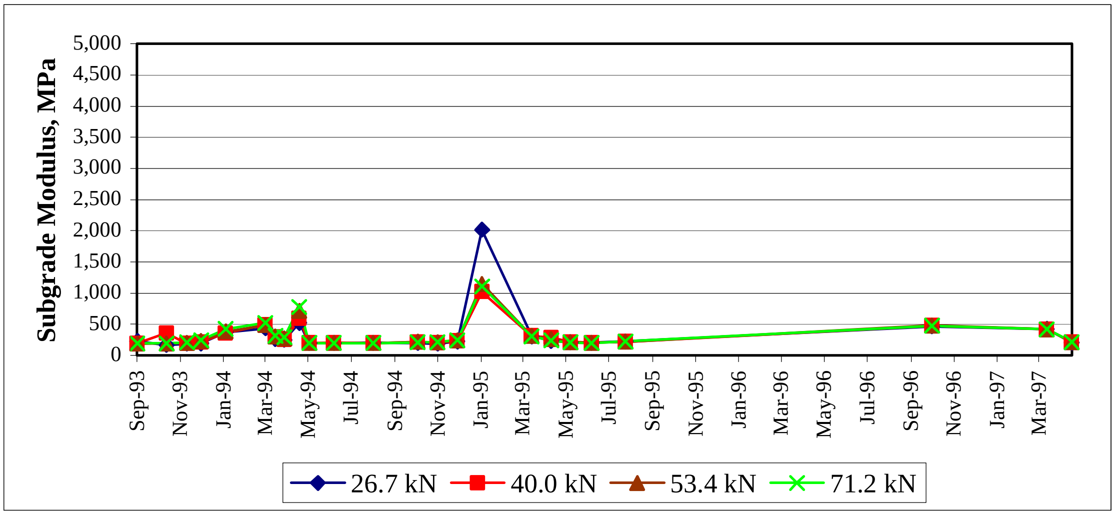
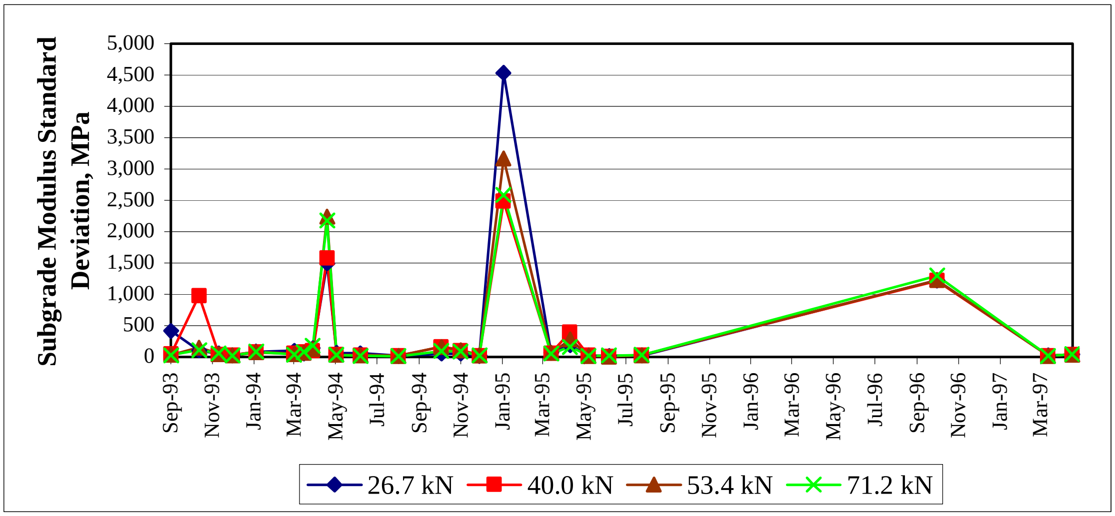


Figure C16. Seasonal Monitoring Site 27-1028 (GPS-1) – Granular Base Layer Backcalculated Statistics.





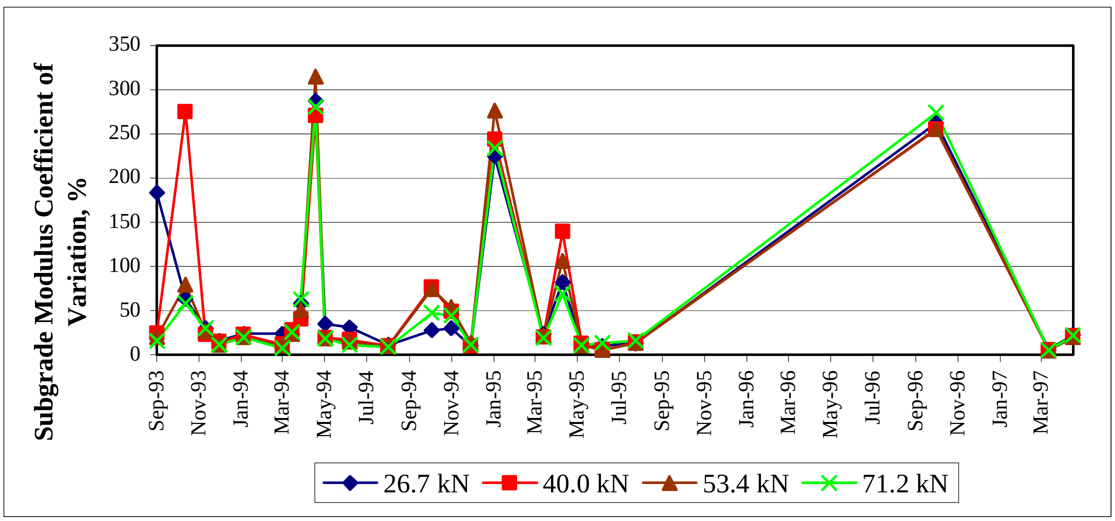


Figure C17. Seasonal Monitoring Site 27-1028 (GPS-1) - Subgrade Layer Backcalculated Statistics.

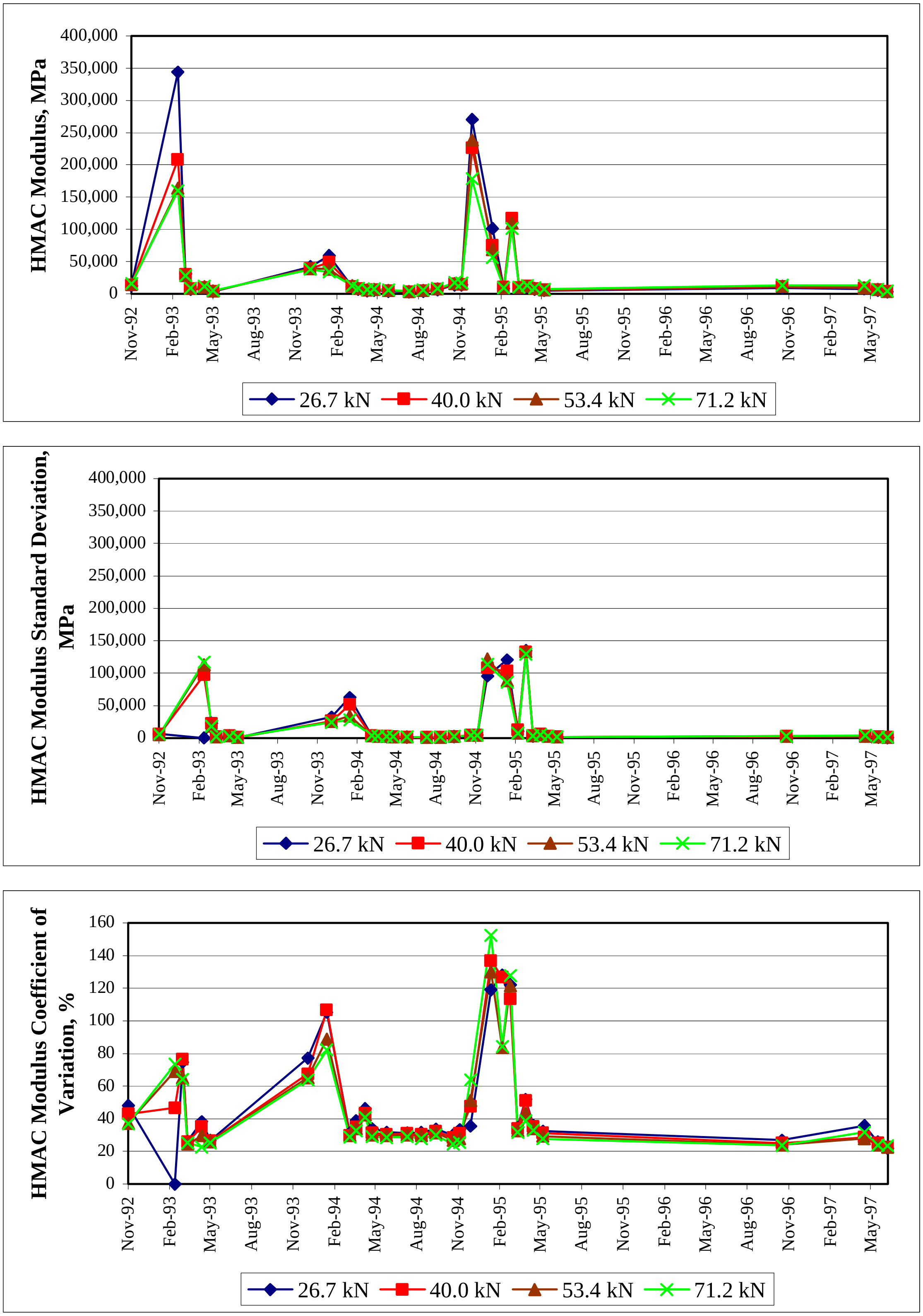
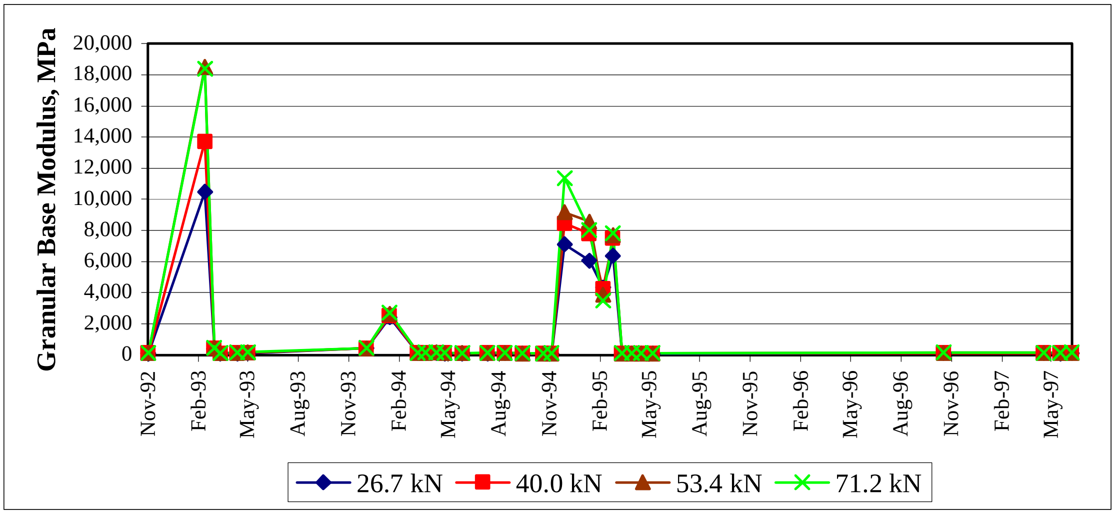
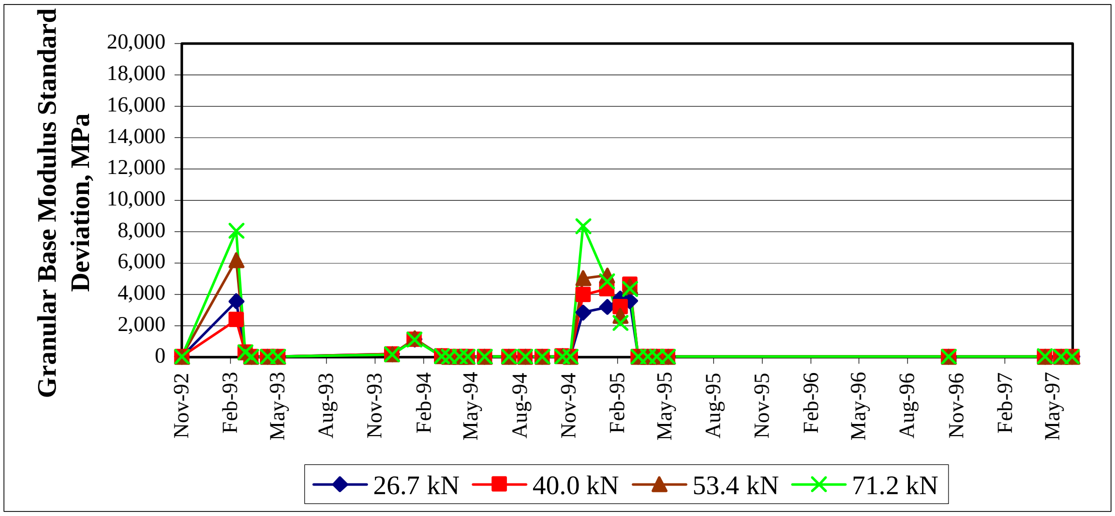


Figure C18. Seasonal Monitoring Site 30-8129 (GPS-1) - HMAC Layer Backcalculated Statistics.





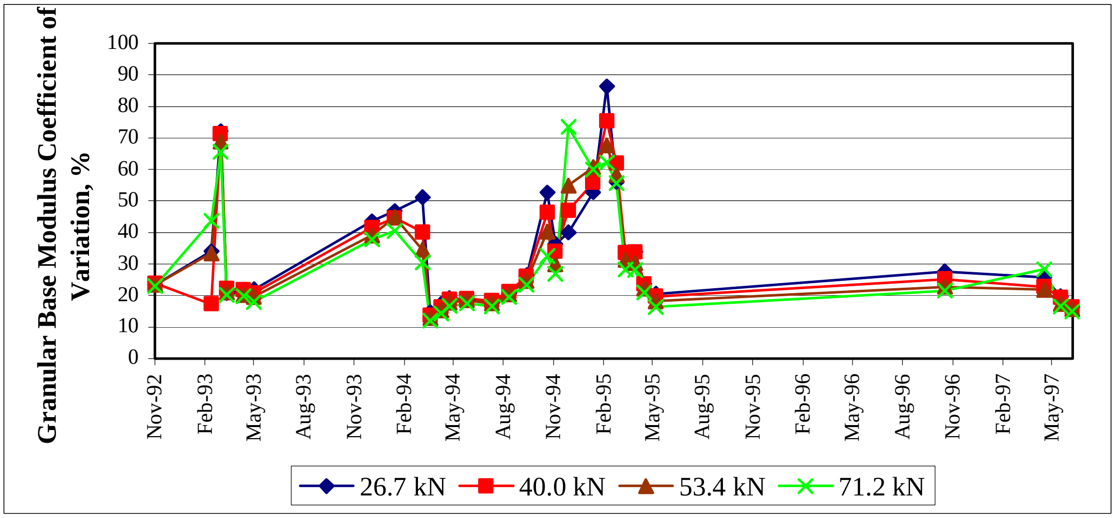


Figure C19. Seasonal Monitoring Site 30-8129 (GPS-1) - Granular Base Layer Backcalculated Statistics.

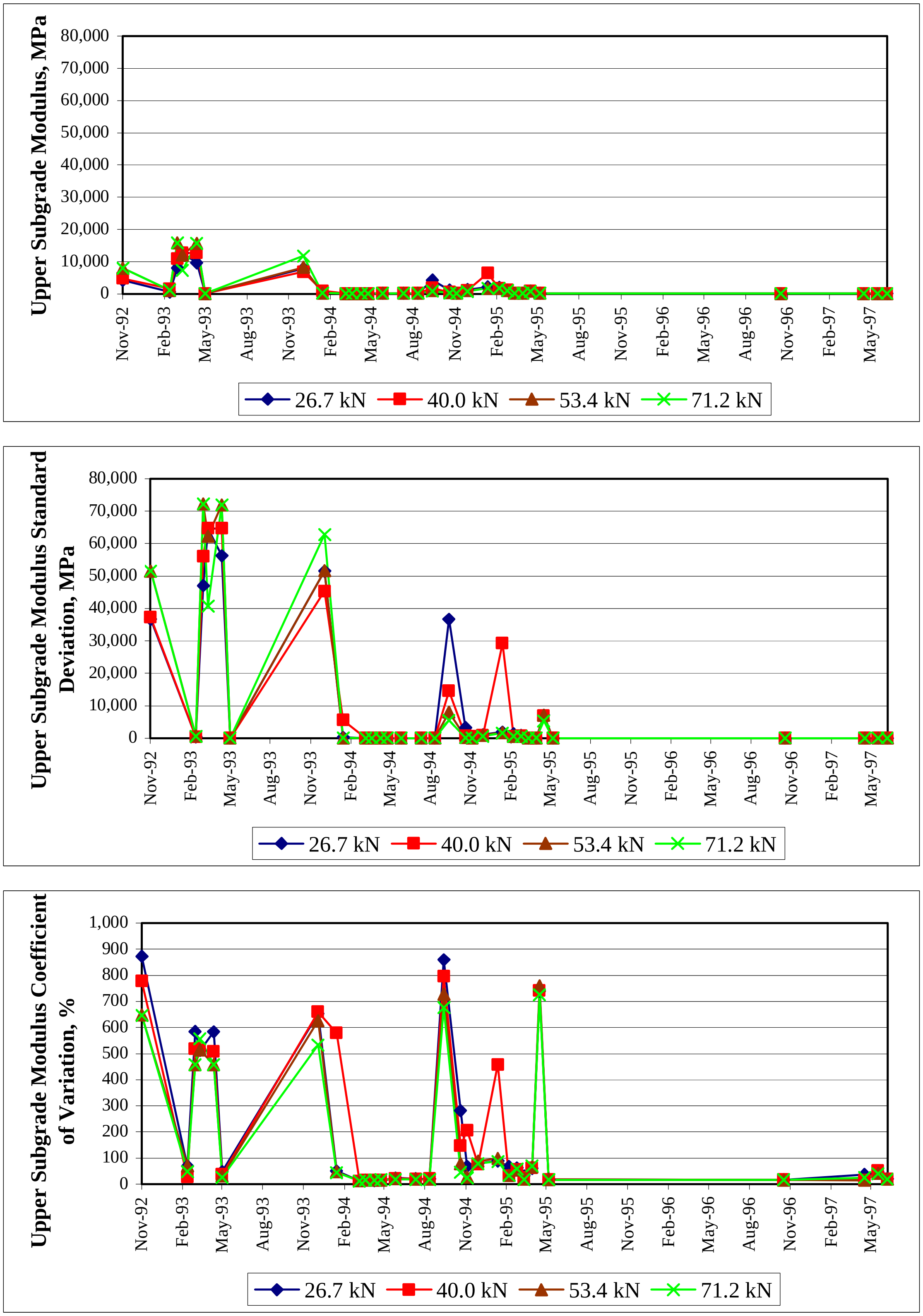


Figure C20. Seasonal Monitoring Site 30-8129 (GPS-1) - Upper Subgrade Backcalculated Statistics.

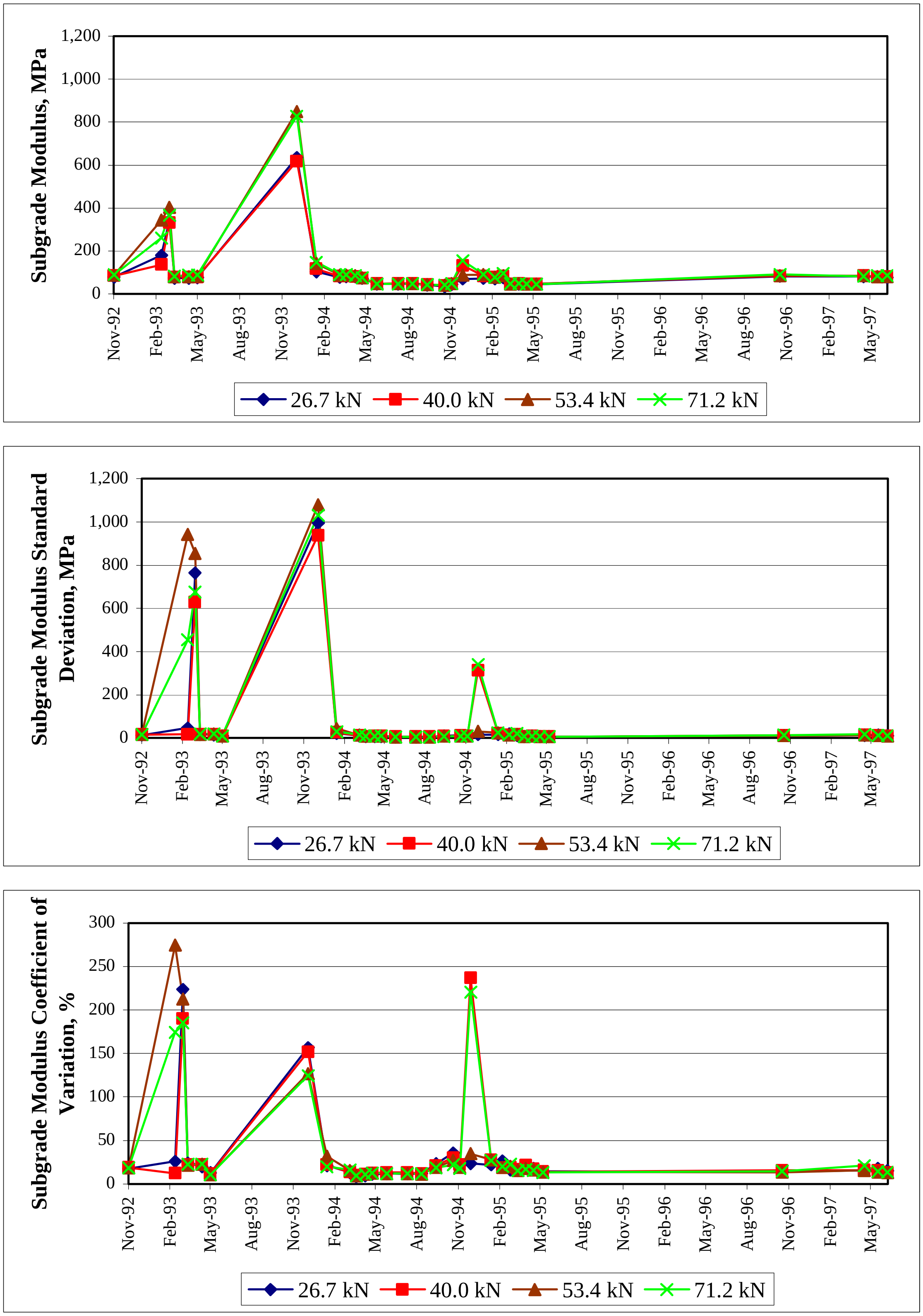


Figure C21. Seasonal Monitoring Site 30-8129 (GPS-1) - Subgrade Layer Backcalculated Statistics.

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Table C1. Definition of the Parameters Included in IMS Table MON_DEFL_FLX_BAKCALC_SECT.

Table Name: **MON_DEFL_FLX_BAKCALC_SECT**

Table Description: Summary of results presented in **MON_DEFL_FLX_BAKCAL_POINT** by section and test date. Results in **MON_DEFL_FLX_BAKCAL_POINT** (see Table C.3) with greater than 2% ERROR_RMSE were excluded from the summary statistics.

FIELD NAME	*Units*	FIELD TYPE	CODES	Data Dictionary Description
STATE_CODE		NUMBER (2,0)	STATE_PROVINCE	Code identifying the state or province.
SHRP_ID		VARCHAR2 (4)		SHRP section identification.
TEST_DATE		DATE		The date the test was performed.
BAKCAL_LAYER_NO		NUMBER (2,0)		Layer numbering scheme used for backcalculation. One is used for the surface layer and is incremented for each additional layer below the surface layer.
DROP_HEIGHT	kN	NUMBER (4,1)	DROP_HEIGHT	An integer code for the height from which the weight was dropped.
CONSTRUCTION_NO		NUMBER (2,0)		Event number indicating pavement layer changes in a section. Set to 1 when a section is chosen for inclusion in the LTPP study and incremented after each pavement layer change. It is in all tables that relate to a section at a specific time.
RECORD_STATUS		VARCHAR2 (1)		A code indicating the general quality of the data as outlined, based on the level of QC checks described in the Data User's Guide.
REF_CONSTRUCTION_NO		NUMBER (2,0)		The CONSTRUCTION_NO used during the backcalculation process to associate the FWD data with the appropriate layer structure at that time.
CALC_PVMT_TEMP_MEAN	°C	NUMBER (4,1)		The average temperature of the CALC_PVMT_TEMP in **MON_DEFL_FLX_BAKCAL_BASIN**. This should approximate the mean surface temperature on a given test date on a given section. Results with greater than 2% RMSE were excluded.
CALC_PVMT_TEMP _STD	°C	NUMBER (4,1)		The standard deviation of the CALC_PVMT_TEMP.
CALC_PVMT_TEMP_MIN	°C	NUMBER (4,1)		The minimum of CALC_PVMT_TEMP.
CALC_PVMT_TEMP_MAX	°C	NUMBER (4,1)		The maximum of CALC_PVMT_TEMP.
RMSE_MAX	%	NUMBER (3,1)		The maximum Root Mean Squared Error (RMSE) calculated by MODCOMP for a given layer on a test date.
ELASTIC_MODULUS_MEAN	MPa	NUMBER (6,1)		The average backcalculated or assumed layer moduli (Young's Modulus) for each layer (including the apparent rigid layer) for each layer in the pavement structure, at each point where the RMSE was less than 2%.
ELASTIC_MODULUS_STD	MPa	NUMBER (6,1)		The standard deviation of the backcalculated layer moduli for a specific layer on a specific date.
ELASTIC_MODULUS_MIN	MPa	NUMBER (6,1)		The minimum layer modulus backcalculated for a specific layer on a specific date.

FIELD NAME	*Units*	FIELD TYPE	CODES	Data Dictionary Description
ELASTIC_MODULUS_MAX	MPa	NUMBER (6,1)		The maximum layer modulus backcalculated for a specific layer on a specific date.
TOTAL_NO_BASINS		NUMBER (3,0)		The total number of basins included in summary statistics.
SECTION_CHARACTERIZATION		VARCHAR2 (5)	DRIFT, JUMP	The word "Drift" is used if the deflections across the test section had a consistent slope. The word "JUMP" is used if the deflections experienced a significant change.
DATA_PROCESS_EXTRACT_ DATE		DATE		Date data was extracted from related IMS tables for computed parameter processing.
COMMENT_MODULUS		VARCHAR2 (200)		Flexible pavement backcalculation section statistics comment.

Table C2. Definition of the Parameters Included in IMS Table MON_DEFL_FLX_BAKCALC_LAYER.

Table Name: **MON_DEFL_FLX_BAKCAL_LAYER**
Table Description: Layer structure and materials inputs used in flexible pavement elastic modulus backcalculation process.

FIELD NAME	*Units*	FIELD TYPE	CODES	Data Dictionary Description
STATE_CODE		NUMBER (2,0)	STATE_PROVINCE	Code identifying the state or province.
SHRP_ID		VARCHAR2 (4)		SHRP section identification.
CN_REF_DATE		DATE		Date used to set CONSTRUCTION_NO.
BAKCAL_LAYER_NO		NUMBER (2,0)		Layer number scheme used for backcalculation. One is used for the surface layer and is incremented for each additional layer below the surface layer.
RECORD_STATUS		VARCHAR2 (1)		A code indicating the general quality of the data as outlined, based on the level of QC, described in the Data User's Guide.
CONSTRUCTION_NO		NUMBER (2,0)		Event number indicating pavement layer changes in a section. Set to 1 when a section is chosen for inclusion in the LTPP study and incremented after each pavement layer change. It is in all tables that relate to a section at a specific time.
REF_CONSTRUCTION_NO		NUMBER (2,0)		The CONSTRUCTION_NO used during the backcalculation process to associate the FWD data with the appropriate layer structure at that time.
BAKCAL_LAYER_THICKNESS	mm	NUMBER (3,0)		The thickness of each individual layer number used in the backcalculation process.
BAKCAL_POISSON_RATIO		NUMBER (3,2)		Poisson's ratio for each material type or layer.
LAYER_TYPE		VARCHAR2 (2)		A code identifying the type of layer.
LAYER_DENSITY	kg/m^3	NUMBER (4,0)		The wet density or unit weight of each individual layer used in the backcalculation process.
AT_REST_PRESSURE_COEFFICIENT		NUMBER (3,2)		The at-rest pressure coefficient of each unbound pavement layer, including the subgrade, used in the backcalculation process.
L05B_LAYER_NO_1		NUMBER (2,0)		The first layer in the **TST_L05B** table to which the backcalculation layer corresponds.
L05B_LAYER_NO_2		NUMBER (2,0)		The second layer in the **TST_L05B** table to which the

FIELD NAME	*Units*	FIELD TYPE	CODES	Data Dictionary Description
				backcalculation layer corresponds.
L05B_LAYER_NO_3		NUMBER (2,0)		The third layer in the **TST_L05B** table to which the backcalculation layer corresponds.
L05B_LAYER_NO_4		NUMBER (2,0)		The fourth layer in the **TST_L05B** table to which the backcalculation layer corresponds.
L05B_LAYER_NO_5		NUMBER (2,0)		The fifth layer in the **TST_L05B** table to which the backcalculation layer corresponds.
L05B_LAYER_NO_6		NUMBER (2,0)		The sixth layer in the **TST_L05B** table to which the backcalculation layer corresponds.
L05B_LAYER_NO_7		NUMBER (2,0)		The seventh layer in the **TST_L05B** table to which the backcalculation layer corresponds.
DATA_PROCESS_EXTRACT_DATE		DATE		Date data was extracted from related IMS tables for computed parameter processing.

Table C3. Definition of the Parameters Included in IMS Table MON_DEFL_FLX_BAKCALC_POINT.

Table Name: **MON_DEFL_FLX_BAKCAL_POINT**

Table Description: Interpreted results of backcalculated elastic layer moduli from Falling Weight Deflectometer measurements for flexible pavement structures performed using Version 4.2 of the MODCOMP computer program.

Field Name	*Units*	FIELD TYPE	Codes	Data Dictionary Description
STATE_CODE		NUMBER (2,0)	STATE_PROVINCE	Code identifying the state or province.
SHRP_ID		VARCHAR2 (4)		SHRP section identification.
TEST_DATE		DATE		The date the test was performed.
BAKCAL_LAYER_NO		NUMBER (2,0)		Layer number scheme used for backcalculation. One is used for the surface layer and is incremented for each additional layer below the surface layer.
LANE_NO		VARCHAR2 (2)	LANE_SPEC	A code indicating the lane or position where the deflection test was performed.
FWD_PASS		NUMBER (1,0)		Whole number indicating the number of passes of deflection testing along each lane. One is used for the first pass and is incremented for each additional pass of deflection testing along the test section.
POINT_LOC	m	NUMBER (6,1)		The distance from the start of the test section to where the test was performed.
DROP_HOUR_MINUTE		VARCHAR2 (4)		The time the drop was made.
DROP_HEIGHT		VARCHAR2 (1)	DROP_HEIGHT	An integer code for the height from which the weight was dropped.
CONSTRUCTION_NO		NUMBER (2,0)		Event number indicating pavement layer changes in a section. Set to 1 when a section is chosen for inclusion in the LTPP study and incremented after each pavement layer change. It is in all tables that relate to a section at a specific time.
RECORD_STATUS		VARCHAR2 (1)		A code indicating the general quality of the data as outlined, based on the level of QC checks described in the Data User's Guide.

Field Name	*Units*	FIELD TYPE	Codes	Data Dictionary Description
REF_CONSTRUCTION_NO		NUMBER (2,0)		The CONSTRUCTION_NO used during the backcalculation process to associate the FWD data with the appropriate layer structure at that time.
ERROR_RMSE	%	NUMBER (3,1)		Root Mean Squared Error (RMSE) calculated by MODCOMP which represents the difference between calculated and measured deflection basin.
ELASTIC_MODULUS	Mpa	NUMBER (6,1)		The backcalculated or assumed layer moduli (Young's Modulus) for each layer (including the apparent rigid layer) in the pavement structure, at each point.
ELASTIC_MODULUS_FLAG		VARCHAR2 (1)	OUTLIER_FLAG	ELASTIC_MODULUS outlier flag code.
SECTION_STAT_INCLUDE _FLAG		VARCHAR2 (1)	SECTION_STAT_ INCLUDE_FLAG	Section statistics inclusion flag code.
MODULUS_ASSUMED		VARCHAR2 (1)	Y, N	"Y" indicates layer elastic modulus was assumed, "N" indicates layer elastic modulus was calculated..
CALC_PVMT_TEMP	°C	NUMBER (4,1)		Temperature at mid-depth of the top layer of the structure used in backcalculation. Data were obtained from **MON_DEFL_TEMP_VALUES** for the end closest to the POINT_LOC being examined. Data is interpolated to the appropriate DROP_HOUR_MINUTE.
DATA_PROCESS_ EXTRACT_DATE	NA	DATE		Date data was extracted from related IMS tables for computed parameter processing.

Code Tables for **MON_DEFL_FLX_BAKCAL_POINT**
Existing IMS codes referenced in the schema: STATE_PROVINCE, LANE_SPEC and DROP_HEIGHT

CODE_TYPE: OUTLIER_FLAG
TITLE: Outlier flag code
SOURCE: MON_T_PROF_INDEX IMS table specifications

CODE TYPE	CODE	DETAIL	ADDL_CODE
OUTLIER_FLAG	1	Referenced field less than 2 standard deviations from section mean	
OUTLIER_FLAG	2	Referenced field greater than or equal to 2 standard deviations from section mean.	

CODE_TYPE: SECTION_STAT_INCLUDE_FLAG
TITLE: Section statistics inclusion flag
SOURCE: MON_T_PROF_INDEX IMS table specifications

CODE TYPE	CODE	DETAIL	ADDL_CODE
SECTION_STAT_INCLUDE_FLAG	1	Parameters in record were included in section statistics	
SECTION_STAT_INCLUDE_FLAG	2	Parameters in record were excluded from section statistics	

APPENDIX D - CORRELATION MATRIX FOR UNBOUND MATERIAL PARAMETERS

	Coef $Log(Oct)^2$	Coef Log(Oct)	Coef. Log(Bulk)	Intercept	t-stat Coef.3	t-stat Coef.2	t-stat Coef.1	t-stat Intercept	R-squared	Std Err of Est.	F-Statistic	Moist. Cont.	Density	Optimum M.C.	Max. Density	Fine (%)	Clay (%)	Liguid Limit	Plasticity Index	Plast. Lmt.	Rel. Density	Rel. Moist.
Coef $Log(Oct)^2$	1.000																					
Coef Log(Oct)	0.893	1.000																				
Coef. Log(Bulk)	-0.112	-0.328	1.000																			
Intercept	0.370	0.673	-0.609	1.000																		
t-stat Coef.3	0.717	0.623	0.161	0.127	1.000																	
t-stat Coef.2	0.591	0.602	0.120	0.217	0.955	1.000																
t-stat Coef.1	0.107	0.015	0.209	0.025	-0.136	-0.272	1.000															
t-stat Intercept	-0.100	-0.032	-0.161	0.248	-0.573	-0.612	0.536	1.000														
R-squared	0.119	-0.010	0.169	-0.072	-0.118	-0.223	0.507	0.341	1.000													
Std Err of Est.	-0.058	-0.028	0.061	-0.161	0.295	0.361	-0.465	-0.607	-0.614	1.000												
F-Statistic	-0.061	-0.109	0.021	-0.040	-0.384	-0.517	0.715	0.578	0.261	-0.311	1.000											
Moist. Cont.	-0.181	-0.247	-0.161	-0.326	-0.214	-0.241	-0.241	0.011	-0.091	0.051	-0.005	1.000										
Density	0.159	0.186	0.067	0.274	0.138	0.139	0.133	0.049	0.042	-0.049	0.022	-0.728	1.000									
Optimum M.C.	-0.156	-0.158	-0.159	-0.157	-0.188	-0.192	-0.165	0.083	-0.044	-0.033	-0.003	0.650	-0.594	1.000								
Max. Density	0.164	0.153	0.141	0.164	0.191	0.186	0.117	-0.066	0.026	0.019	-0.038	-0.670	0.776	-0.881	1.000							
Fine (%)	-0.174	-0.249	-0.159	-0.268	-0.262	-0.312	-0.163	0.108	-0.056	-0.052	0.062	0.591	-0.352	0.457	-0.445	1.000						
Clay (%)	-0.270	-0.281	-0.206	-0.173	-0.315	-0.312	-0.242	0.122	-0.139	-0.015	0.041	0.662	-0.437	0.533	-0.522	0.726	1.000					
Liguid Limit	-0.280	-0.275	-0.259	-0.118	-0.348	-0.334	-0.250	0.214	-0.124	-0.100	0.023	0.662	-0.375	0.539	-0.480	0.594	0.767	1.000				
Plasticity Index	-0.274	-0.249	-0.241	-0.106	-0.366	-0.342	-0.192	0.252	-0.076	-0.139	0.046	0.624	-0.406	0.518	-0.517	0.554	0.791	0.889	1.000			
Plast. Lmt.	-0.220	-0.237	-0.216	-0.102	-0.247	-0.247	-0.251	0.123	-0.145	-0.036	-0.006	0.546	-0.254	0.432	-0.328	0.495	0.560	0.879	0.563	1.000		
Rel. Density	-0.027	0.017	-0.064	0.077	-0.044	-0.034	-0.022	0.088	0.005	-0.036	0.027	-0.009	0.097	0.639	-0.421	0.070	0.078	0.093	0.085	0.078	1.000	
Rel. Moist.	-0.071	-0.157	-0.029	-0.277	-0.059	-0.088	-0.197	-0.117	-0.109	0.172	-0.039	0.637	-0.281	-0.017	0.004	0.271	0.271	0.262	0.202	0.263	-0.295	1.000

APPENDIX E – TRAFFIC LOAD – RELATED FIGURES

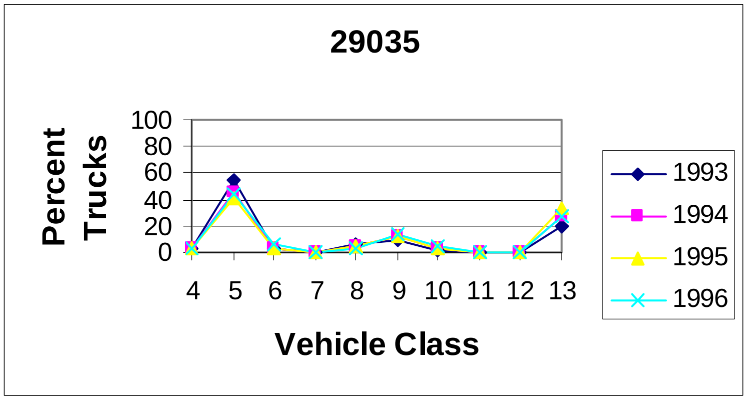


Figure E1 – Vehicle Class Distribution For Section 029035

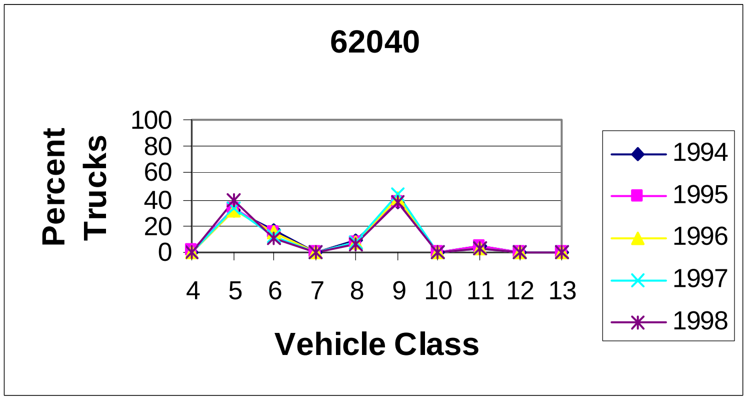


Figure E2 – Vehicle Class Distribution For Section 062040

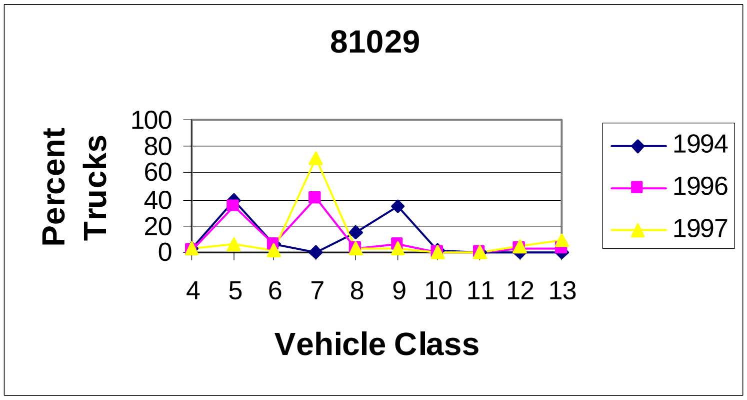


Figure E3 – Vehicle Class Distribution For Section 081029

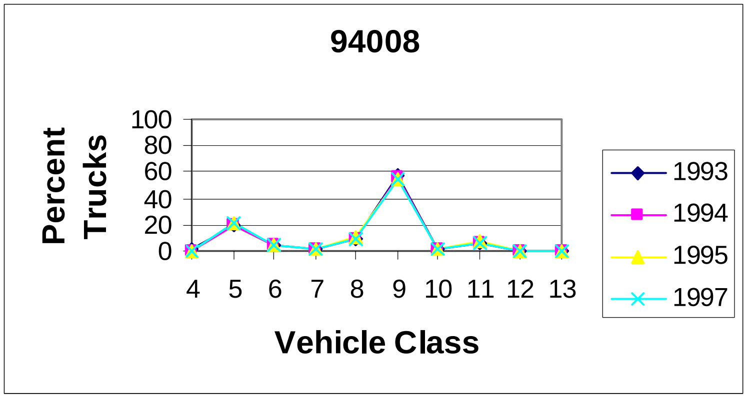


Figure E4 – Vehicle Class Distribution For Section 094008

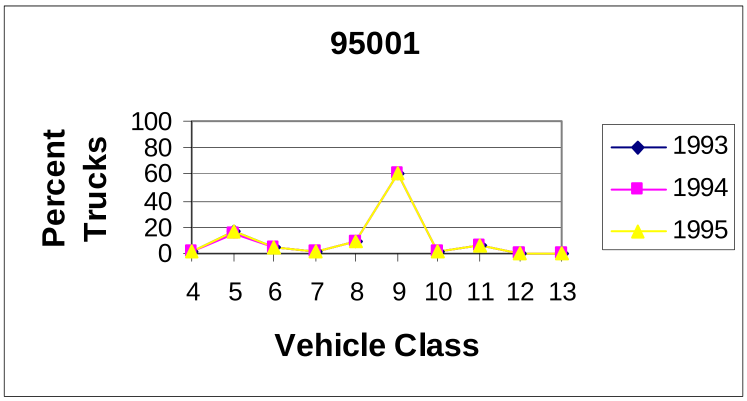


Figure E5 – Vehicle Class Distribution For Section 095001

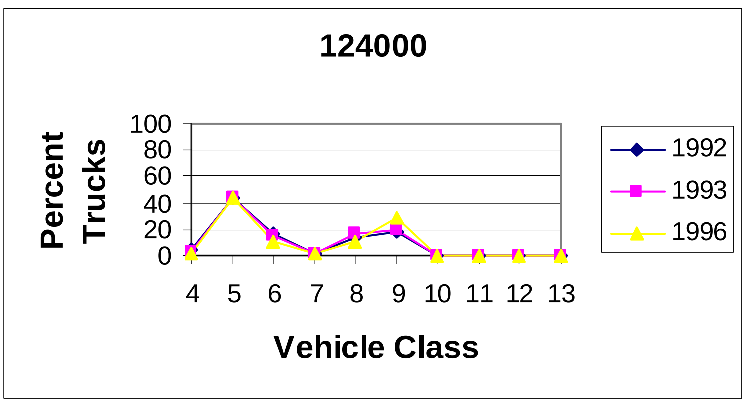


Figure E6 – Vehicle Class Distribution For Section 124000

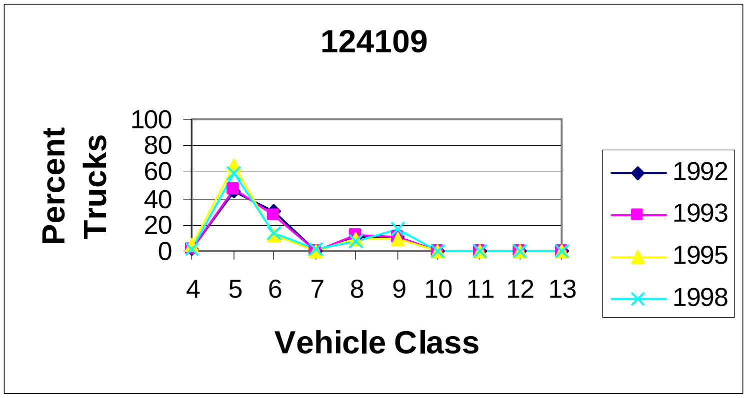


Figure E7 – Vehicle Class Distribution For Section 124109

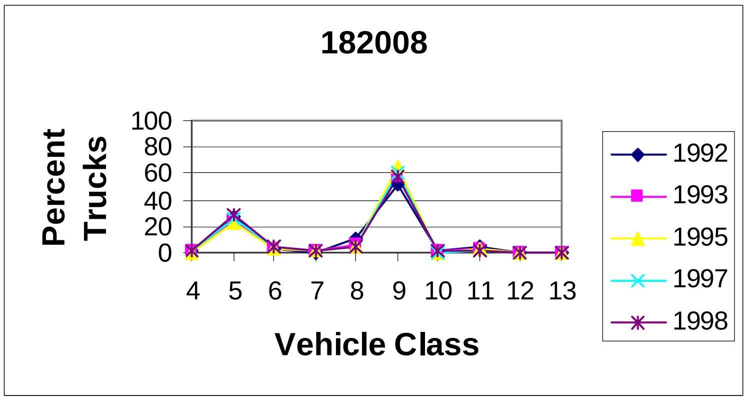


Figure E8 – Vehicle Class Distribution For Section 182008

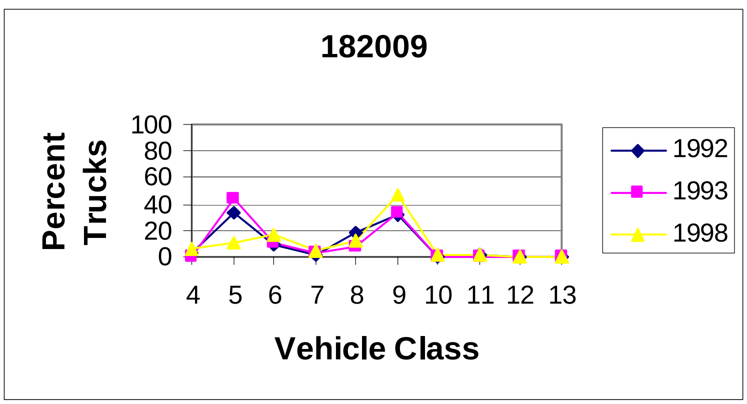


Figure E9 – Vehicle Class Distribution For Section 182009

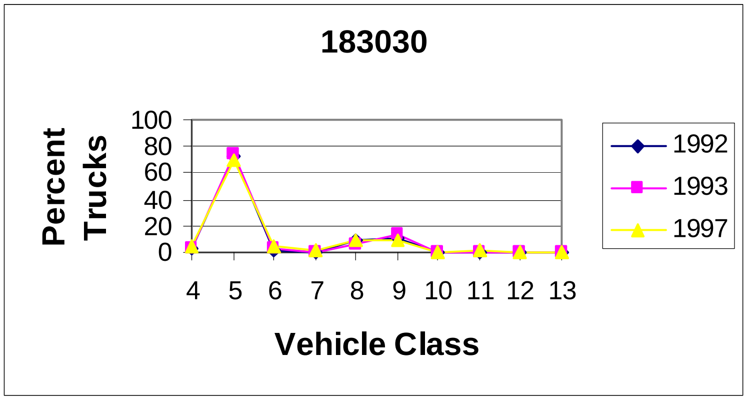


Figure E10 – Vehicle Class Distribution For Section 183030

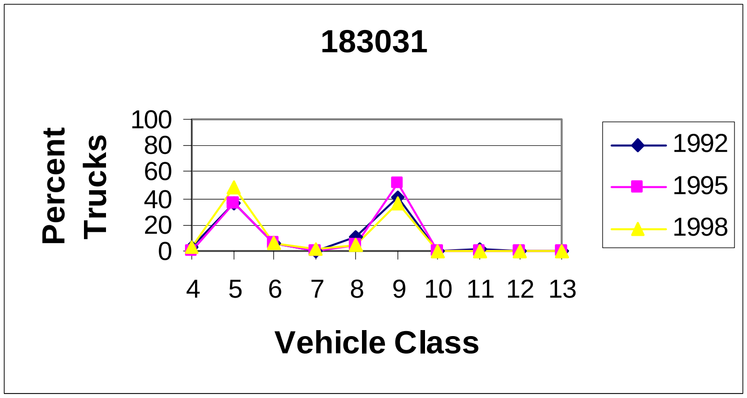


Figure E11 – Vehicle Class Distribution For Section 183031

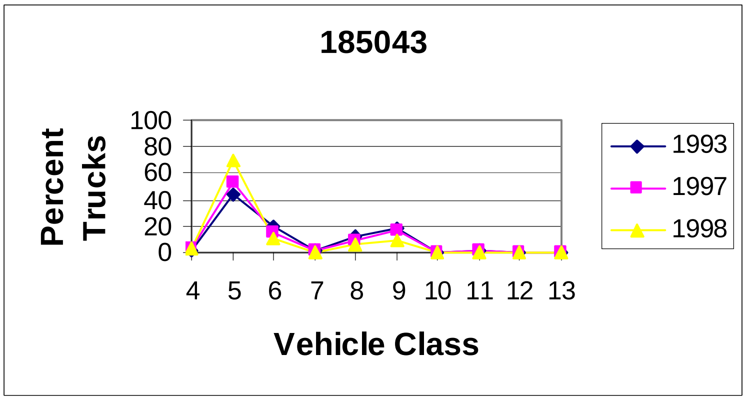


Figure E12 – Vehicle Class Distribution For Section 185043

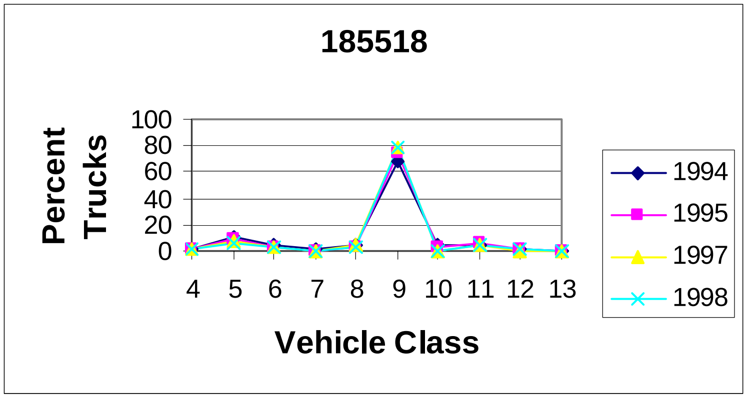


Figure E13 – Vehicle Class Distribution For Section 185518

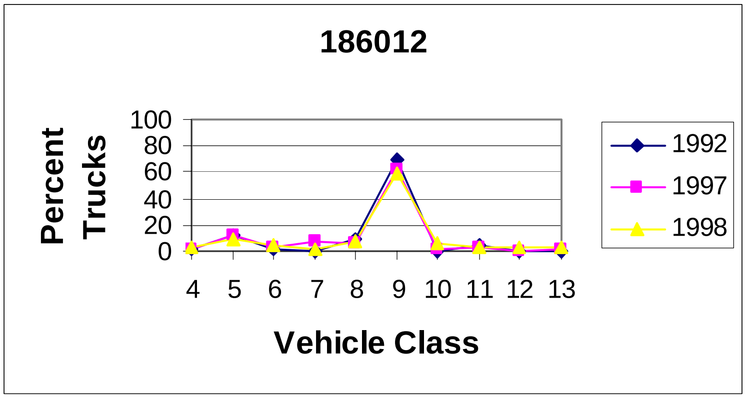


Figure E14 – Vehicle Class Distribution For Section 186012

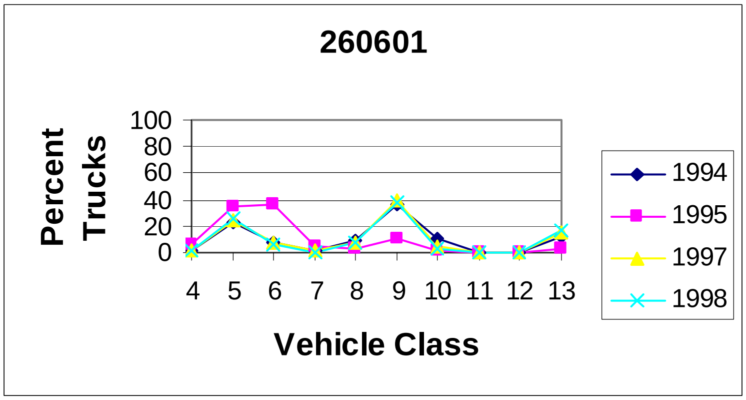


Figure E15 – Vehicle Class Distribution For Section 260601

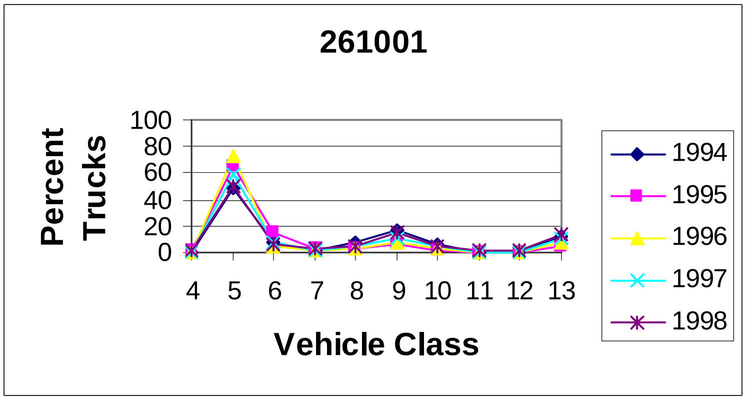


Figure E16 – Vehicle Class Distribution For Section 261001

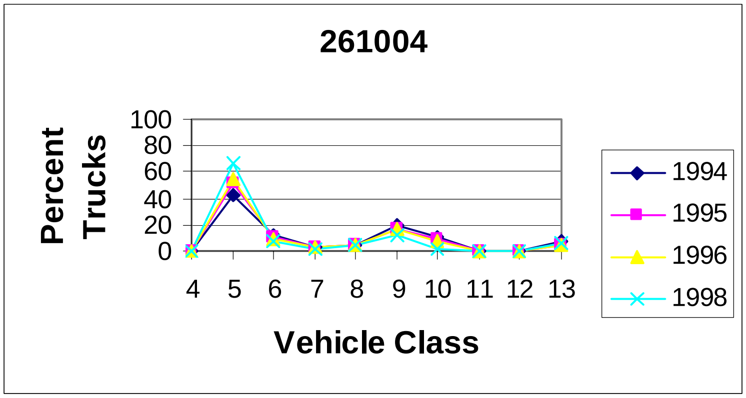


Figure E17 – Vehicle Class Distribution For Section 261004

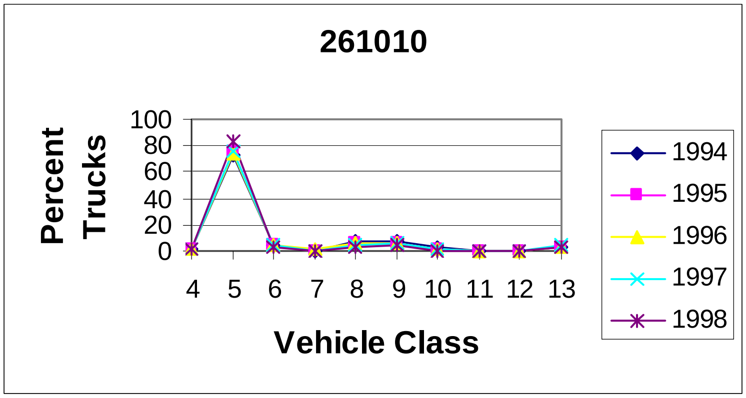


Figure E18 – Vehicle Class Distribution For Section 261010

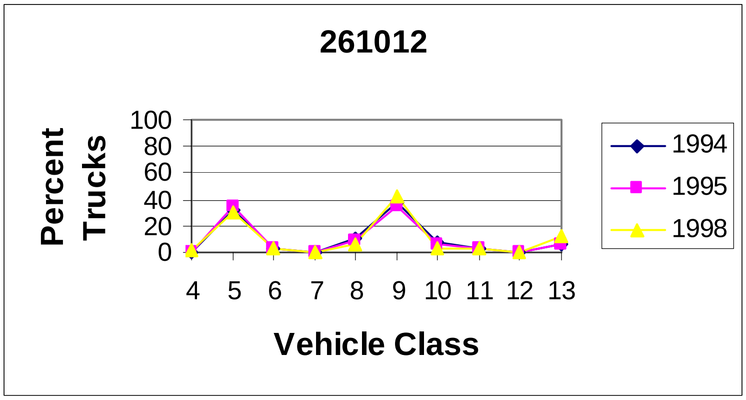


Figure E19 – Vehicle Class Distribution For Section 261012

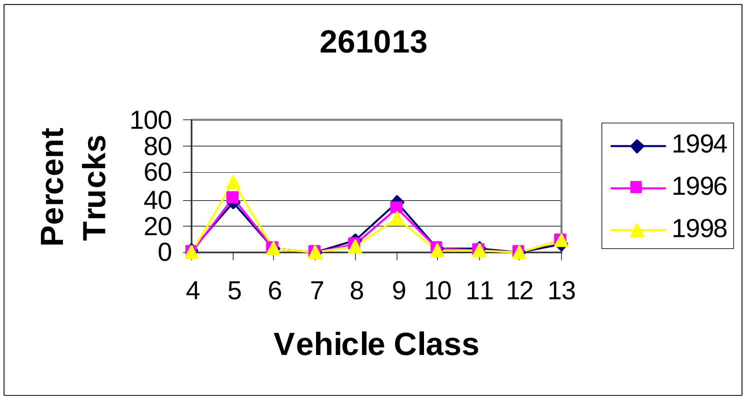


Figure E20 – Vehicle Class Distribution For Section 261013

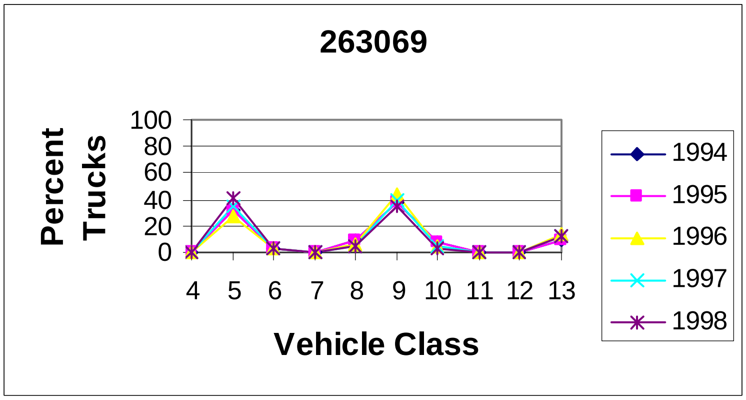


Figure E21 – Vehicle Class Distribution For Section 263069

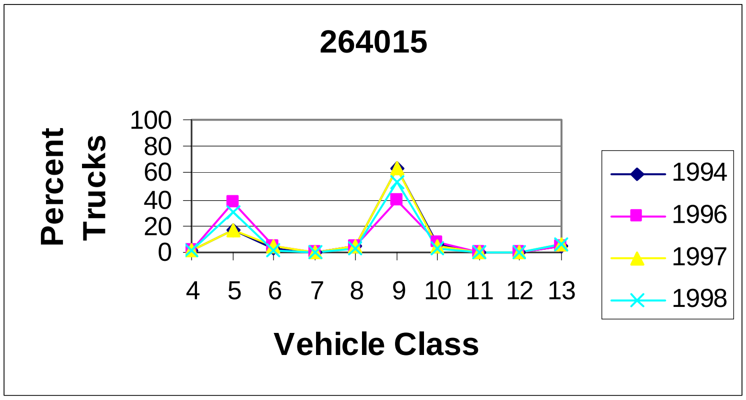


Figure E22 – Vehicle Class Distribution For Section 264015

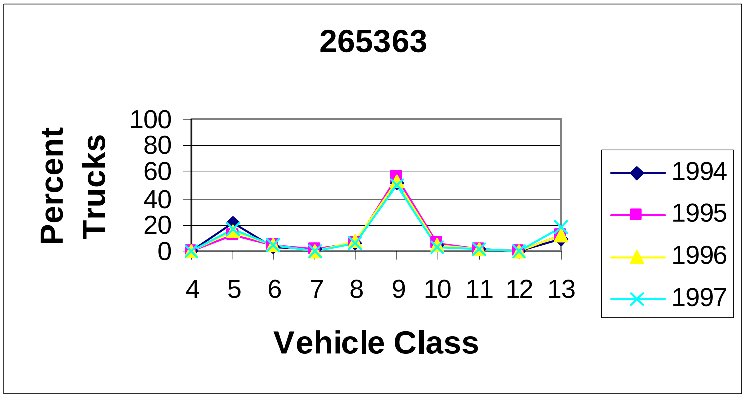


Figure E23 – Vehicle Class Distribution For Section 265363

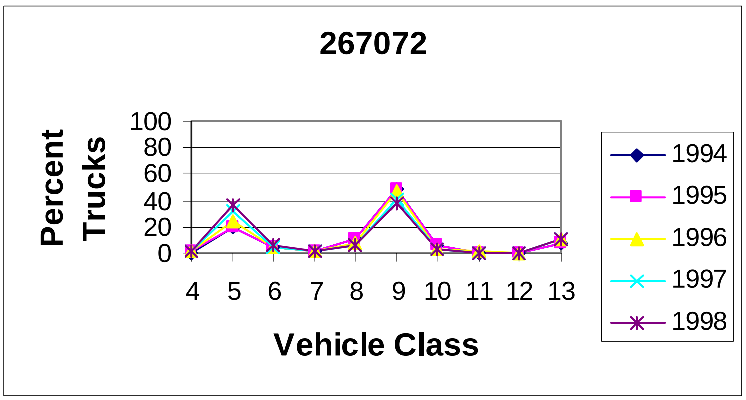


Figure E24 – Vehicle Class Distribution For Section 267072

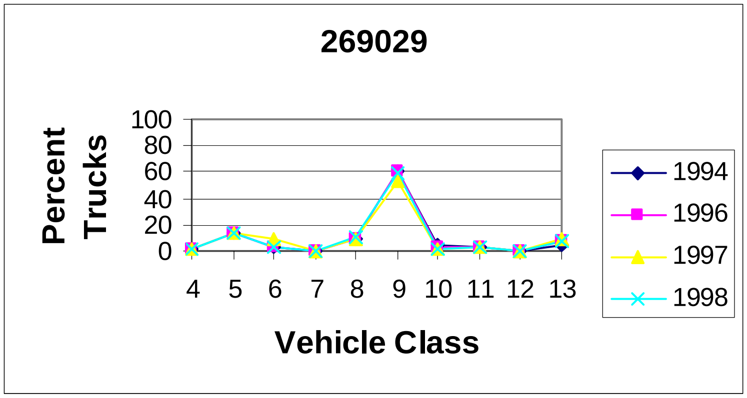


Figure E25 – Vehicle Class Distribution For Section 269029

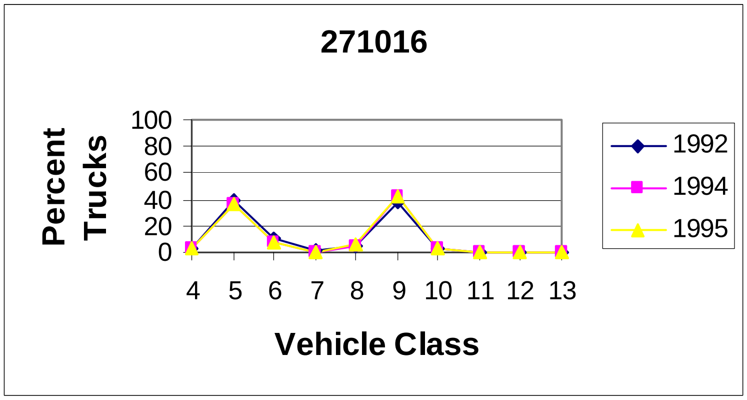


Figure E26 – Vehicle Class Distribution For Section 271016

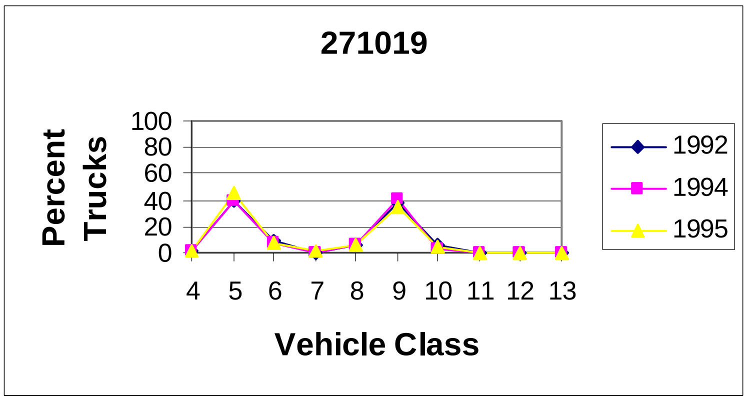


Figure E27 – Vehicle Class Distribution For Section 271019

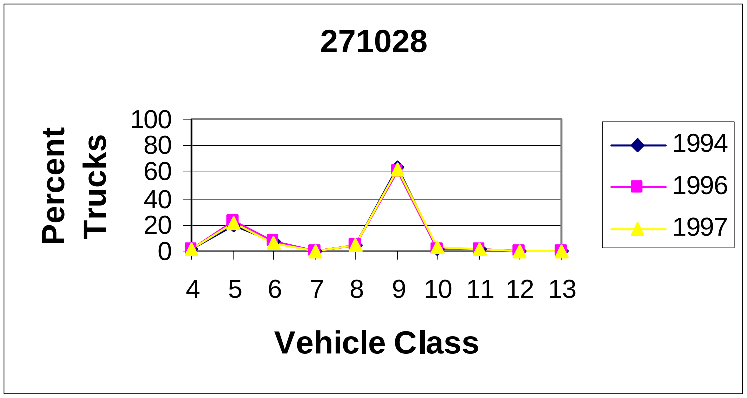


Figure E28 – Vehicle Class Distribution For Section 271028

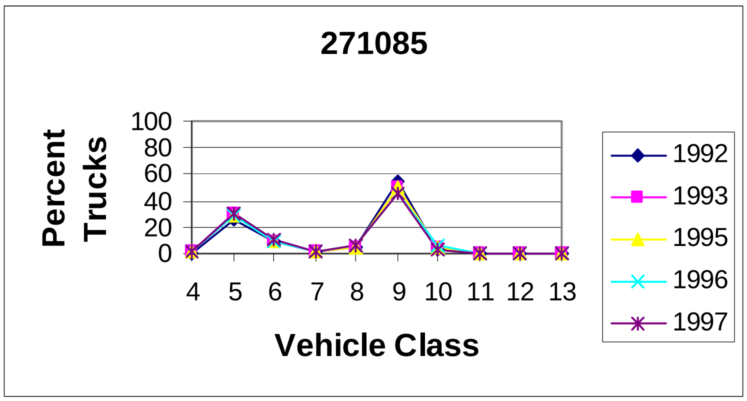


Figure E29 – Vehicle Class Distribution For Section 271085

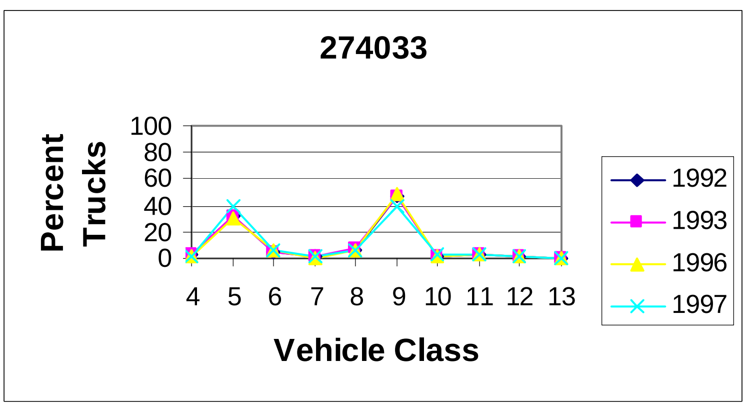


Figure E30 – Vehicle Class Distribution For Section 274033

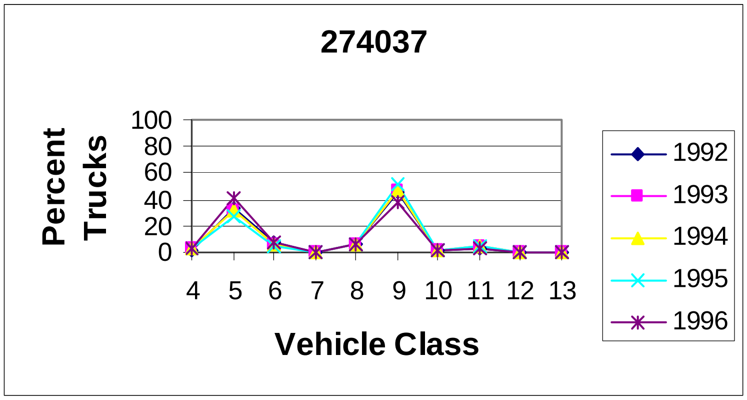


Figure E31 – Vehicle Class Distribution For Section 274037

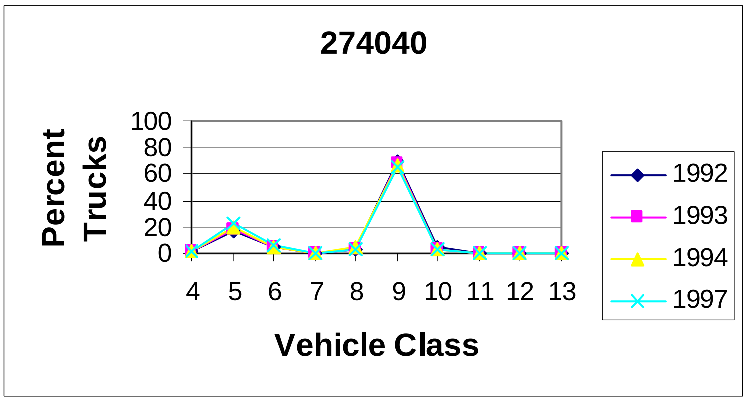


Figure E32 – Vehicle Class Distribution For Section 274040

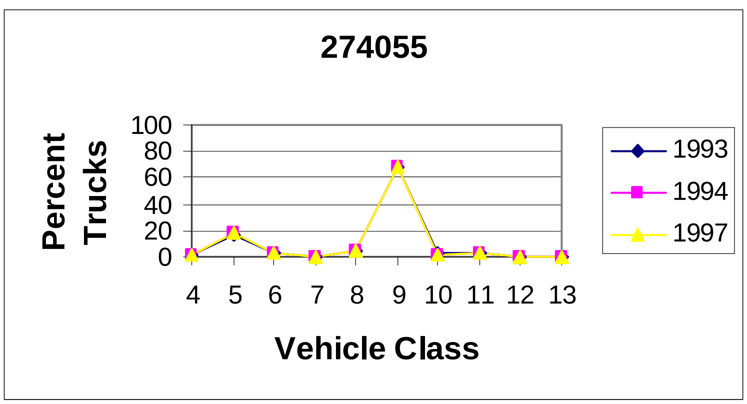


Figure E33 – Vehicle Class Distribution For Section 274055

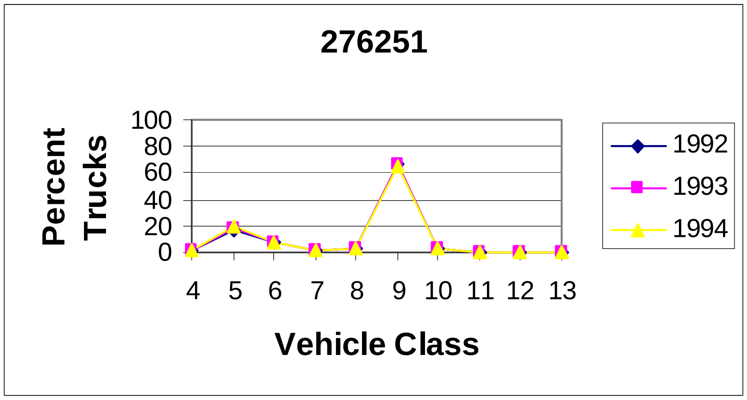


Figure E34 – Vehicle Class Distribution For Section 276251

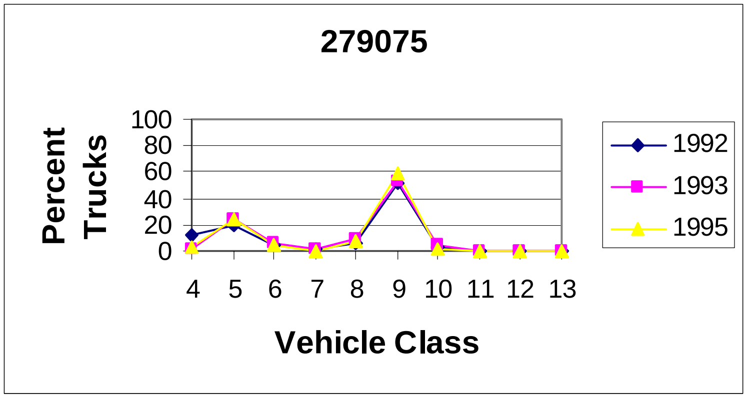


Figure E35 – Vehicle Class Distribution For Section 279075

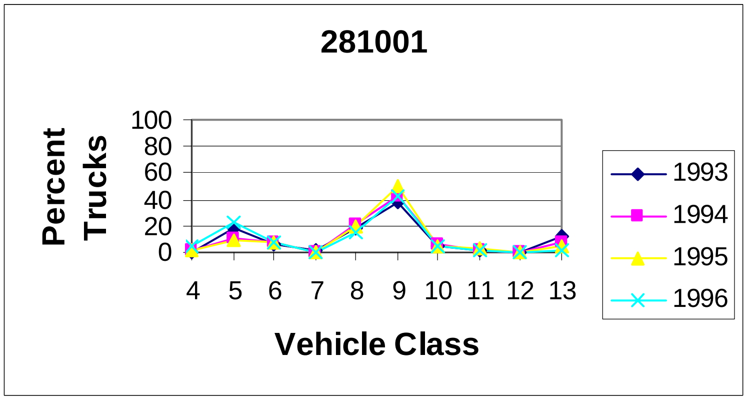


Figure E36 – Vehicle Class Distribution For Section 281001

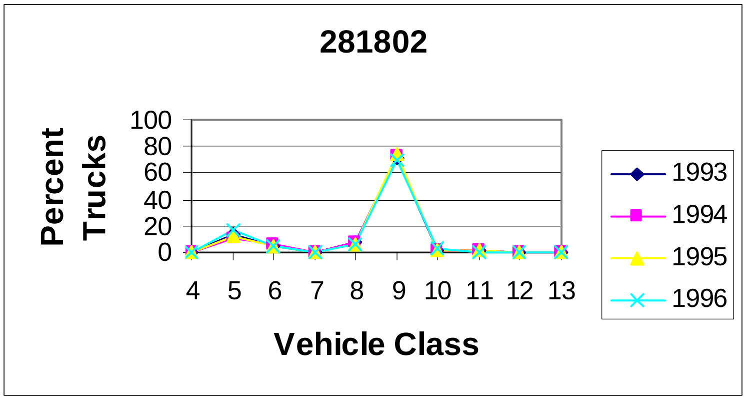


Figure E37 – Vehicle Class Distribution For Section 281802

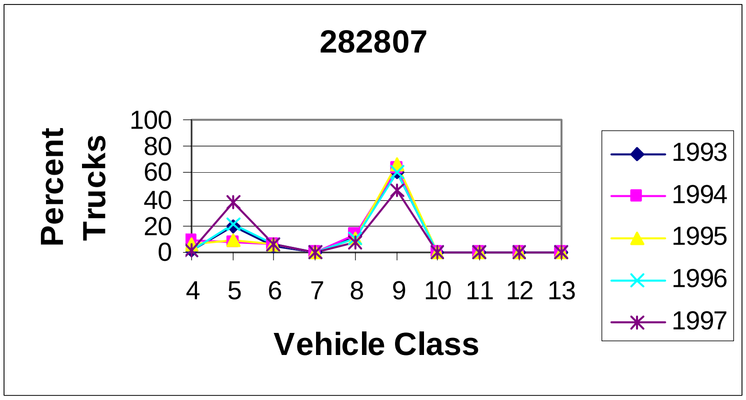


Figure E38 – Vehicle Class Distribution For Section 282807

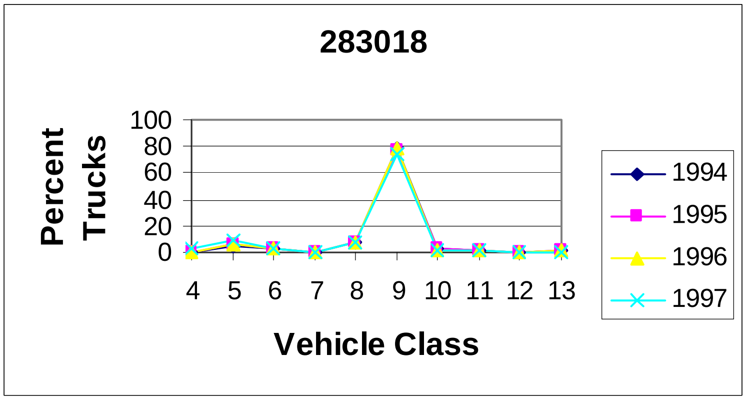


Figure E39 – Vehicle Class Distribution For Section 283018

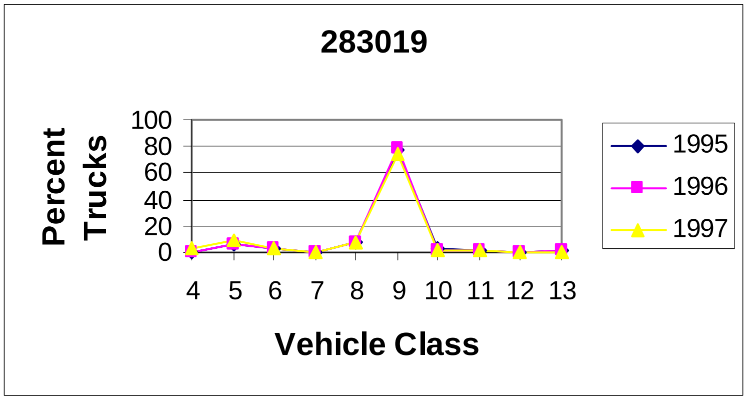


Figure E40 – Vehicle Class Distribution For Section 283019

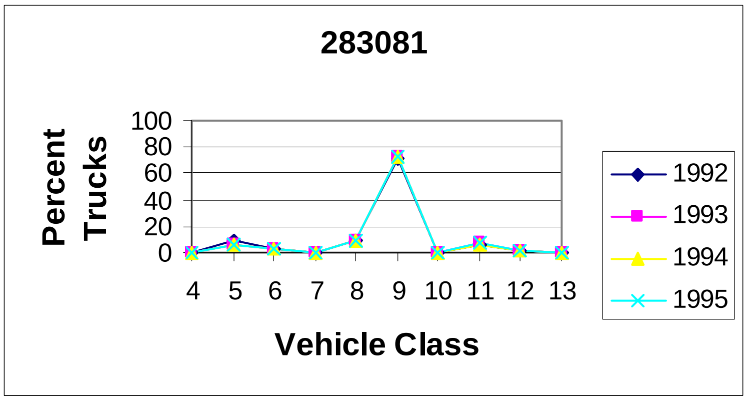


Figure E41 – Vehicle Class Distribution For Section 283081

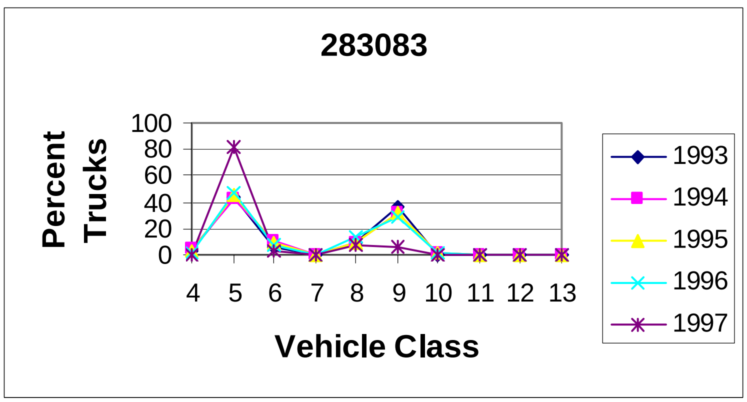


Figure E42 – Vehicle Class Distribution For Section 283083

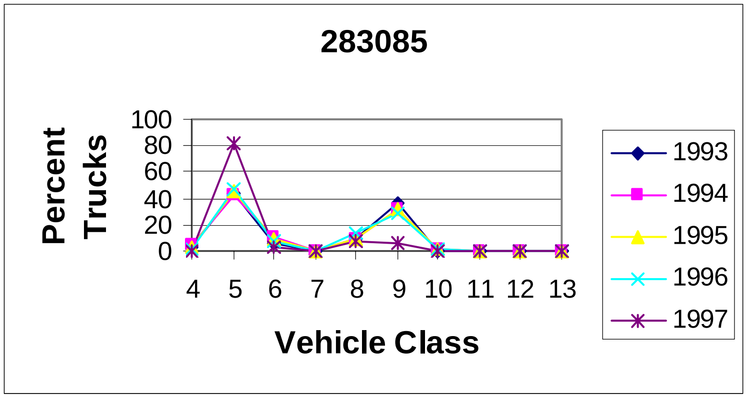


Figure E43 – Vehicle Class Distribution For Section 283085

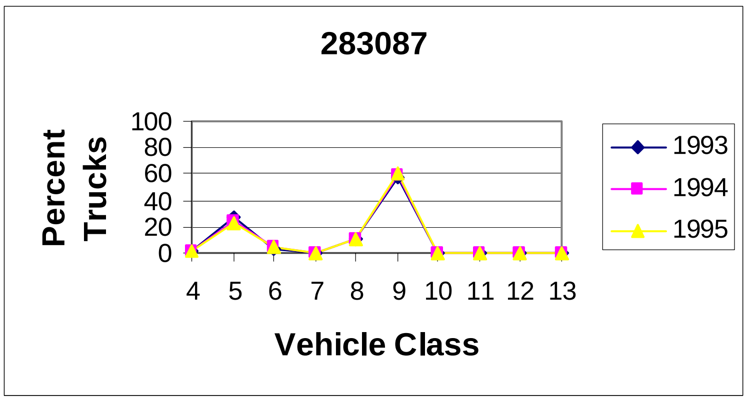


Figure E44 – Vehicle Class Distribution For Section 283087

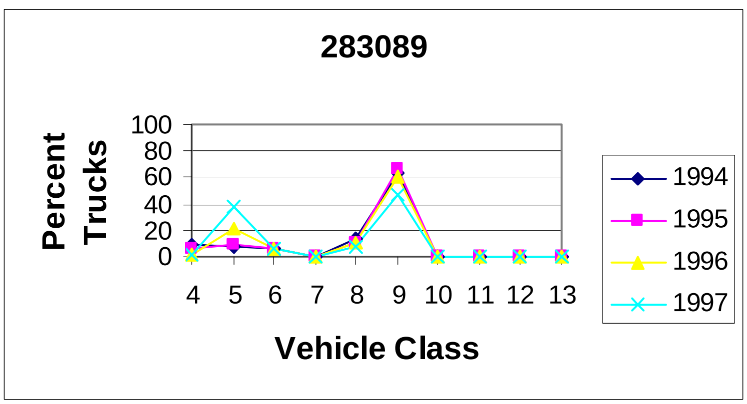


Figure E45 – Vehicle Class Distribution For Section 283089

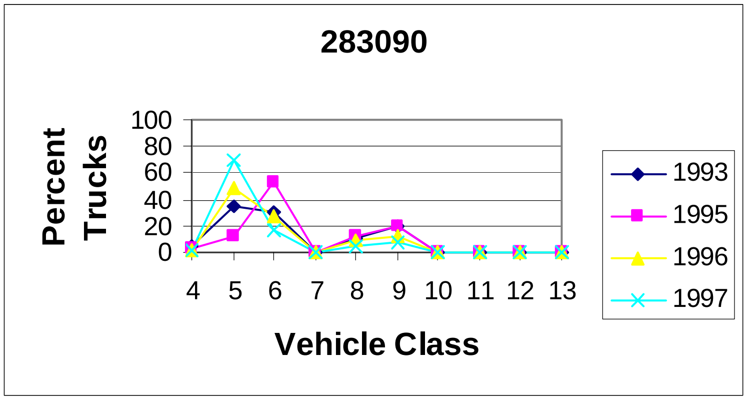


Figure E46 – Vehicle Class Distribution For Section 283090

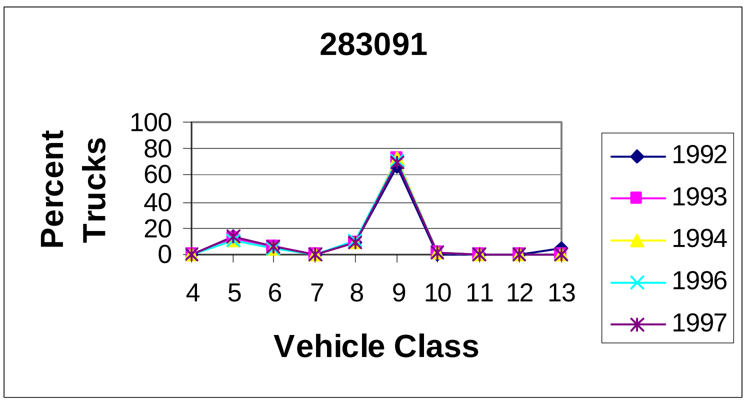


Figure E47 – Vehicle Class Distribution For Section 283091

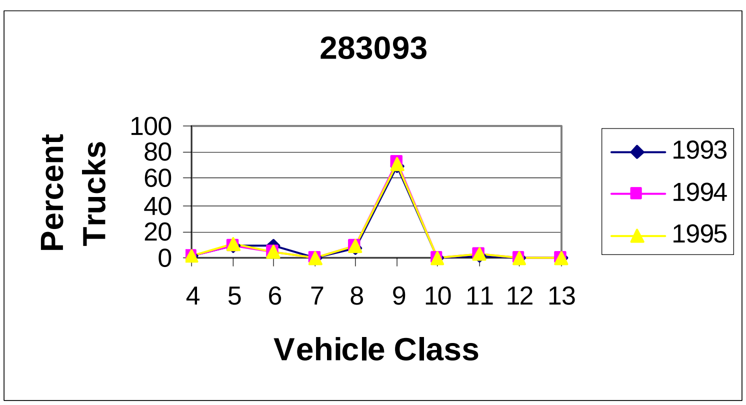


Figure E48 – Vehicle Class Distribution For Section 283093

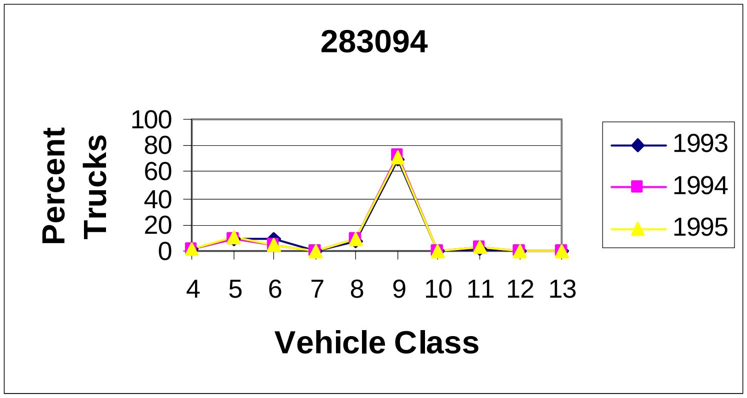


Figure E49 – Vehicle Class Distribution For Section 283094

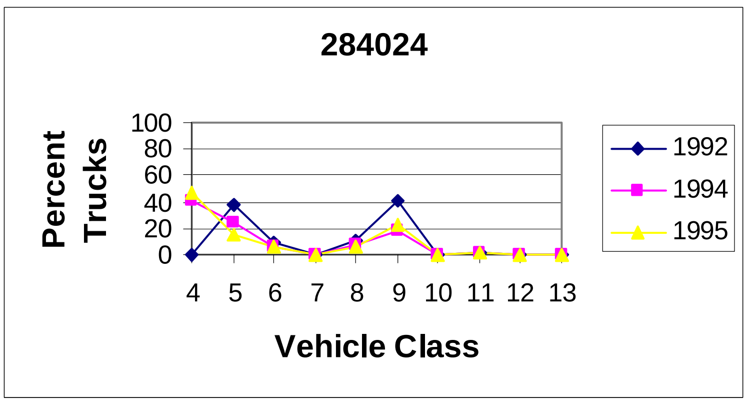


Figure E50 – Vehicle Class Distribution For Section 284024

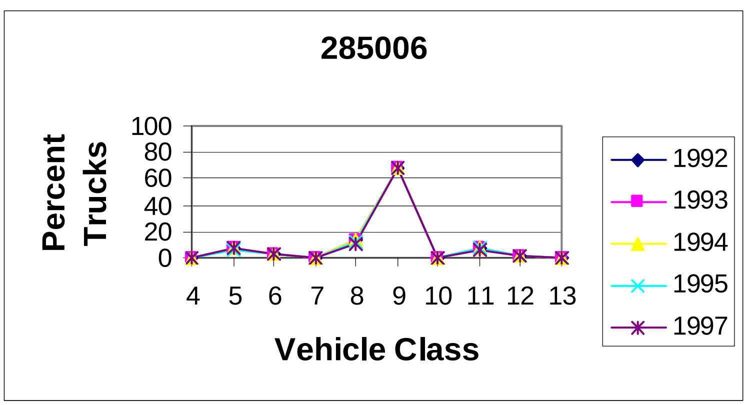


Figure E51 – Vehicle Class Distribution For Section 285006

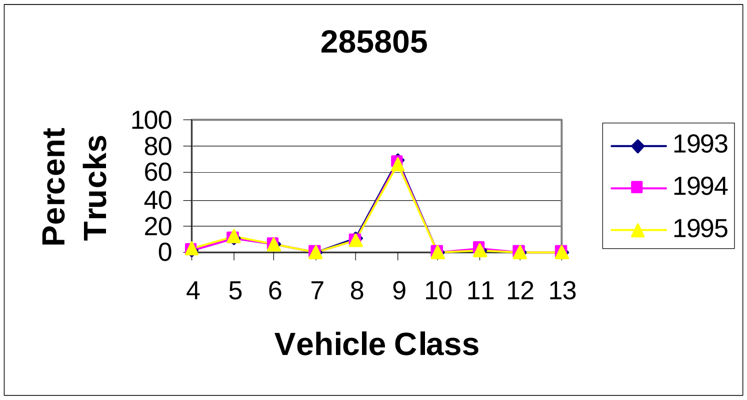


Figure E52 – Vehicle Class Distribution For Section 285805

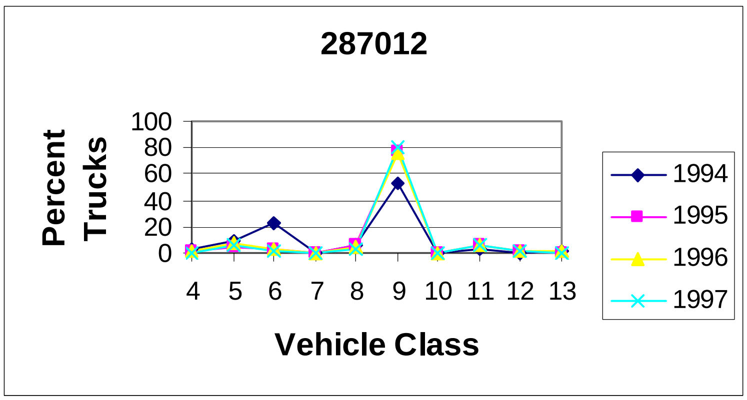


Figure E53 – Vehicle Class Distribution For Section 287012

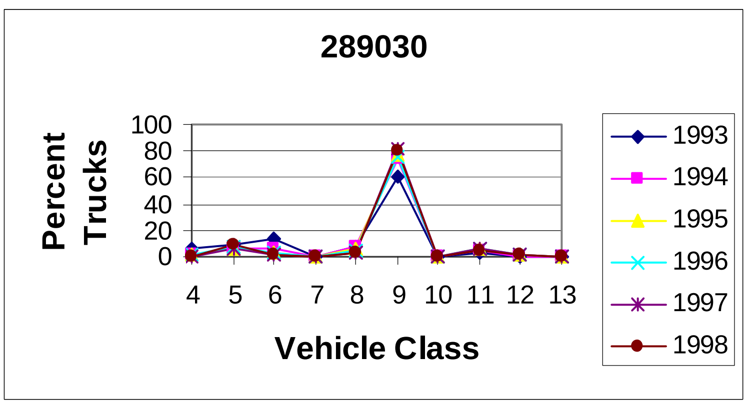


Figure E54 – Vehicle Class Distribution For Section 289030

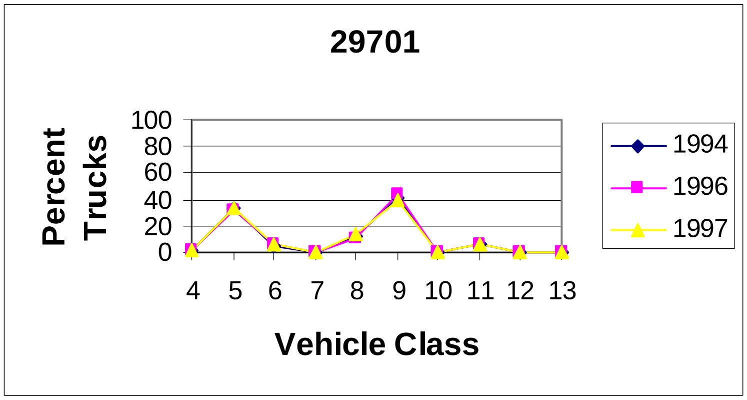


Figure E55 – Vehicle Class Distribution For Section 290701

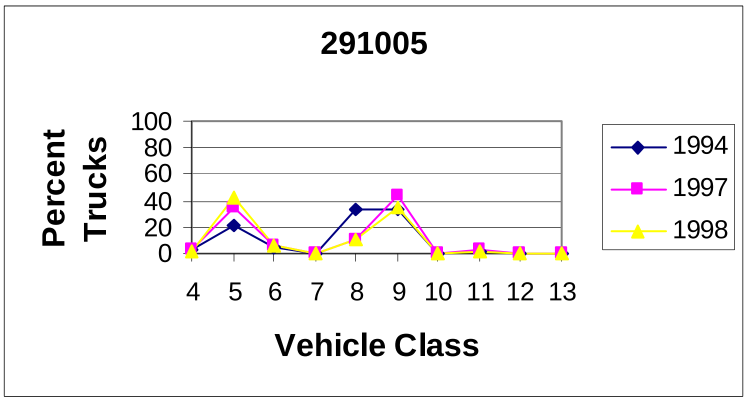


Figure E56 – Vehicle Class Distribution For Section 291005

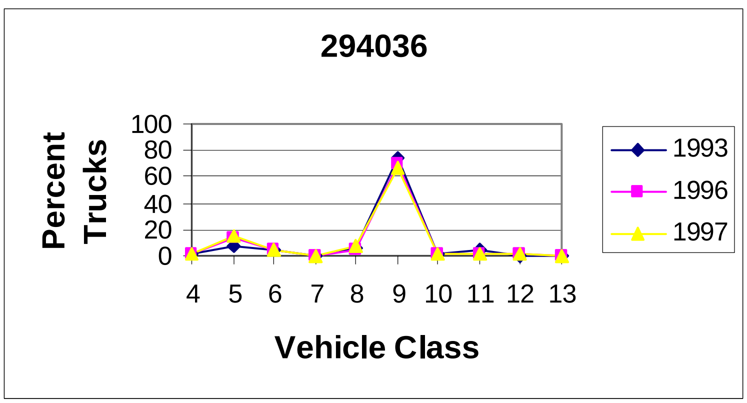


Figure E57 – Vehicle Class Distribution For Section 294036

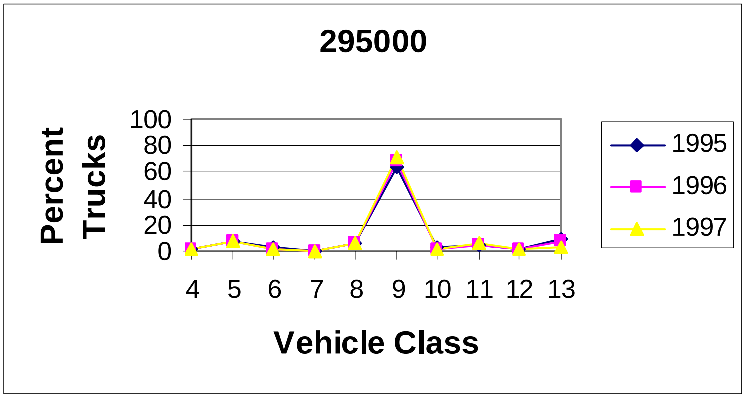


Figure E58 – Vehicle Class Distribution For Section 295000

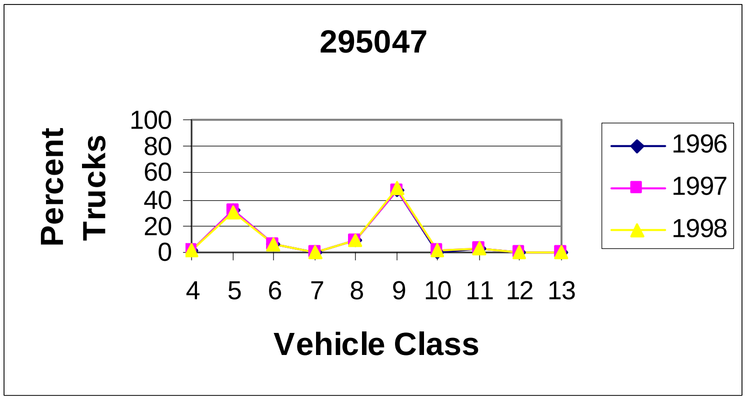


Figure E59 – Vehicle Class Distribution For Section 295047

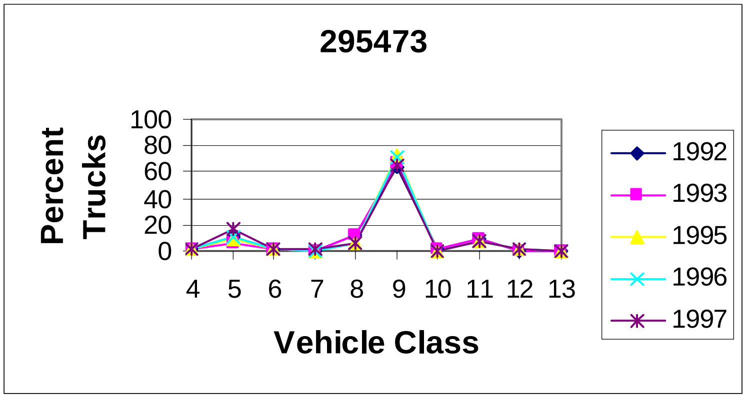


Figure E60 – Vehicle Class Distribution For Section 295473

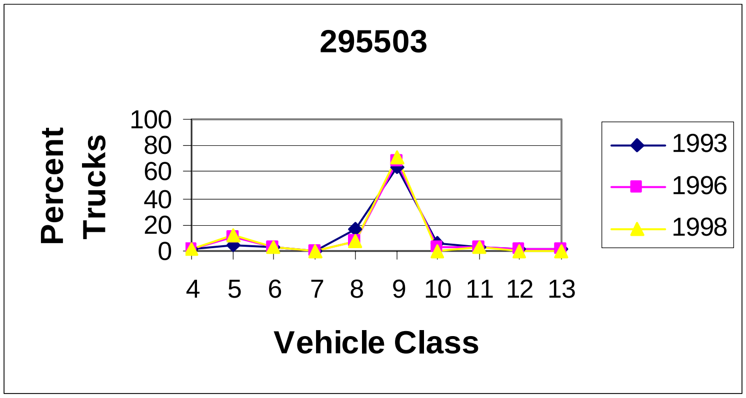


Figure E61 – Vehicle Class Distribution For Section 295503

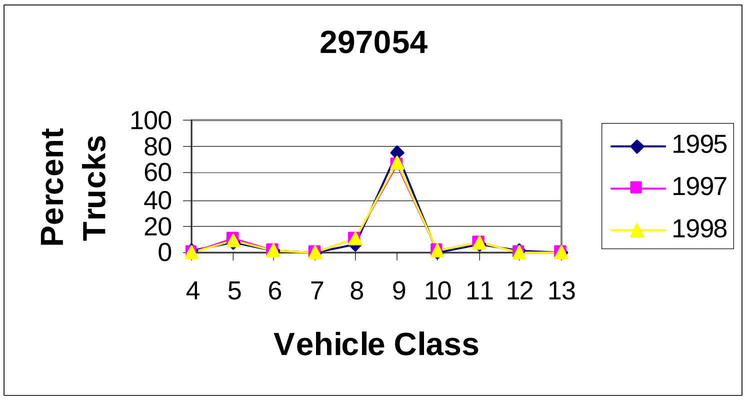


Figure E62 – Vehicle Class Distribution For Section 297054

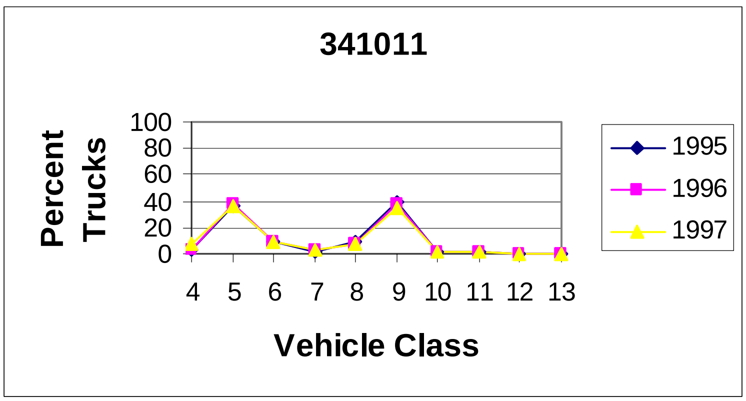


Figure E63 – Vehicle Class Distribution For Section 341011

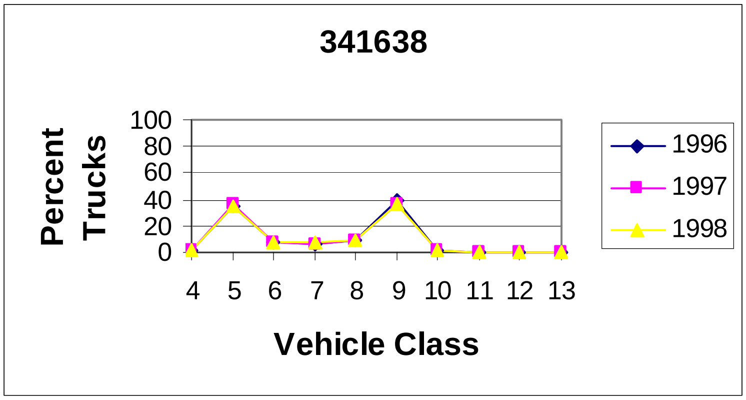


Figure E64 – Vehicle Class Distribution For Section 341011

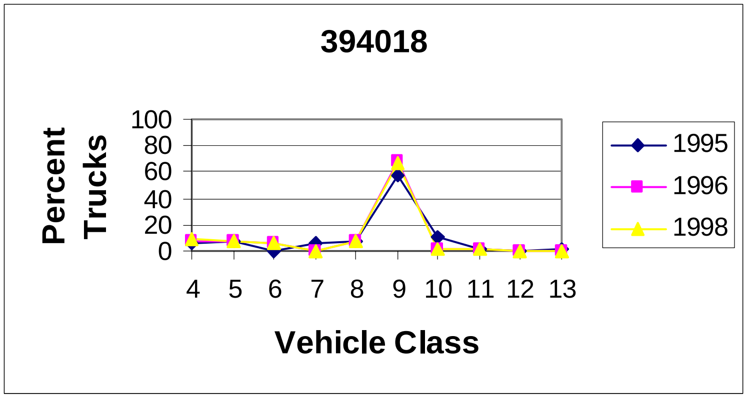


Figure E65 – Vehicle Class Distribution For Section 394018

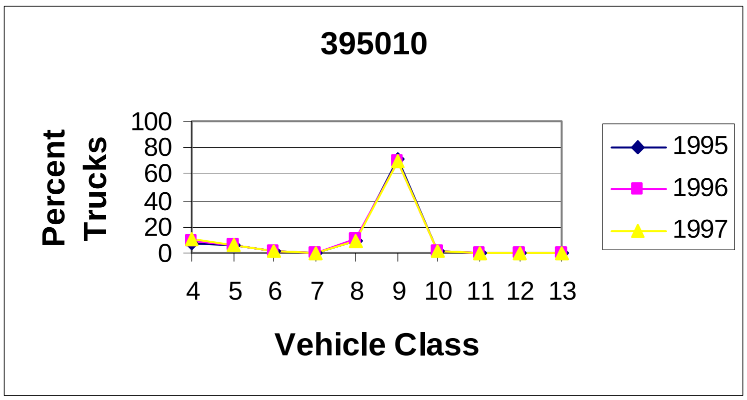


Figure E66 – Vehicle Class Distribution For Section 395010

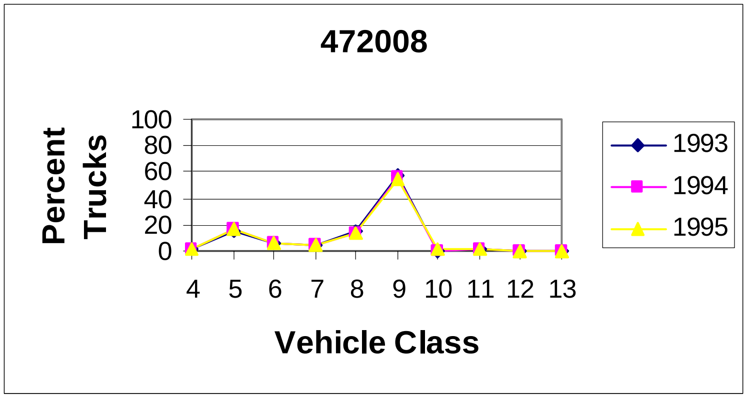


Figure E67 – Vehicle Class Distribution For Section 472008

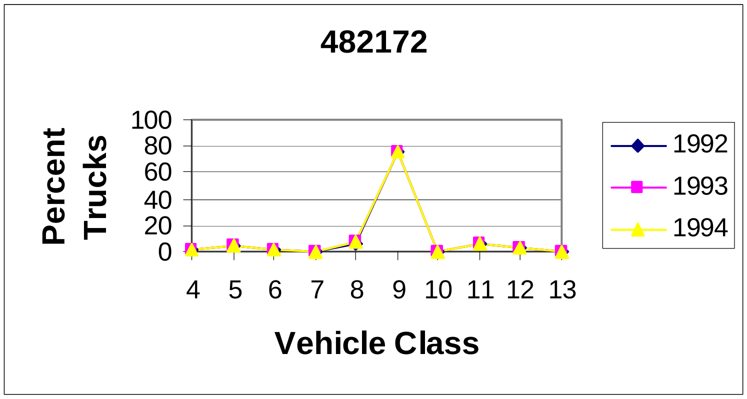


Figure E68 – Vehicle Class Distribution For Section 472008

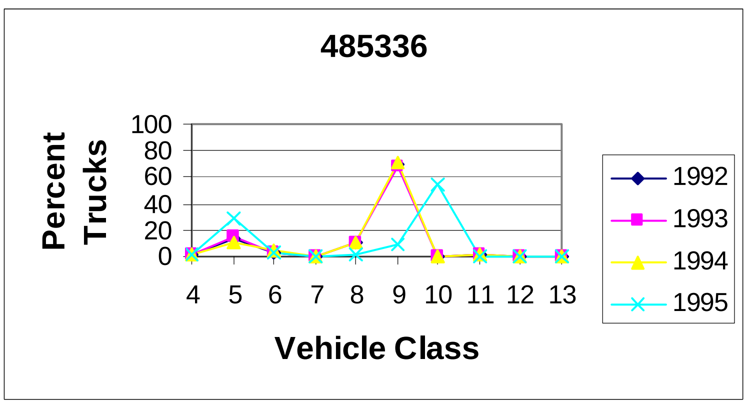


Figure E69 – Vehicle Class Distribution For Section 485336

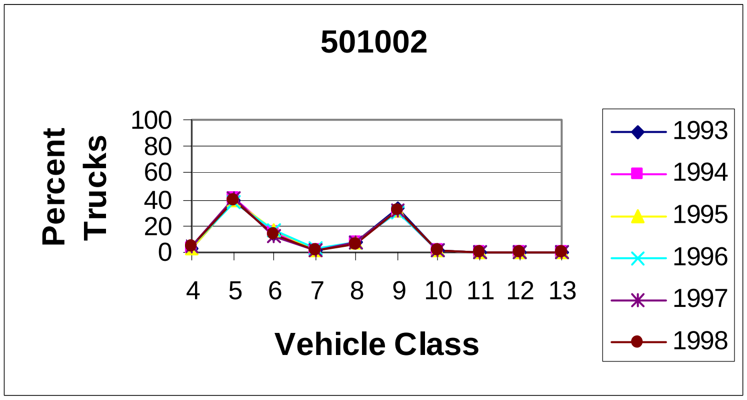


Figure E70 – Vehicle Class Distribution For Section 501002

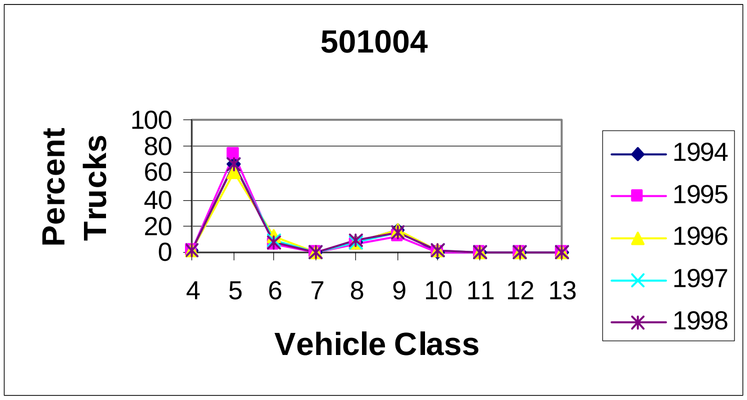


Figure E71 – Vehicle Class Distribution For Section 501004

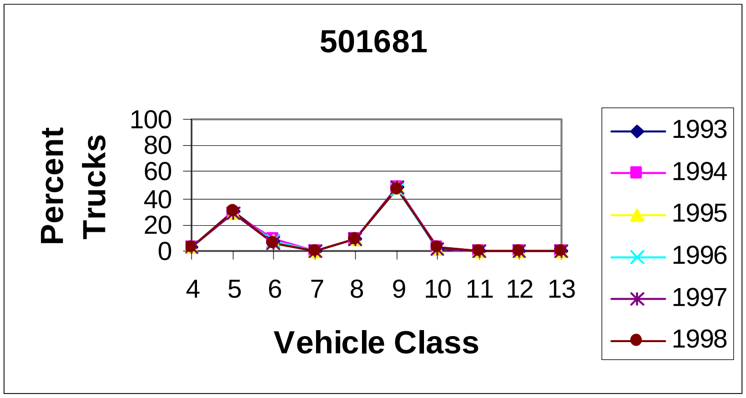


Figure E72 – Vehicle Class Distribution For Section 501681

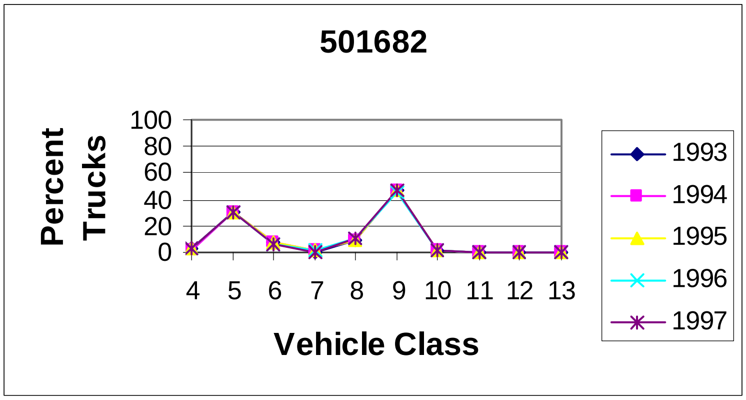


Figure E73 – Vehicle Class Distribution For Section 501682

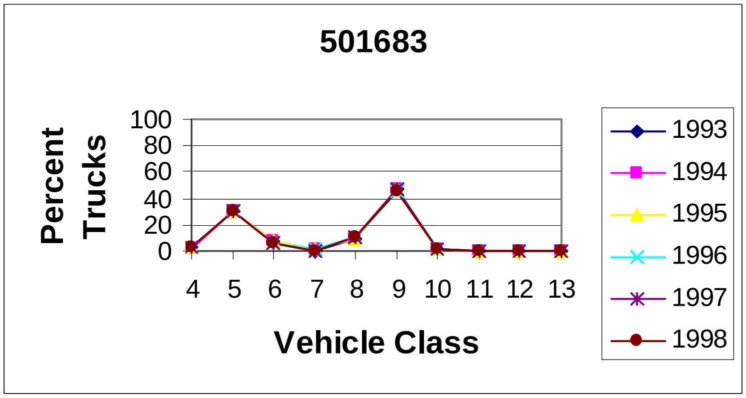


Figure E74 – Vehicle Class Distribution For Section 501683

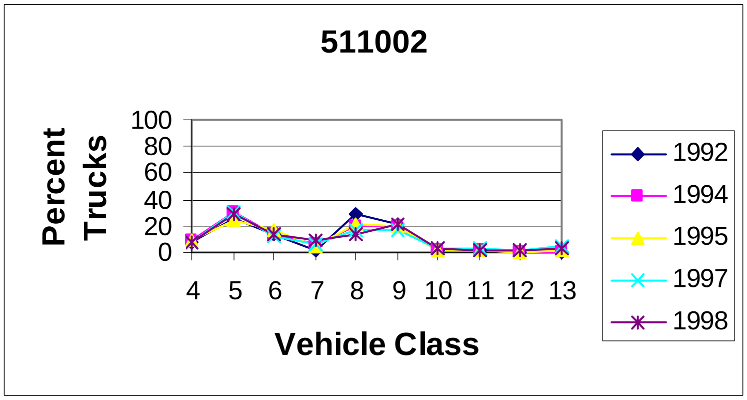


Figure E75 – Vehicle Class Distribution For Section 511002

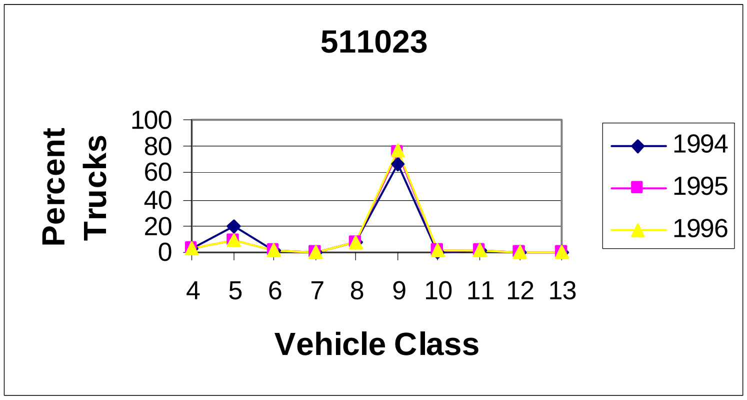


Figure E76 – Vehicle Class Distribution For Section 511023

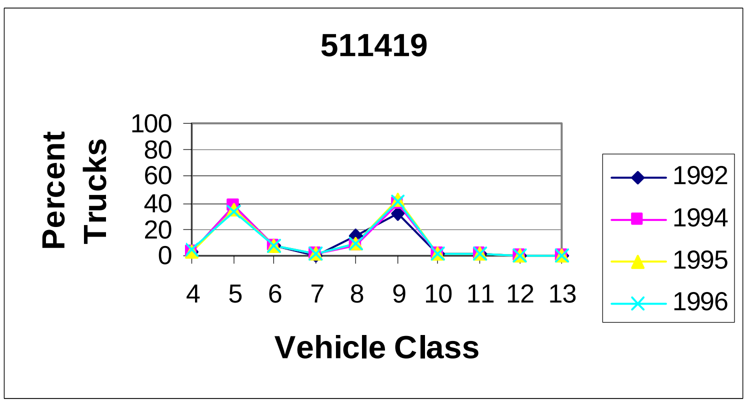


Figure E77 – Vehicle Class Distribution For Section 511419

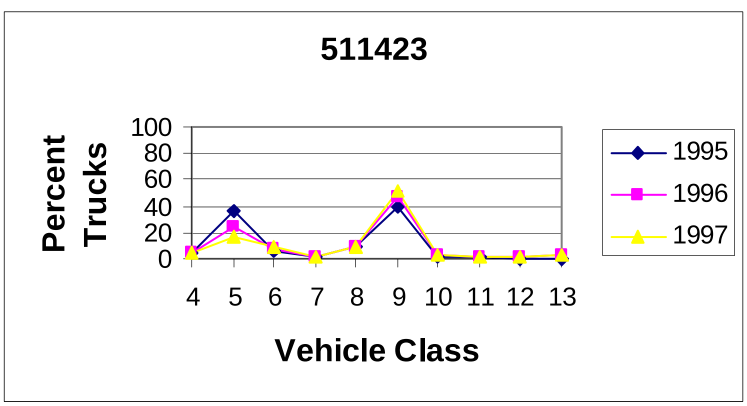


Figure E78 – Vehicle Class Distribution For Section 511423

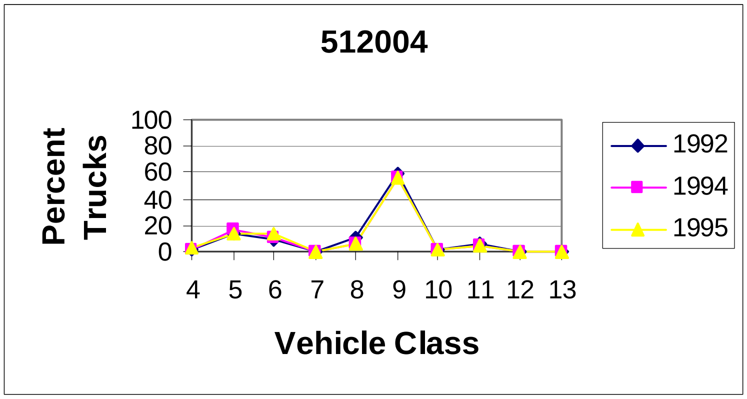


Figure E79 – Vehicle Class Distribution For Section 512004

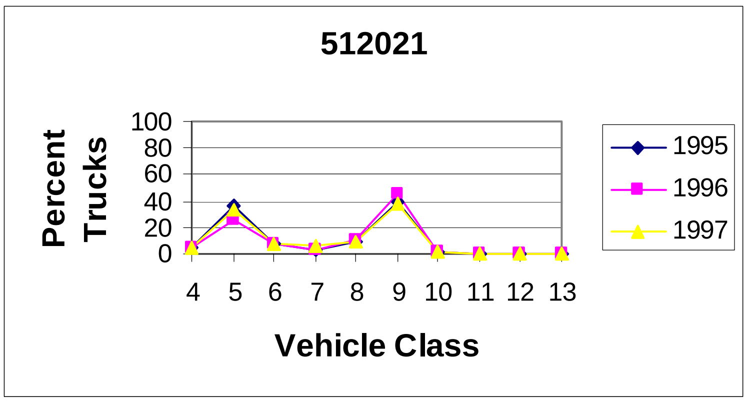


Figure E80 – Vehicle Class Distribution For Section 512021

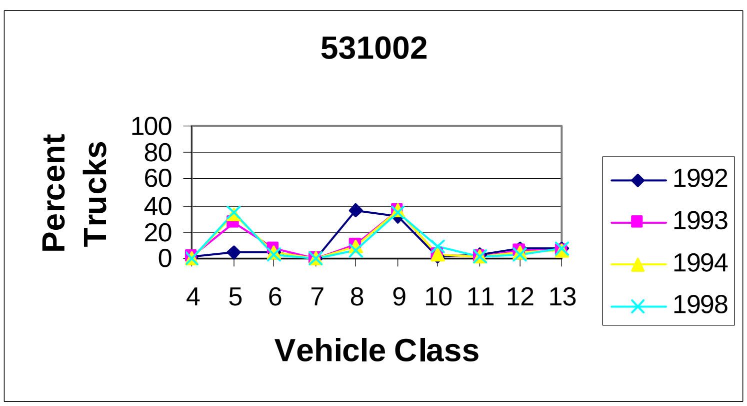


Figure E81 – Vehicle Class Distribution For Section 512021

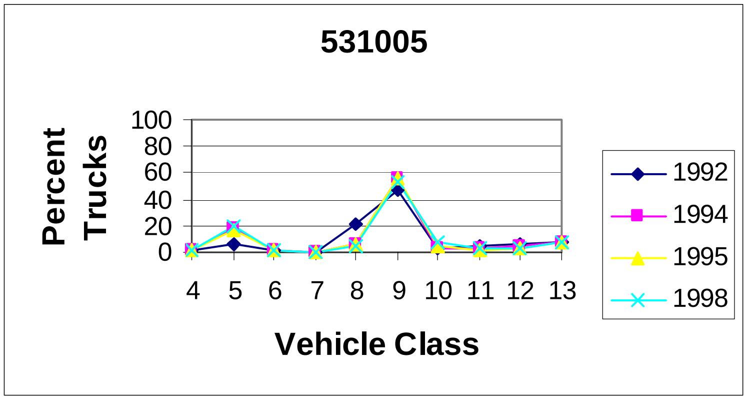


Figure E82 – Vehicle Class Distribution For Section 531005

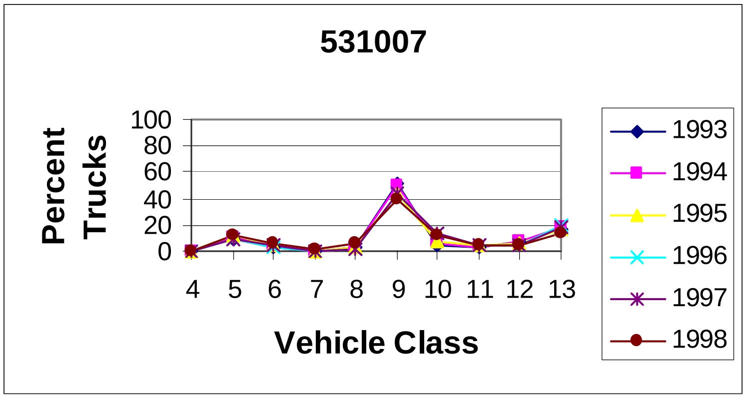


Figure E83 – Vehicle Class Distribution For Section 531007

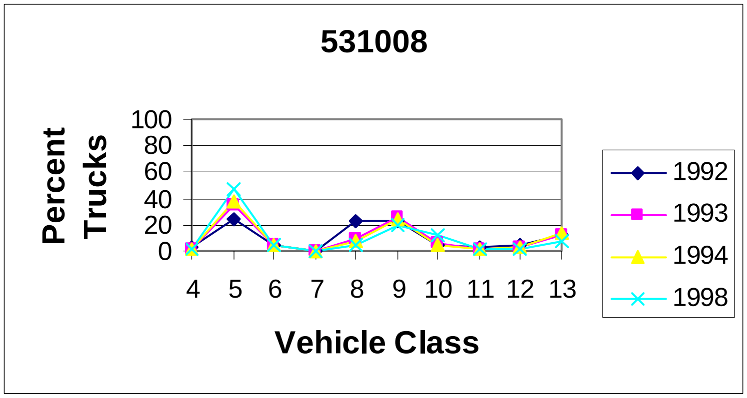


Figure E84 – Vehicle Class Distribution For Section 531008

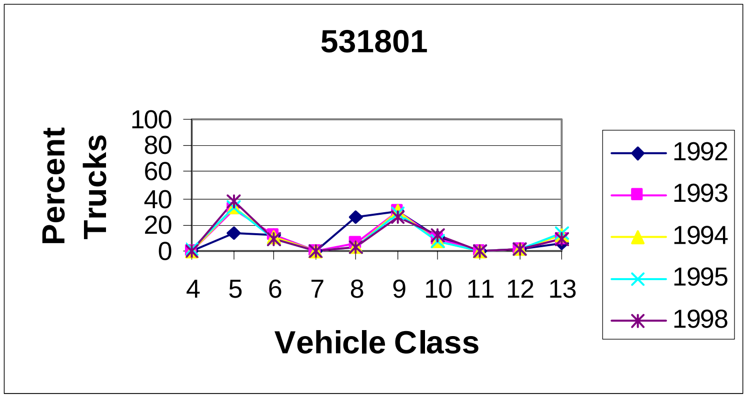


Figure E85 – Vehicle Class Distribution For Section 531801

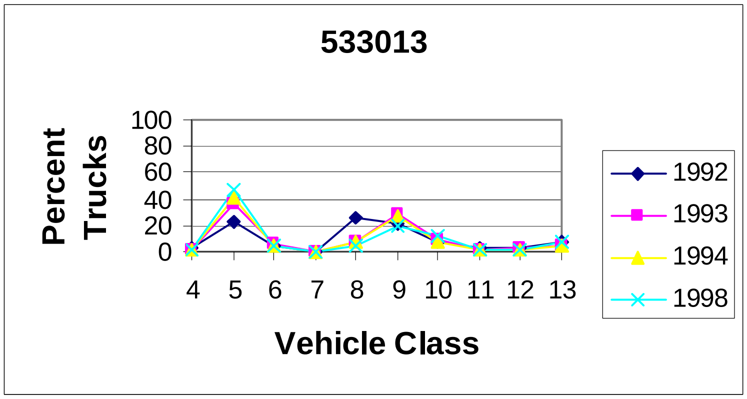


Figure E86 – Vehicle Class Distribution For Section 533013

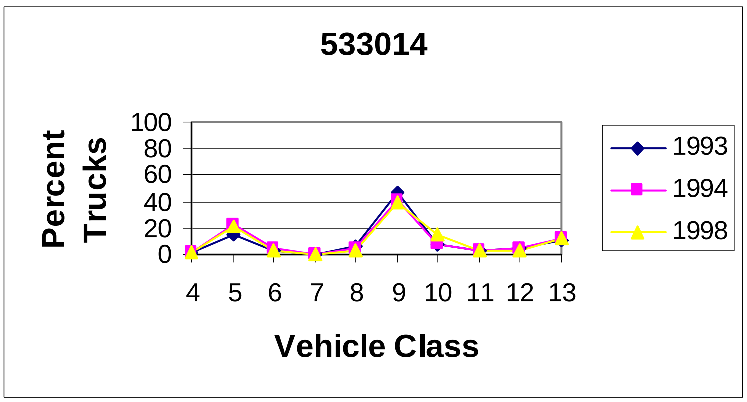


Figure E87 – Vehicle Class Distribution For Section 533014

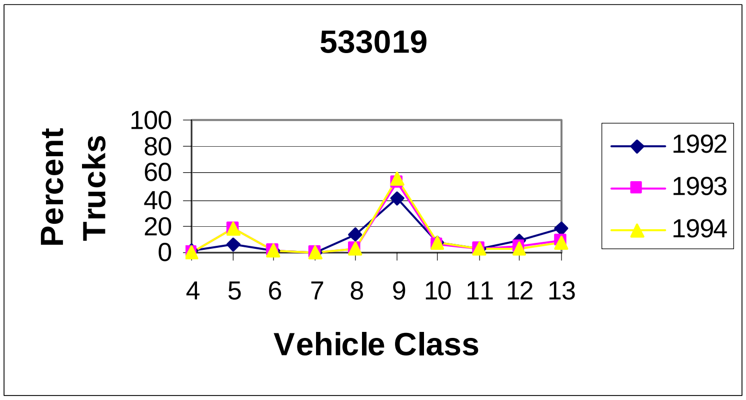


Figure E88 – Vehicle Class Distribution For Section 533019

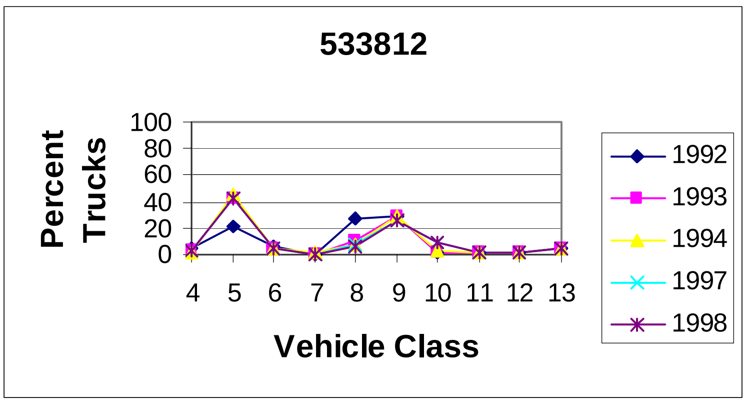


Figure E89 – Vehicle Class Distribution For Section 533812

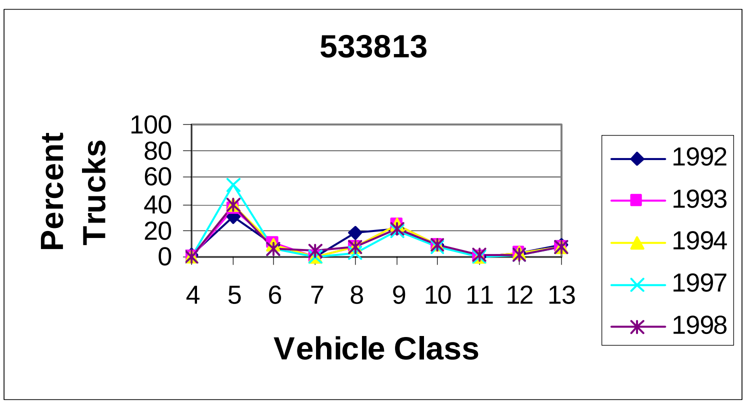


Figure E90 – Vehicle Class Distribution For Section 533813

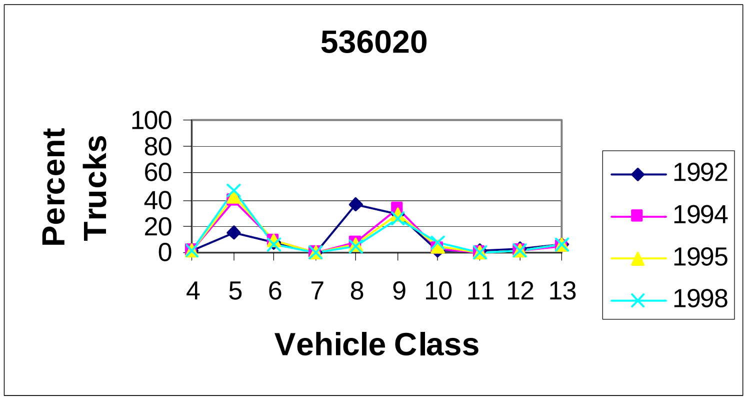


Figure E91 – Vehicle Class Distribution For Section 536020

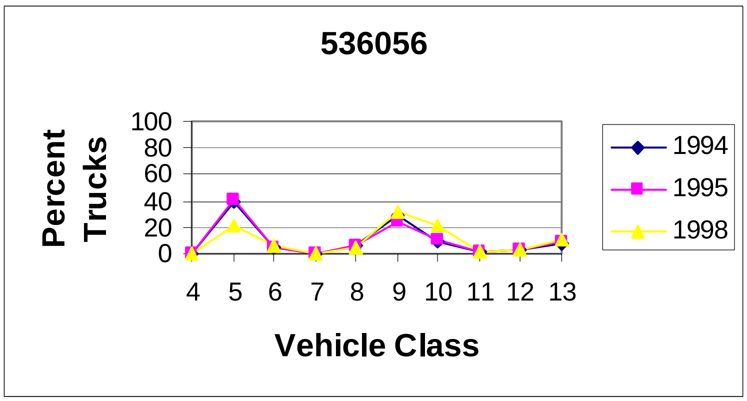


Figure E92 – Vehicle Class Distribution For Section 536056

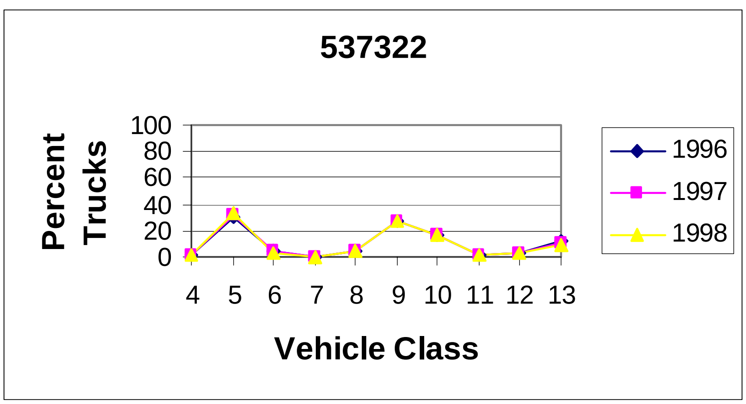


Figure E93 – Vehicle Class Distribution For Section 536056

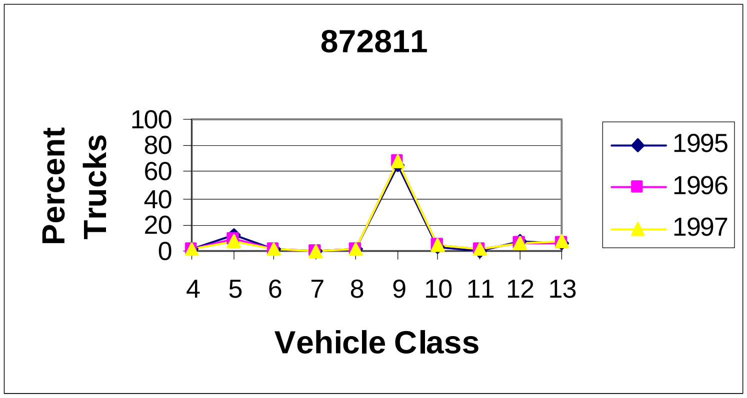
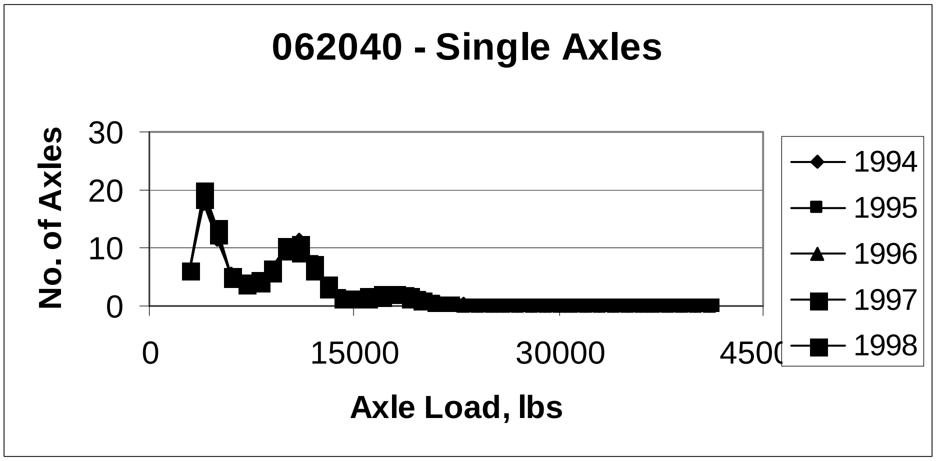
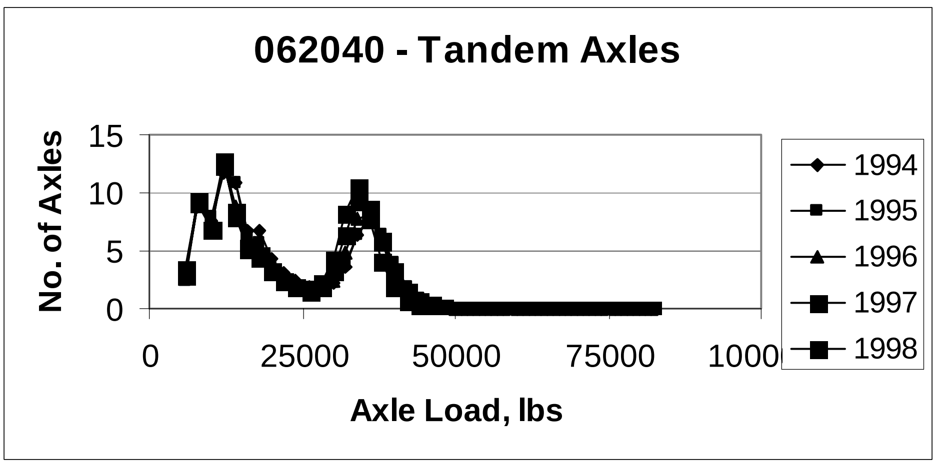


Figure E94 – Vehicle Class Distribution For Section 872811





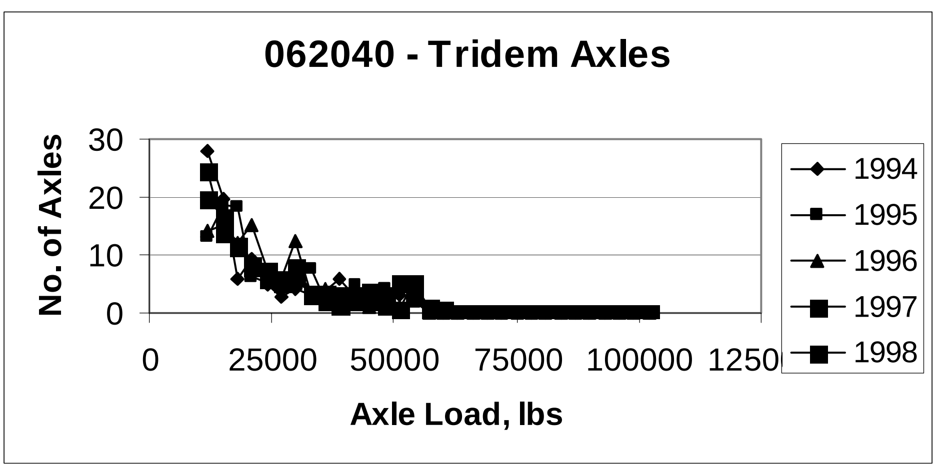
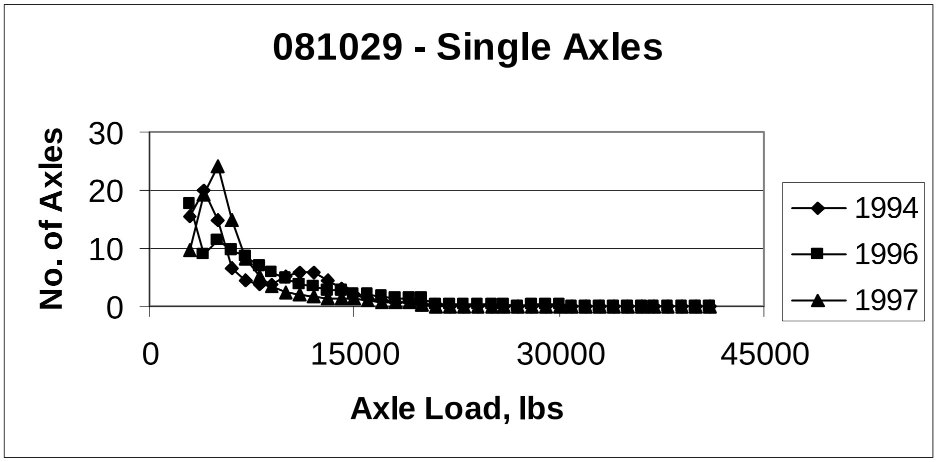
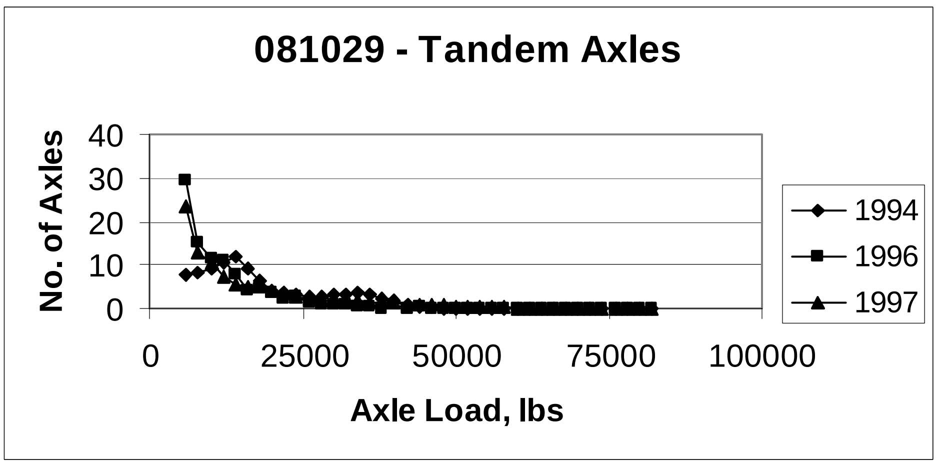


Figure E95 – Distribution of Axle Loads for Section 062040





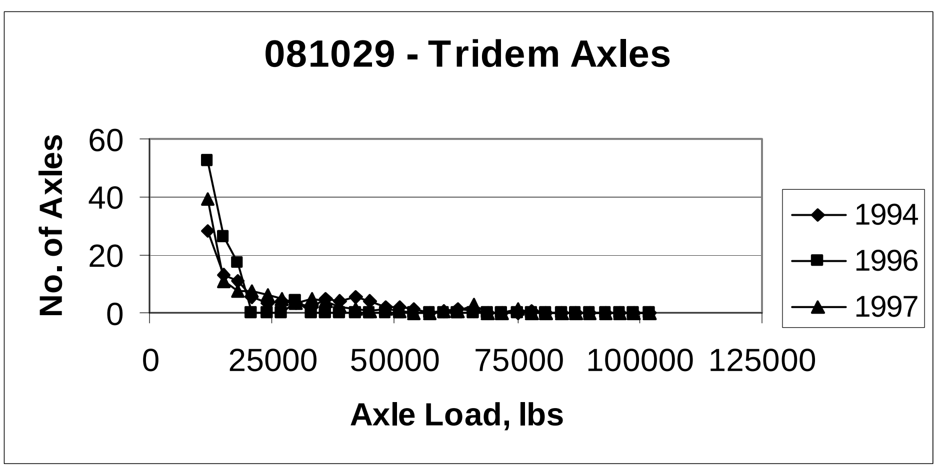
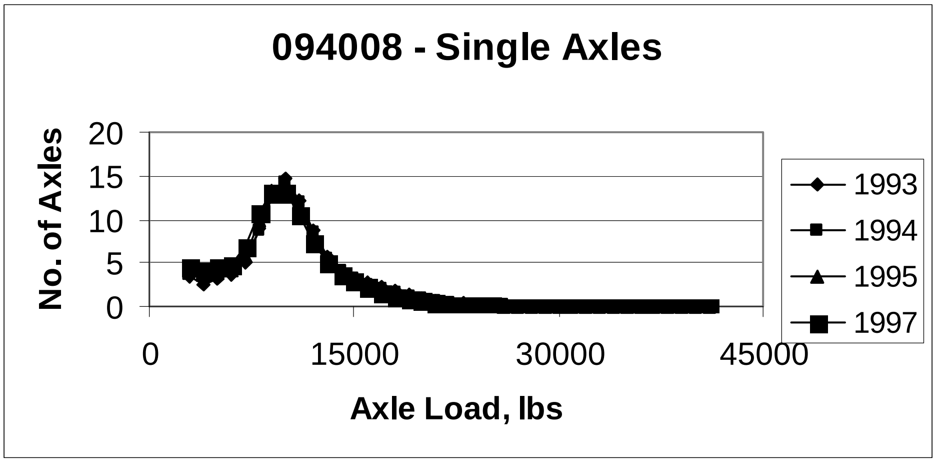
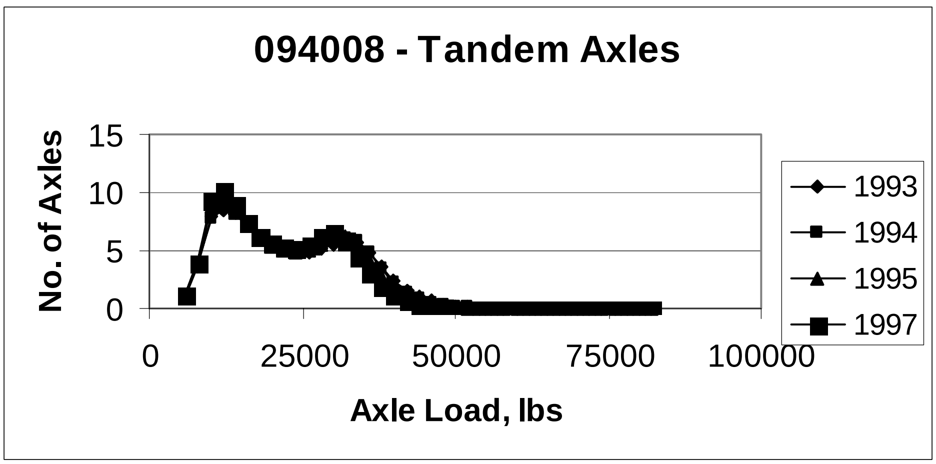


Figure E96 – Distribution of Axle Loads for Section 081029





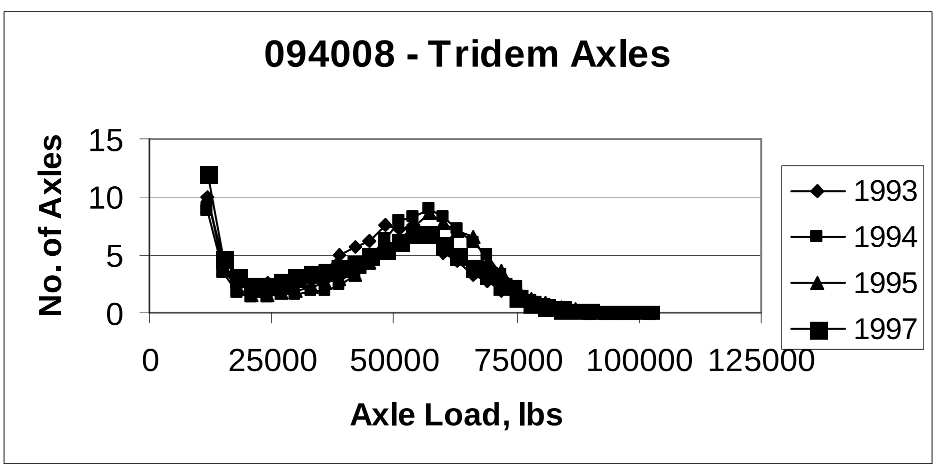
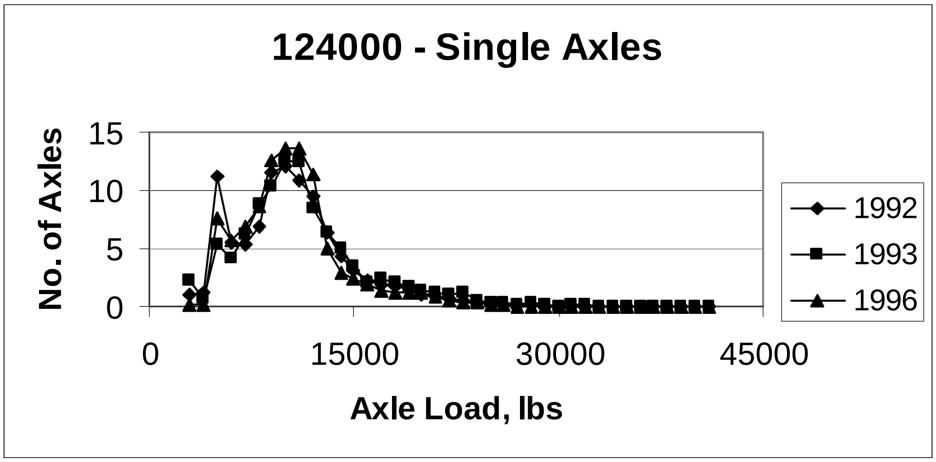
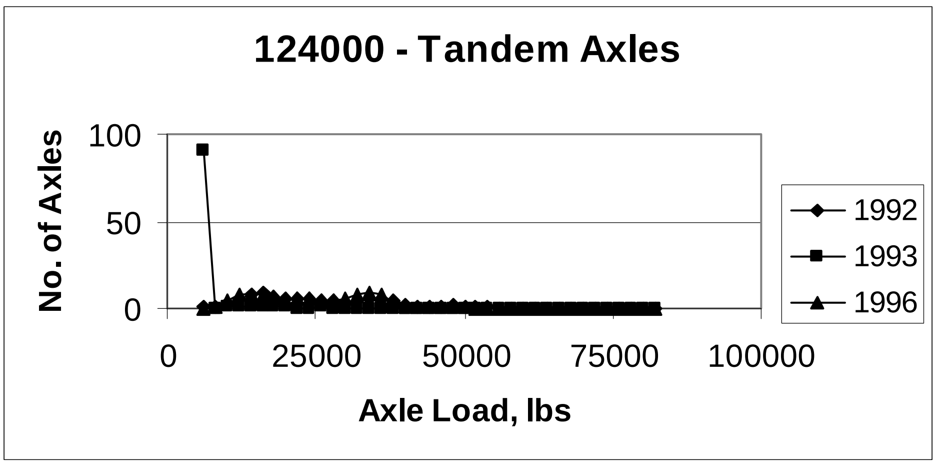


Figure E97 – Distribution of Axle Loads for Section 094008





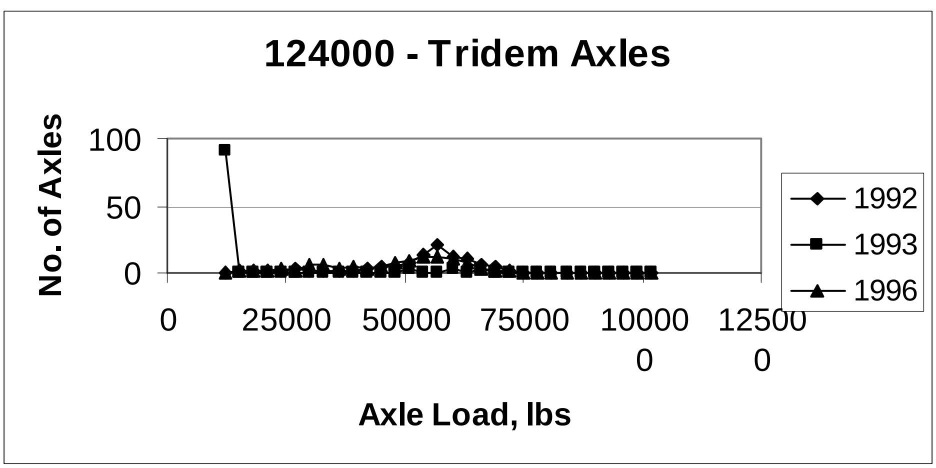
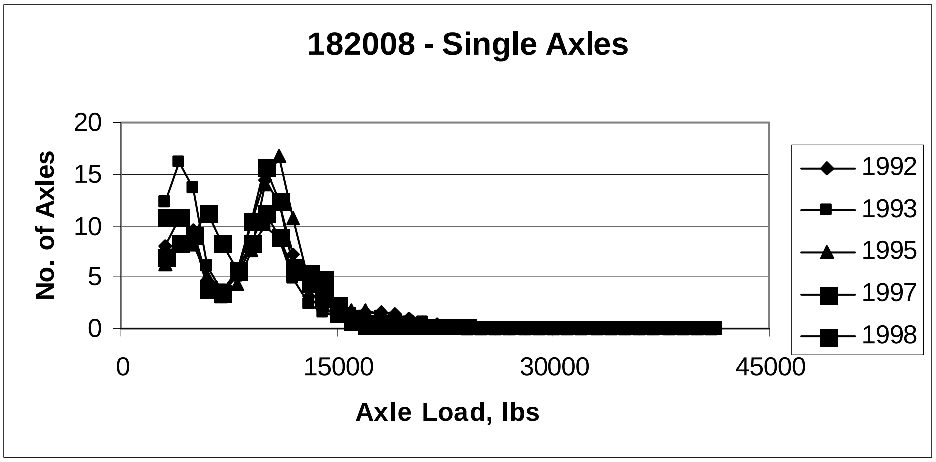
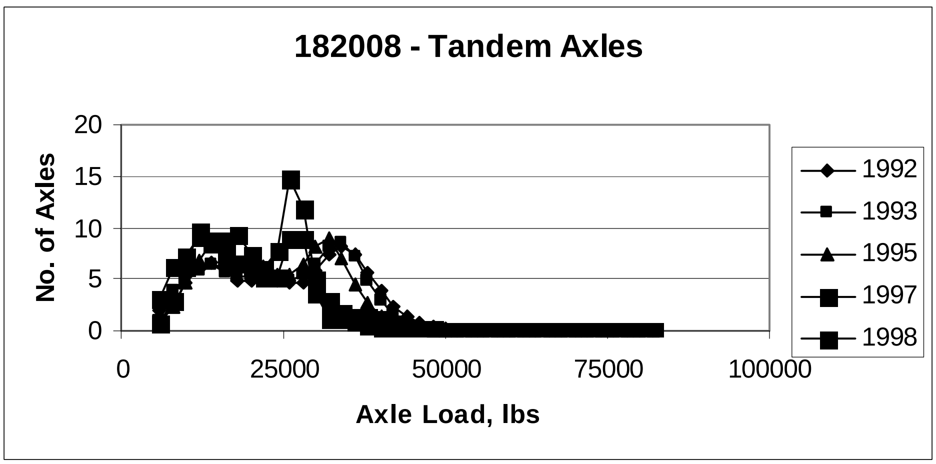


Figure E98 – Distribution of Axle Loads for Section 124000





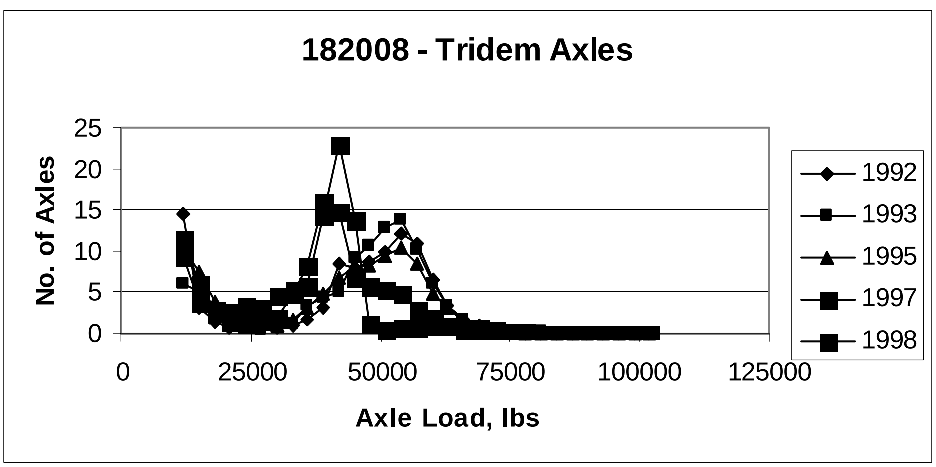
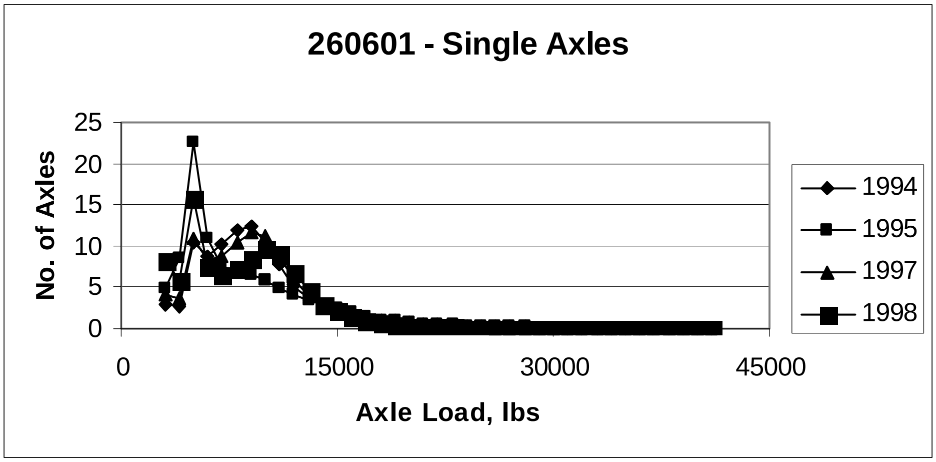
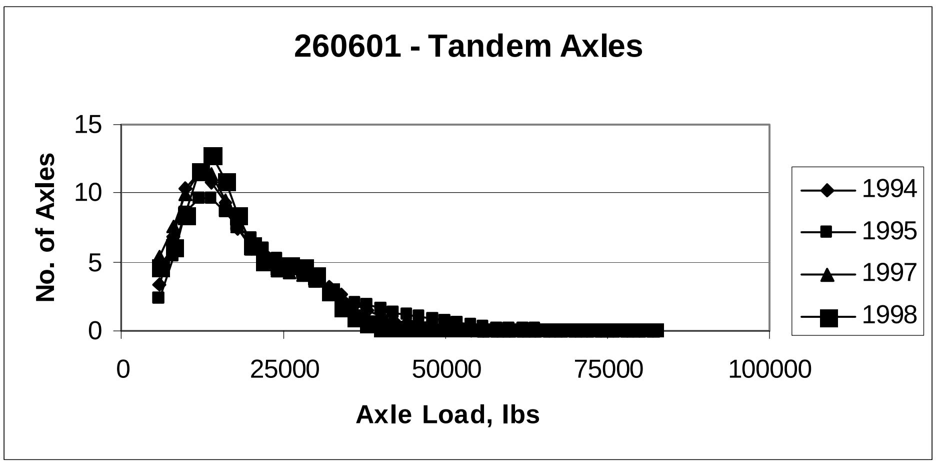


Figure E99 – Distribution of Axle Loads for Section 182008





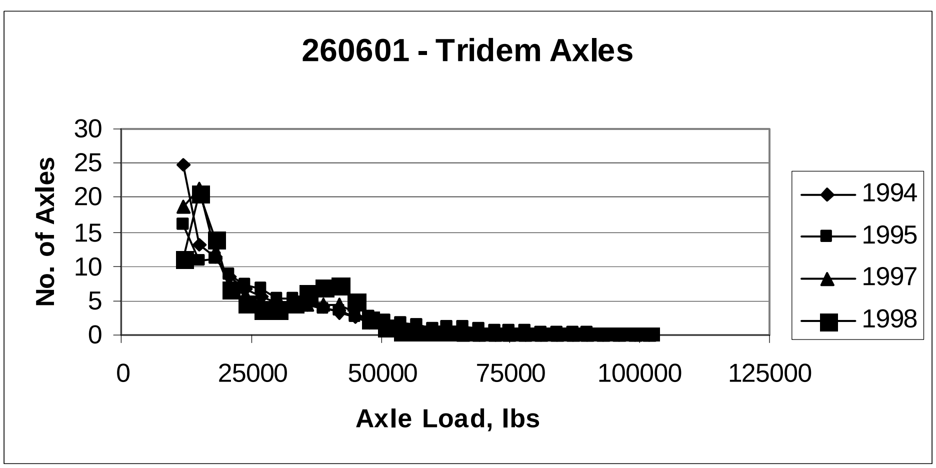
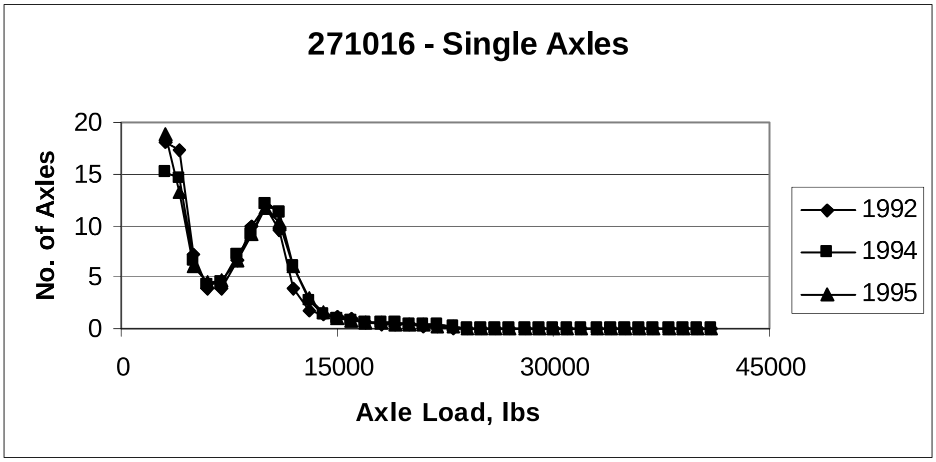
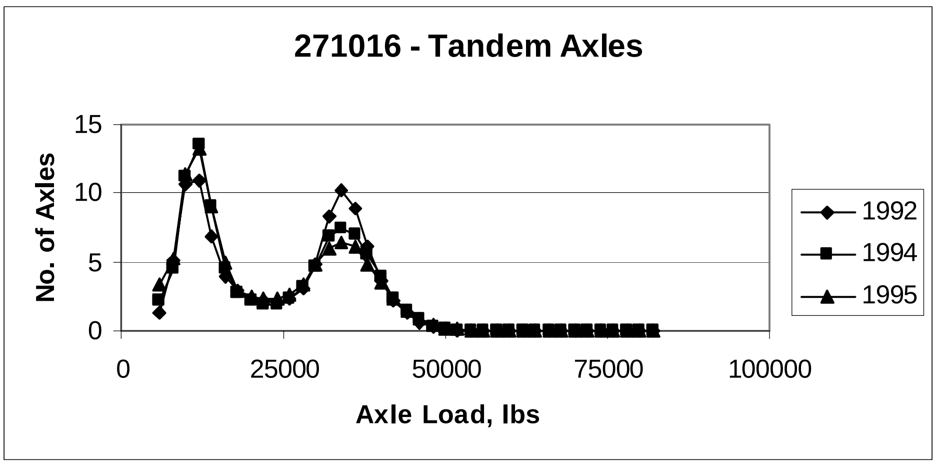


Figure E100 – Distribution of Axle Loads for Section 260601





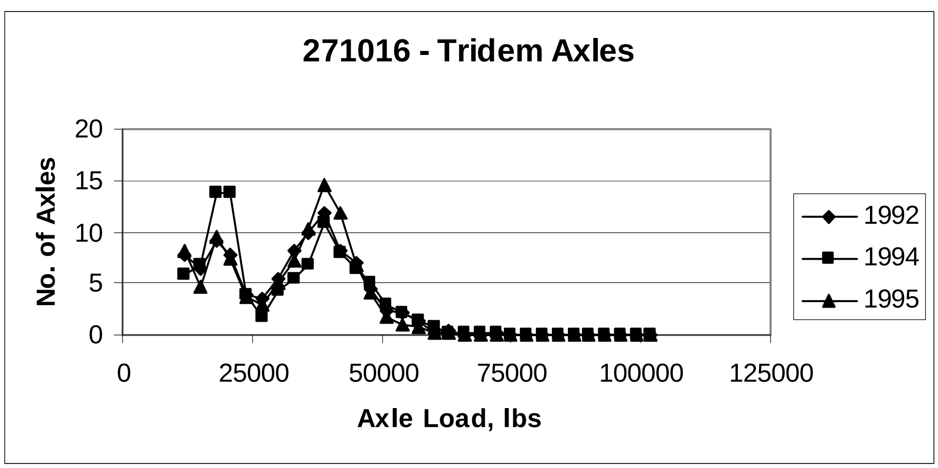
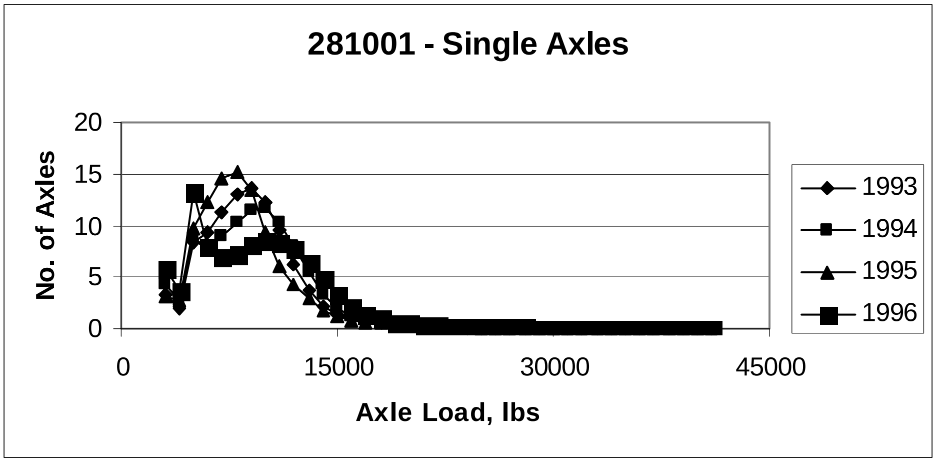
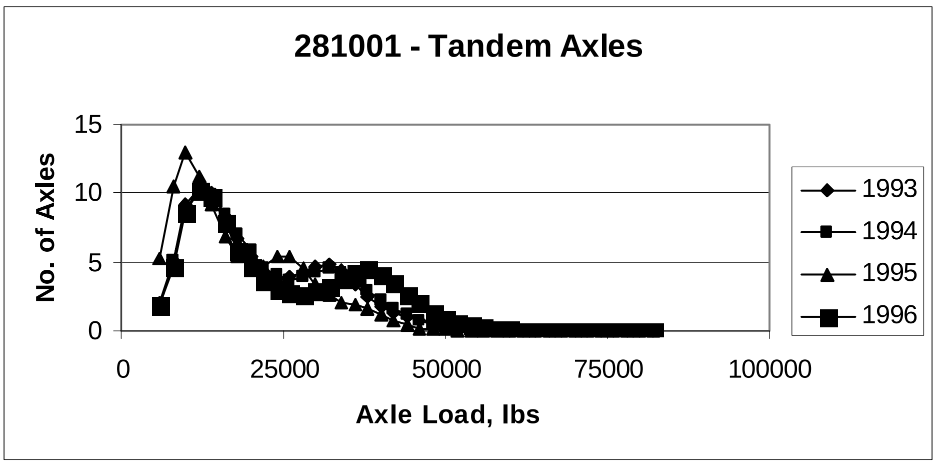


Figure E101 – Distribution of Axle Loads for Section 271016





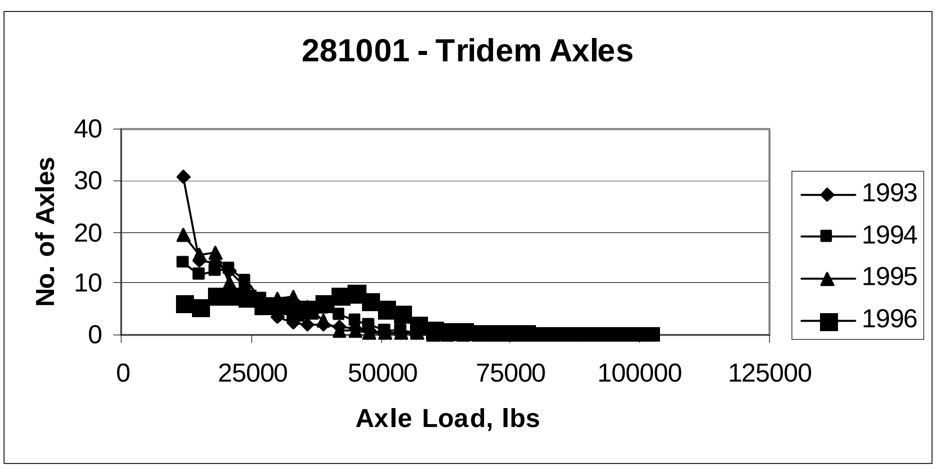
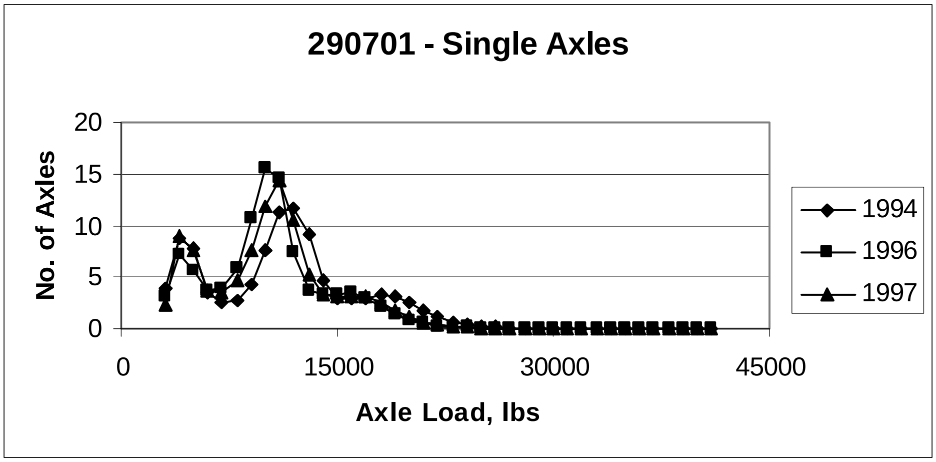
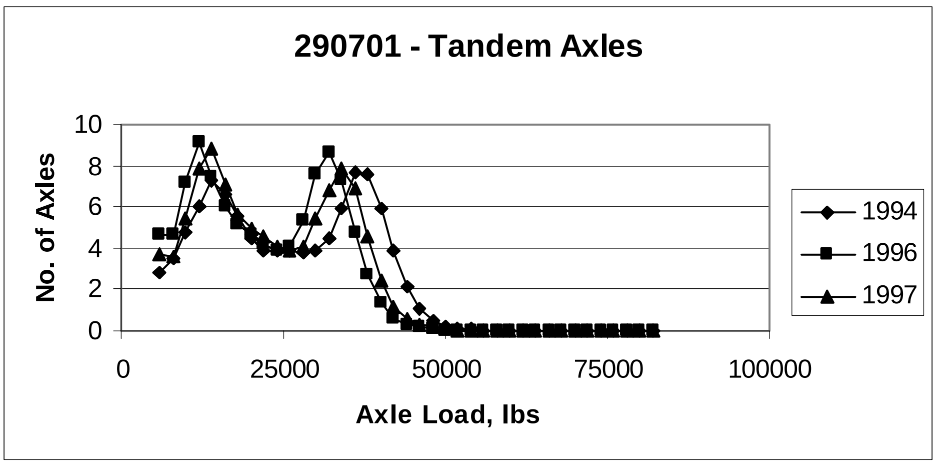


Figure E102 – Distribution of Axle Loads for Section 281001





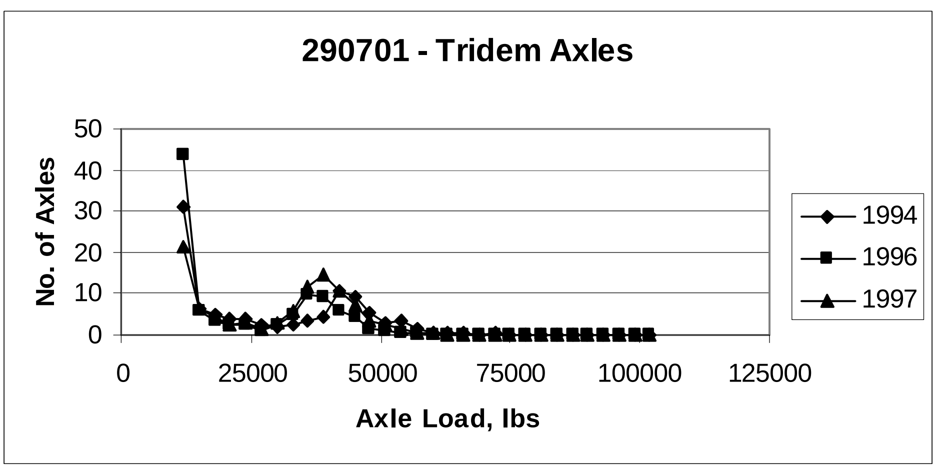
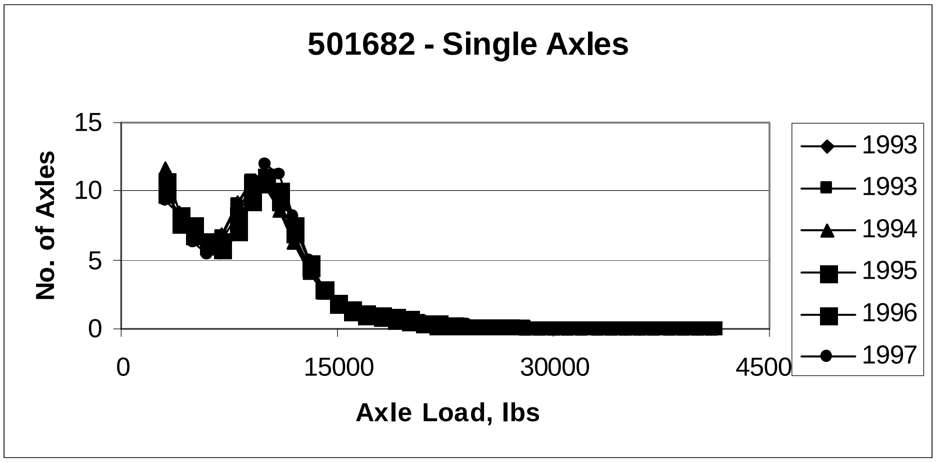
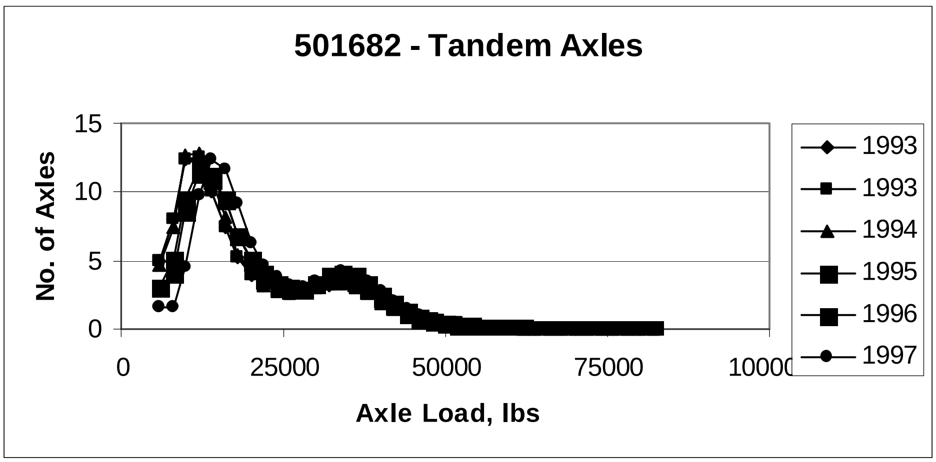


Figure E103 – Distribution of Axle Loads for Section 290701





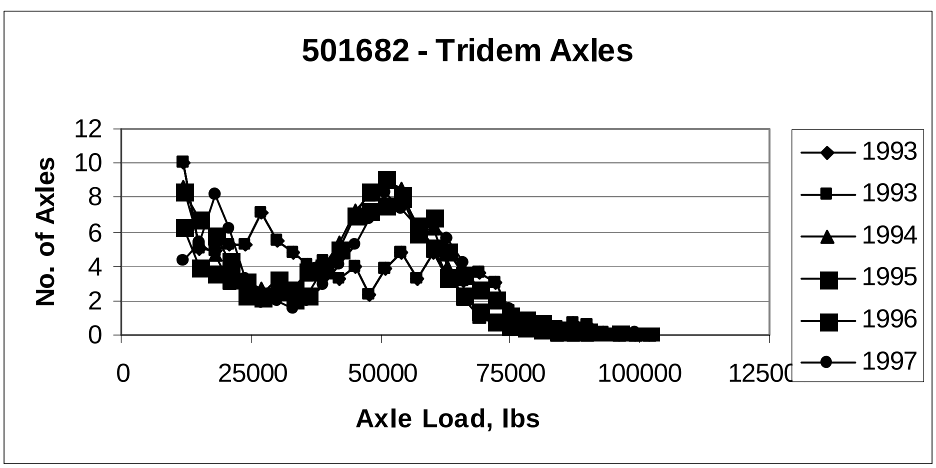


Figure E104 – Distribution of Axle Loads for Section 501682

APPENDIX F

Temperature-Modulus and Spatial Variation of Back-Calculated Moduli

The deflection data from the Seasonal Monitoring Program (SPM) of LTPP was analyzed to determine relationships between back-calculated moduli and the pavement temperature, as reported by Lukanen, Stubstad and Briggs in report FHWA RD 99-085. Each test result for every section visit was back-calculated using the same parameters and back-calculation method. For the purpose of evaluation variations, the analysis results from the sections tested in SMP Round 2 were evaluated. The analysis results used were the station-by-station regression coefficients for the relationship between Log(M_r) and the mid-depth temperature. That equation provides a linearized relationship with the same model that was used to evaluate the laboratory test results.

Variation of the Modulus-Temperature Regression

SMP Round 2 consisted of 342 pavement locations that were tested an average of 26 different times throughout the year. The in-pavement temperatures were measured at half-hour intervals, allowing the temperature of the pavement at the time of test to be interpolated. There are inherent variations in both the deflection measurements, temperature measurements and back-calculation methods used, which in this case was WESDEF. However, these issues are not discussed here. An understanding of the overall repeatability of the process can be obtained by reviewing the standard error of estimate of the regressions for each of the 342 locations. Figure F1 shows the distribution of the standard error of estimate [the SEE is computed for Log(M_r) but these results were converted to an arithmetic plus or minus percentage factor for the plot] for the regression results. The bulk of the results ranged between 10 and 20 percent and the mode is 14 percent.

Spatial Variation of Back-Calculated Moduli

Using the same 15 SMP sections, the regression equation for each station was used to calculate the moduli at 5, 25 and 40°C. The variation of the calculated results would more closely reflect the spatial variation than other data currently available. Figure F2 shows the coefficient of variation of the predicted back-calculated moduli for the three target temperatures described above. It is interesting to see that the spatial variation at the colder temperature of 5°C has a broader range of variations than the corresponding range for the 40°C moduli. The 25°C moduli range of variation falls between these two. Overall, the variations observed with the back-calculated moduli tend to have the opposite behavior as that seen in the laboratory, as described above, where the variations are the smallest for the 5°C test results and largest at the 40°C results. A possible explanation as to why the back-calculated moduli variations are more sensitive to colder temperatures is that the cold asphalt is a more dominant structural component of an asphalt pavement, and anomalies such as cracks or other defects have a more dramatic effect of the back-calculated moduli. The “compensating layer” effect also has an influence on the stability of the back-calculation results. Variations within the pavement of hot-mix asphalt do not play as dominant a role in the overall variations in moduli; apparently any existing cracks or other surface course anomalies do not affect back-calculated moduli results to the same degree.

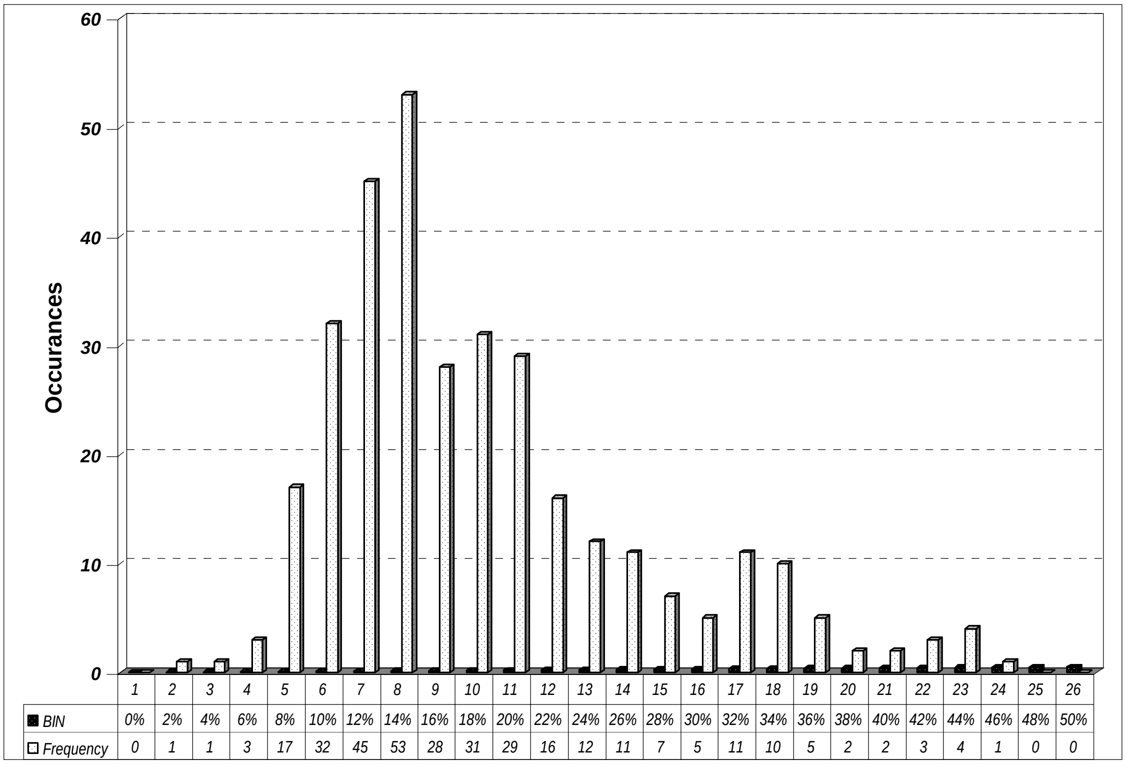


Figure F1. Frequency distribution of regression errors for Back-Calculated Asphalt Modulus vs. Temperature Regressions

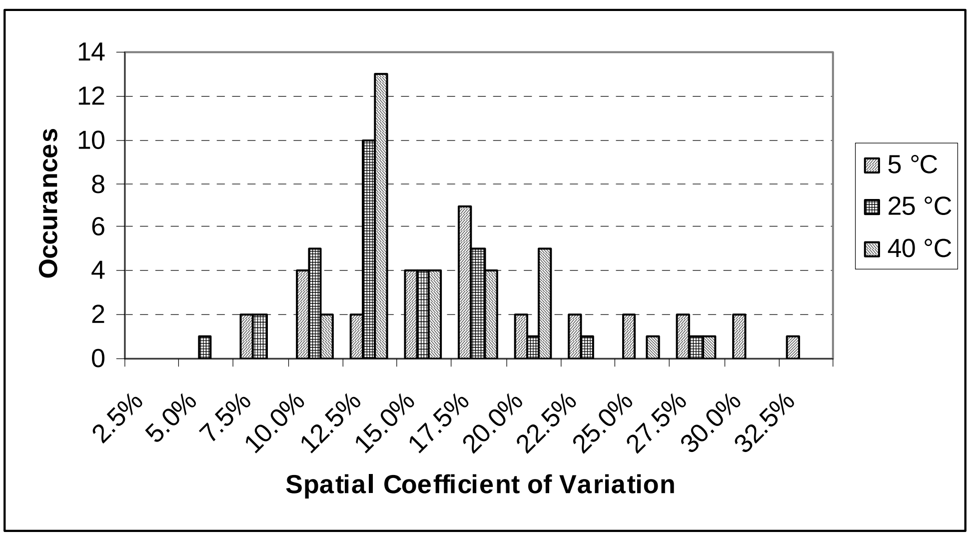


Figure F2. Spatial Variation of the Back-Calculated Asphalt Moduli at the 15 Round 2 SMP Sections.