Report No. FHWA-KS-07-8
FINAL REPORT

ACCELERATED TESTING FOR STUDYING PAVEMENT DESIGN AND PERFORMANCE (FY 2003)

Evaluation of the Chemical Stabilized Subgrade Soil
(CISL Experiment No. 12)

Stefan Romanoschi, Ph.D., P.E.
Paul Lewis
Octavian Dumitru
Sathish Banda
Kansas State University
Manhattan, Kansas

January 2008

KANSAS DEPARTMENT OF TRANSPORTATION

Division of Operations
Bureau of Materials and Research

KANSAS
DEPARTMENT OF TRANSPORTATION

1 Report No. FHWA-KS-07-8	2 Government Accession No.	3 Recipient Catalog No.
4 Title and Subtitle ACCELERATED TESTING FOR STUDYING PAVEMENT DESIGN AND PERFORMANCE (FY 2003) Evaluation of the Chemical Stabilized Subgrade Soil (CISL Experiment No. 12)		5 Report Date January 2008 6 Performing Organization Code
7 Author(s) Stefan Romanoschi, Ph.D., P.E., Paul Lewis, Octavian Dumitru, Sathish Banda		8 Performing Organization Report No.
9 Performing Organization Name and Address Kansas State University Department of Civil Engineering 2118 Fiedler Hall Manhattan, KS 66506-5000		10 Work Unit No. (TRAIS) 11 Contract or Grant No. C1355
12 Sponsoring Agency Name and Address Kansas Department of Transportation Bureau of Materials and Research 700 SW Harrison Street Topeka, Kansas 66603-3745		13 Type of Report and Period Covered Final Report Summer 2002- June 2003 14 Sponsoring Agency Code RE-0328-01

15 Supplementary Notes
For more information write to address in block 9.

16 Abstract

The Midwest States Accelerated Pavement Testing Pooled Fund Program, financed by the highway departments of Missouri, Iowa, Kansas and Nebraska, has supported an accelerated pavement testing (APT) project to compare the performance of stabilized clayey embankment soil when Portland cement, fly ash, lime and a commercial product were used as stabilizing agents. The project aimed to improve the practices related to the design of flexible pavements when the top of the subgrade is improved by chemical stabilization. The experiments were conducted at the Civil Infrastructure Systems Laboratory (CISL) of Kansas State University. The test program consisted of constructing four flexible pavement structures and subjecting them to full-scale accelerated loading test.

The study indicated that cement and lime are the most effective stabilizers for the studied soil. These stabilizers resulted in lower vertical compressive stresses at the top of the subgrade and lower rut depth at the pavement surface than the fly ash-treated soil. After more than two million axle load repetitions, the pavement with cement stabilized embankment soil exhibited much less surface cracking than the pavement with fly-ash stabilized embankment. The commercial product proved not to be effective in stabilizing the non-sulfate clayey soil used in this experiment, when the embankment is constructed at the same moisture content and compaction level as for the other three chemicals. The unconfined compression strength measured on laboratory prepared samples of soil stabilized with the commercial chemical compound was very similar to that of the untreated soil.

17 Key Words Pavement Design, Chemical Stabilized Subgrade Soil		18 Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161	
19 Security Classification (of this report) Unclassified	20 Security Classification (of this page) Unclassified	21 No. of pages 183	22 Price

Form DOT F 1700.7 (8-72)

ACCELERATED TESTING FOR STUDYING PAVEMENT DESIGN AND PERFORMANCE (FY 2003)

Evaluation of the Chemical Stabilized Subgrade Soil (CISL Experiment No. 12)

Final Report

Prepared by

Stefan Romanoschi, Ph.D., P.E.
Paul Lewis
Octavian Dumitru
Sathish Banda
Department of Civil Engineering
Kansas State University
Manhattan, Kansas

A Report on Research Sponsored By

THE KANSAS DEPARTMENT OF TRANSPORTATION
TOPEKA, KANSAS

January 2008

NOTICE

The authors and the state of Kansas do not endorse products or manufacturers. Trade and manufacturer's names appear herein solely because they are considered essential to the object of this report.

This information is available in alternative accessible formats. To obtain an alternative format, contact the Office of Transportation Information, Kansas Department of Transportation, 700 SW Harrison Street, Topeka, Kansas 66603-3745 or phone (785) 296-3585 (Voice) (TDD).

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the views or the policies of the state of Kansas. This report does not constitute a standard, specification or regulation.

ABSTRACT

The Midwest States Accelerated Pavement Testing Pooled Fund Program, financed by the highway departments of Missouri, Iowa, Kansas and Nebraska, has supported an accelerated pavement testing (APT) project to compare the performance of stabilized clayey embankment soil when Portland cement, fly ash, lime and a commercial product were used as stabilizing agents. The project aimed to improve the practices related to the design of flexible pavements when the top of the subgrade is improved by chemical stabilization. The experiments were conducted at the Civil Infrastructure Systems Laboratory (CISL) of Kansas State University. The test program consisted of constructing four flexible pavement structures and subjecting them to full-scale accelerated loading test.

The study indicated that cement and lime are the most effective stabilizers for the studied soil. These stabilizers resulted in lower vertical compressive stresses at the top of the subgrade and lower rut depth at the pavement surface than the fly ash-treated soil. After more than two million axle load repetitions, the pavement with cement stabilized embankment soil exhibited much less surface cracking than the pavement with fly-ash stabilized embankment. The commercial product proved not to be effective in stabilizing the non-sulfate clayey soil used in this experiment, when the embankment is constructed at the same moisture content and compaction level as for the other three chemicals. The unconfined compression strength measured on laboratory prepared samples of soil stabilized with the commercial chemical compound was very similar to that of the untreated soil.

ACKNOWLEDGEMENTS

The research project was selected, designed and monitored by the members of the Midwest States Accelerated Pavement Testing Pooled Fund Technical Committee. The committee includes Mr. Andy Gisi, Kansas Department of Transportation (KDOT), Chair, Mr. George Woolstrum, Nebraska Department of Roads (NDOR), Mr. John Donahue, Missouri Department of Transportation (MDOT), and Mr. Mark Dunn, Iowa Department of Transportation (IADOT). The authors acknowledge the cooperation and supervision of all committee members in this study. Contributions of Mr. Sagar Bethu and Mr. Cristian Dumitru in various phases of this study are also gratefully acknowledged. The research team is also grateful to Sergeant Joe French from the Motor Vehicle Inspection Division of the Kansas Highway Patrol, for his help in the measurement and calibration of the ATL axle load.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER ONE - INTRODUCTION AND BACKGROUND

1.1 Report Organization

This manuscript is the final report that describes the research project conducted under KDOT Contract C1355, “Midwest Accelerated Testing Pooled Fund – FY 2003”, (KSU Research Project No. 5-34367). This contract is funded by the Midwest States Accelerated Pavement Testing Pooled Fund Program. States participating in this program are Iowa, Kansas, Missouri and Nebraska.

The purpose of the project is to conduct the experiment selected by the Midwest States Accelerated Testing Pooled Funds Technical Committee for the Fiscal Year 2002 (FY-02). The experiment titled “Evaluation of Chemical Stabilized Subgrade Soil“ is the twelfth experiment conducted at the Civil Infrastructures Systems Lab (CISL), formerly known as the Accelerated Testing Lab (ATL), and is, therefore, now identified as CISL-Exp#12. The first two ATL experiments, ATL-Exp#1 and #2 were reported in reference [1], ATL-Exp#3 through #6 were reported in reference [2], ATL-Exp#7 is reported in reference [3], ATL-Exp#8 is reported in reference [4], CISL Exp #9 and 10 are reported in reference [5] and CISL Exp #11 is reported in reference [6].

This report describes the following aspects of CISL-Experiment #12:

1. A description of the experiment: This includes the experiment objectives, test setup and testing strategies followed.
2. The material and methods used for pavement construction and the pavement response monitoring instrumentation.

3. The experimental work performed in terms of the total number of load cycles applied to each specimen, testing conditions (load magnitude, temperature, etc.), and the time schedule.
4. A summary of the data collected: the results from response monitoring instrumentation and the distresses measured at the pavement surface and, the evolution of the response and distress data with the number of load cycles applied.
5. A comparison of the measured horizontal tensile strains and the vertical stresses and the corresponding stresses and strains estimated with a linear-elastic pavement structure model.
6. The conclusions drawn from the results obtained and performance observed.
7. Recommendations to the participating highway agencies for practical implementation and future experiments.

1.2 Project Overview

The goal of this research was to evaluate the performance of four chemicals when used as stabilizers for the clayey embankment soil underneath asphalt pavements. The objective was accomplished by conducting full-scale accelerated pavement tests at the Civil Infrastructure Systems Laboratory on four flexible pavement sections for which the top six inches of the clayey embankment soil were stabilized with cement, fly-ash, lime and EMC-squared.

The work described in this report examines the experimental aspects of the research study. This mainly entails the application of full-scale axle loads on full-scale flexible pavements. The experimental work was conducted at the Civil Infrastructure

Systems laboratory (CISL) of Kansas State University. The work also includes monitoring and recording deflection, strain, soil pressure, and temperature in the pavement structures tested. Mechanistic responses were calculated and compared with the observed data. The Falling Weight Deflectometer (FWD) deflection data were used to characterize the pavement layers.

This experimental investigation, together with the observed performance of similar situations on in-service highways and supplemented with additional analytical studies, can help the state highway agencies establish special provisions/standards for the use of the four chemicals as stabilizers for the clayey embankment soil underneath asphalt pavements. It may also lead to standard guidelines for instrumentation of in-service highway pavements in the states participating in the Pooled Fund Program. Further work could include numerical modeling, and comparative studies with other research in the United States and abroad.

1.3 Chemical Stabilization of Embankment Soils

Chemical soil stabilization always involves treatment of the soil with some type of chemical compound, which when added to the soil, would result in “chemical reaction.” The chemical reaction modifies/enhances the physical and engineering properties of a soil, such as, volume stability and strength. Chemical stabilization has proven to be an effective technique for improving the engineering properties of subgrade soils in four Midwestern states- Iowa, Kansas, Nebraska and Missouri. Portland cement, quick lime and Class C fly ash are the most common stabilizers used for non-sulfate, clayey subgrade soils.

Chemical stabilization of subgrade soils is extensively used in the four states partly to control volume change of soils and provide all-weather paving platforms. A typical flexible pavement in Kansas will have the top six inches of embankment soil mixed in place with hydrated lime, normally added as slurry. A thick asphalt concrete layer is then placed on top of the stabilized soil. Although many flexible pavement projects in the Midwestern states have clayey subgrade soils stabilized with Portland cement, fly ash and lime, no study has been done to date to compare the performance of these stabilizers for the same soil.

The Midwest States Accelerated Pavement Testing Pooled Fund Program, financed by the highway departments of Missouri, Iowa, Kansas and Nebraska, has supported an accelerated pavement testing (APT) project to compare the performance of stabilized clayey embankment soil when Portland cement, fly ash, lime and a commercial product were used as stabilizing agents. The project aimed to determine if the lime stabilization, the most common method used in Kansas for the chemical stabilization of embankment soils, is the best optimum method. The experiments were conducted at the Civil Infrastructure Systems Laboratory (CISL) of Kansas State University. The test program consisted of constructing four full-scale pavement structures and subjecting them to full-scale accelerated loading.

A fourth commercial stabilizing product was also included in the study. Several studies recommended this commercial stabilizer as effective in the stabilization of sulfate soils but its effectiveness on stabilizing non-sulfate bearing soil has not been proven.

CHAPTER TWO - LABORATORY STUDY FOR THE DESIGN OF STABILIZED SOIL

Before the construction of the CISL pavement sections, a preliminary study was conducted to determine the properties of the untreated and treated soil and to determine the optimum contents for the four stabilizers. The preliminary study was conducted on the soil used in the construction of CISL pavement sections and on two additional soils, which were classified as A-6 and A-4 under the AASHTO soil classification system. A fifth stabilizer, Cement Kiln Dust (CKD) was also included in the preliminary laboratory work. The detailed description of the work conducted in the preliminary laboratory study, as well as a review of literature related to design and field performance of chemically stabilized soil mixture, are given by Banda [7].

2.1 Materials

2.1.1 Chemical stabilizers

Four stabilizers were used in this APT study: cement, fly ash, lime and EMC SQUARED® Stabilizer (1000), a concentrated non-ionic liquid stabilizer. The cement used was commercially available Type I Portland cement. Lime was obtained from Mississippi Lime Company in Illinois. Lime was classified as quick lime. The chemical and physical properties of lime provided by the manufacturer are shown in Tables 2.1 and 2.2, respectively.

Table 2.1: Chemical Properties of the Quick Lime

CaO Total	97.5%
CaO – Available	94.5
CO_2	0.5
Acid Insolubles	0.5
$CaSO_4$	0.08
S-Equivalent	0.02
SiO_2	0.8
Al_2O_3	0.11
Fe_2O_3	0.08
MgO	1.0
LOI	0.6
P_2O_5	0.015
MnO	0.0015

Table 2.2: Physical Properties of the Quick Lime

Specific Gravity	3.3
pH	12.4
BET Surface Area	2.0 m^2/g
Apparent Dry Bulk Density-Loose	50 lbs./$ft.^3$
Apparent Dry Bulk Density-	60 lbs./$ft.^3$

The fly-ash was obtained from Fly Ash Management Inc., Topeka, Kansas. The manufacturer classified the provided fly ash as a class C fly ash. The chemical and physical properties of the fly ash were provided by the manufacturer and are given in Tables 2.3 and 2.4, respectively. The EMC SQUARED® Stabilizer (1000) is a concentrated liquid stabilizer manufactured by Soil Stabilization Products Company, Inc. in California. The physical characteristics of this chemical are shown in Table 2.5.

Table 2.3: Chemical Properties of Class C Fly Ash

Chemical Analysis	Results	ASTM C-618 SPEC.F/C
Silicon Dioxide, SiO_2 (%)	29.10	-
Aluminum Oxide, Al_2O_3 (%)	19.61	-
Iron Oxide, Fe_2O_3 (%)	5.76	-
Sum of SiO_2, Al_2O_3 &	54.47	70/50 Min
Calcium Oxide, CaO (%)	28.14	-
Magnesium Oxide, MgO	8.17	-
Sodium Oxide, Na_2O (%)	2.39	-
Potassium Oxide, K_2O (%)	0.38	-
Sulfur Trioxide, SO_3 (%)	3.02	5.0 Max
Moisture Content, (%)	0.16	3.0 Max
Loss on Ignition (%)	0.11	6.0 Max

Table 2.4: Physical Properties of Class C Fly Ash

Physical Analysis	Results	ASTM C-618 SPEC.F/C
Amount Retained on No.325 Sieve, %	12.4	34 Max
Strength Activity Index Portland Cement @ 7 days, % of Portland Cement @ 28 days, % of	 108 109	 75 Min 75 Min
Water Requirement, % of Control	93	105 Max
Autoclave Expansion, %	+0.07	0.8 Max
Specific Gravity	2.77	
Increase of Drying Shrinkage, %	-0.00	0.03 Max
Reactivity of Cement Alkalies, %		-
Reduction of Mortar Expansion, %		
Mortar Expansion, % of LA Cement	95	100 Max
Air Entrainment of Mortar, %	0.028	-

Table 2.5: Physical Characteristics of EMC SQUARED® Stabilizer [8]

Phase	Liquid
Specific Gravity	1.14 – 1.18
Flash Point	None
Flash Point (dehydrated)	Above 300° F (149°C)
pH Range	7.0 ± 2
Buffer Capacity	None
Abrasiveness	None
Freezing Point	32° F (0°C)
Corrosive Characteristics	None
Inorganic Alkali Equivalent	None
Petroleum Solvents	None
Solvents (Organic)	None
Detergents	None

2.1.2 Subgrade soil

The soil used in the construction of the four experimental pavement sections at the CISL laboratory was obtained from Bayer Construction Co. in Manhattan, Kansas. This type of soil is commonly used for the construction of embankment layers underneath flexible and rigid pavements in the four Mid-West States.

The soil was a clay soil with 96% passing the No.200 sieve. The soil was classified as A-7-6 according to AASHTO soil classification system [9]. The properties of the untreated soil are given in Table 3.6.

Table 2.6: Properties of the Untreated Soil

Percent passing No. 200 sieve (%)	96
Liquid Limit	44
Plastic Limit	19
Plasticity Index	25
MDD (Standard Proctor) (Kg/m^3) / [pcf]	1553 / [96.9]
OMC (%) (Standard Proctor)	23
MDD (Kg/m^3) (Modified Proctor) / [pcf]	1750 / [109.2]
OMC (%) (Modified Proctor)	14

2.1.2.1 Sieve Analysis of the Untreated Soil

A sample of soil weighing 500 grams was dried to constant mass and washed through No.200 (0.075 mm opening) sieve. The portion of the soil retained on the sieve was then collected into a bin and dried in the oven to determine the mass of the soil retained on the No.200 sieve. The percentage of the soil passing No.200 sieve was then determined as:

% Passing #200 = 100 – [(mass retained on # 200 sieve)/500]*100

The percentage of soil passing No.200 sieve obtained for the studied soil was 96%.

2.1.2.2 Atterberg Limit Tests of the Untreated Soil

The liquid limit, plastic limit, and plasticity index of the soil was determined according to ASTM D 4318-00 [10]. Liquid limit tests were performed according to the multiple point method. The sample for the test was prepared by thoroughly mixing the portion of the air-dried soil passing No.40 sieve with water. Using a spatula, a portion of the sample was spread to form an approximately horizontal surface in the brass cup of the liquid limit device to a depth of about 10mm at its deepest point. A groove was formed in the prepared soil surface through the line joining the highest point to the lowest point on the rim of the cup by drawing the grooving tool perpendicular to the surface of the cup throughout its movement. The two halves of the soil on the either side of the grove were then allowed to flow together by dropping the cup through a height of 10 mm using the crank of liquid limit device. The number of the drops (N) required to close the groove along a distance of 13mm (0.5 inch) was recorded. The procedure was repeated by mixing different water contents with the soil to obtain three values of N: one between 25 and 35 drops, one between 20 and 30 drops and one between 15 to 25 drops. A graph was drawn between number of drops and water content. It was found that the water content corresponding to 25 drops, which is recorded as the Liquid Limit (LL), was 44.

The sample for the Plastic Limit was prepared by mixing soil with water, sufficiently to allow the soil mass to roll on a glass plate without sticking to the hands. A portion of the sample (2 grams) was then formed into an ellipsoidal mass and rolled on a glass plate under the pressure of the fingers that was sufficient to roll the mass into a thread of uniform diameter throughout its length. The rolling was continued until the diameter of the thread was 3.2 mm. The thread was then broken into several pieces and formed into ellipsoidal mass and rolled again following the same procedure. The procedure was continued until the thread crumbled and was not possible to be pressed and rolled into 3.2 mm diameter. The water content of the soil mass at this stage was reported as the Plastic Limit of the soil. The Plastic Limit for the soil was found to be 19.

2.1.2.3 Moisture-Density Tests

The Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) of the soil were determined according to ASTM D 698-00 [10]. The portion of the soil passing

No.4 sieve was thoroughly mixed with different water contents and allowed to stand for 6 hours by placing it in the trays covered with flat plates to ensure the uniformity of the soil-water mix. The mix was then placed in the standard Proctor mold and compacted in three layers of equal thickness. Each layer was compacted by dropping the standard Proctor rammer for 25 times and was scarified before placing next layer to ensure good bonding between layers. After compaction, the wet density of the mix was determined from the weight of the mix in the mold, moisture content of the mix, and volume of the mold. Dry density was determined from the values of wet density and moisture content. The process was repeated for several water contents.

The variation of dry density with water content is shown in Figure 2.1. The higher dry density obtained from the graph is recorded as MDD. The moisture content corresponding to MDD is considered as OMC.

The Moisture-Density relation was also determined for the modified Proctor test according to ASTM D 1557 [10]. The results of this test are shown in Figure 2.2.

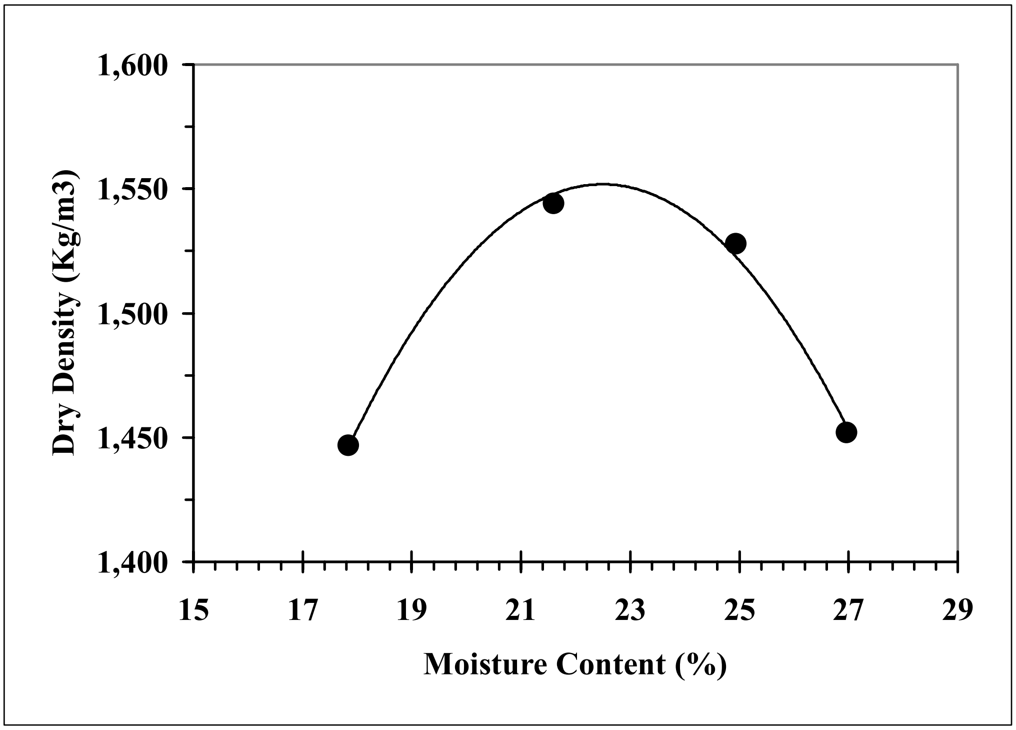


Figure 2.1: Results of Standard Proctor tests conducted on the Untreated Soil

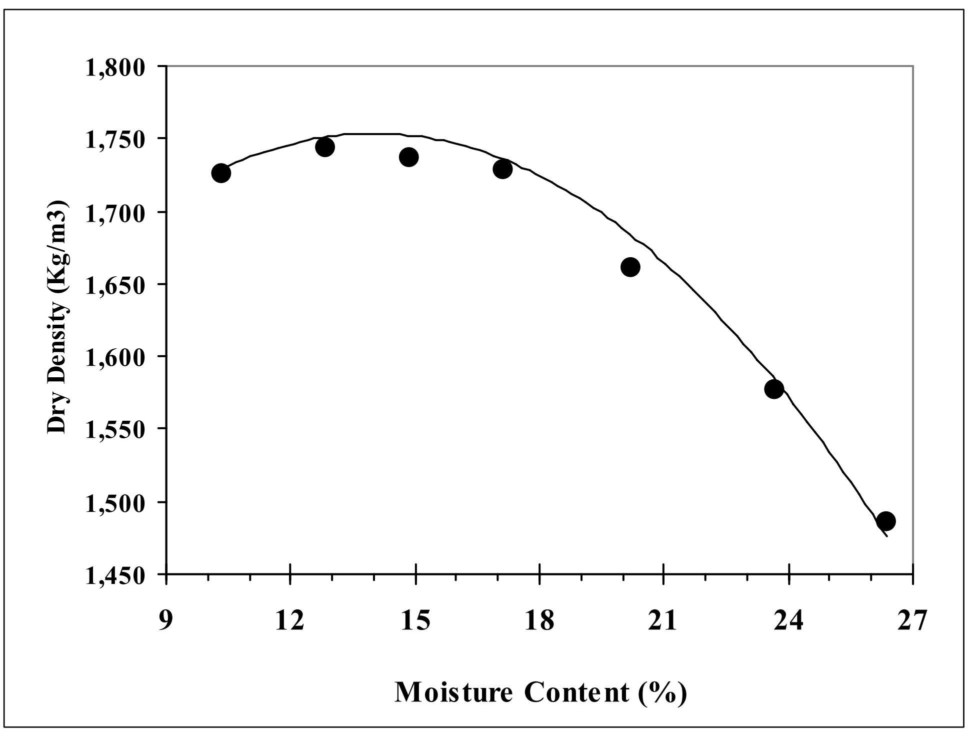


Figure 2.2: Results of Modified-Proctor test conducted on the Untreated Soil

2.2 Selection of Trial Contents for Stabilizers

Cement: The amount of cement required to stabilize a soil depends on soil type and desired strength and durability. Generally, the quantity of the cement required to stabilize a soil increases as the clay content of the soil increases. Minimum Unconfined Compressive Strength (UCS) of 2.8 MN/m^2 for 7-day curing at constant temperature of 25^0C has been widely used as strength criteria [11]. The cement content requirements recommended by Portland Cement Association for the stabilization of various soils are shown in Table 2.7.

Table 2.7: Cement Content Requirements for Soil Stabilization [12]

AASHTO Soil Group	Cement, Percentage by Dry Weight of Soil
A-1-a	3 – 5
A-1-b	5 – 8
A-2-4	5 – 9
A-2-5	5 – 9
A-2-6	5 – 9
A-2-7	5 - 9
A-3	7 – 11
A-4	7 – 12
A-5	8 – 13
A-6	9 – 15
A-7	10 - 16

For stabilization of the studied soil, 7% cement (by dry weight of soil) was selected as the initial trial cement content. In the selection, it was considered that the test is conducted indoors and the stabilized layer will not be subjected to freeze-thaw action and that in the current construction practice in the four Mid-West States, cements contents higher than 10% are almost never used because of the high cost of the materials.

Moisture-Density tests were conducted for the soil treated with 7% of cement within two hours of mixing, following the same procedure as for the untreated soil. In these tests, the portion of the soil passing No.4 sieve was initially dry mixed with cement and then different amounts of water were added and the soil was mixed again. The results of these tests are given in Figure 2.3.

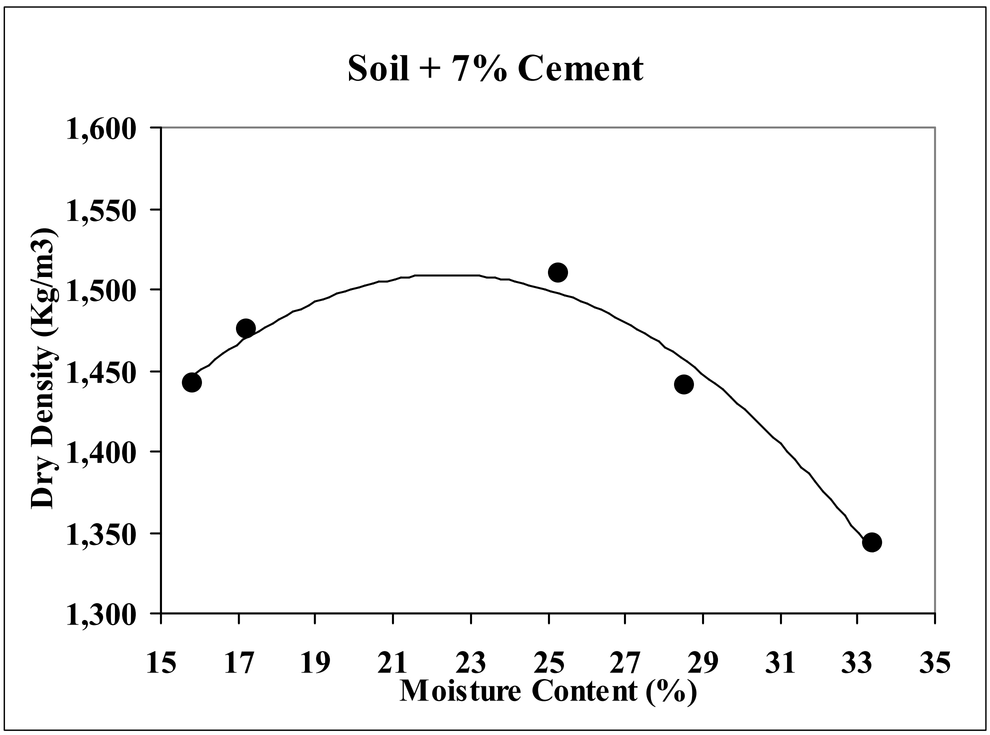


Figure 2.3: Results of Moisture-Density tests on Soils Treated with Cement

Lime: The estimated lime content required to stabilize the soil is typically obtained by determining the pH of soil-lime slurry. The pH test was conducted according to ASTM D 6276-99a [10]. Five samples of air-dried soil passing No.40 sieve, each weighing 20 grams, were poured in plastic bottles with closed caps. Lime contents of 2%, 3%, 4%, 5%, and 6% were mixed with each soil sample and mixed thoroughly by shaking the plastic bottles. Distilled water (100 ml) was added to each plastic bottle containing dry soil-lime mix and the soil-lime-water solution was mixed thoroughly. The solution was mixed by shaking the bottle for 30 sec at 10 minutes interval for one hour. After one hour, the pH of the solution in each bottle was determined by using pH meter. The results of this test for each soil are presented in Table 2.8.

Table 2.8: Results of the pH Test on the Soil-Lime Slurry

Soil	2%L	3%L	4%L	5%L	6%L	10% L (2 grams)
PH	13.06	13.13	13.17	13.17	13.24	13.27

According to Little [13], if the pH of the solution is above 12.4, the optimum content is the lowest of the two consecutive lime contents that have the same pH. From the test results, 4% lime for A-7-6 soil was selected as the initial trial lime content.

Standard Proctor Density tests were conducted for the soil treated with 4% lime. The portion of the soil passing No.4 sieve was initially dry mixed with lime and then different amounts of water were added to the dry mix and the mixture was mixed again. After mixing, the mix was placed in trays covered with flat plates and allowed to mellow for one hour. Moisture-Density tests were conducted using this mix by following the same procedure described for untreated soil. The results of these tests are shown in Figure 2.4.

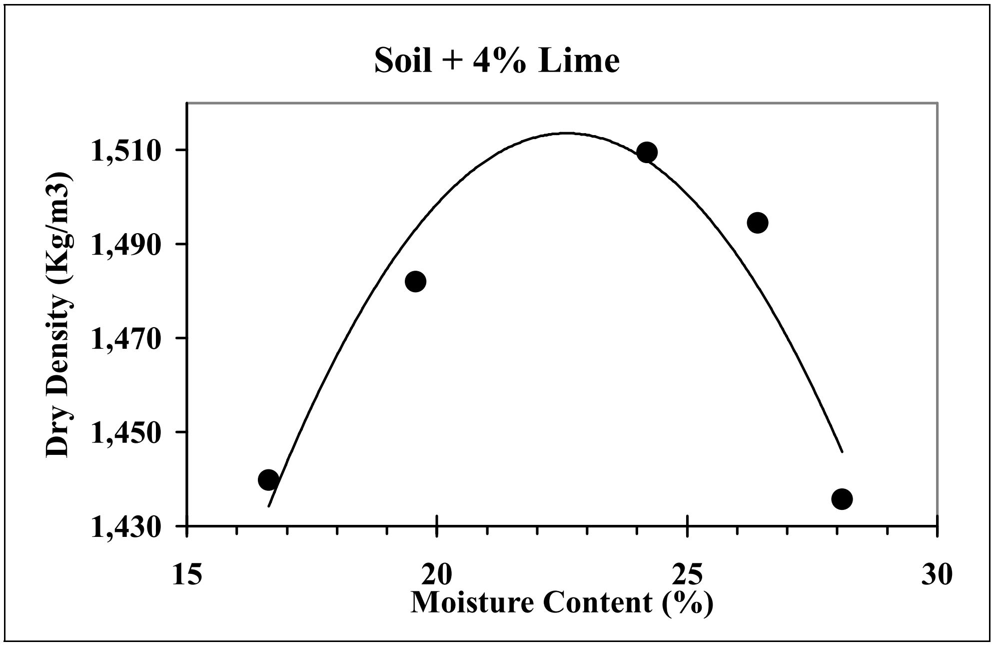


Figure 2.4 Results of Moisture-Density tests Conducted on Soils Treated with Lime

<u>Fly Ash</u>: The optimum content of fly ash required for the stabilization of the soils to meet the required Unconfined Compressive Strength (UCS) criteria was determined by conducting moisture-density tests according to ASTM D 558-00 [10]. Fly ash, having the contents of 12%, 14%, 16% and 18% was mixed on dry weight basis and moisture-density tests were conducted to find the MDD of the combined mixture. The portion of the soil passing No.4 sieve was initially dry mixed with fly ash and then different amounts of water were added to the dry mix and mixed again. Moisture-Density tests were performed on this mix (soil-fly ash-water) within two hours of the initial mixing following the same procedure described for untreated soil. The results of these tests are shown in Figure 2.5. Figure 2.6 shows the relationship between fly ash contents and maximum dry densities of fly ash treated soils. The initial trial fly ash content is typically

selected as the content that leads to the maximum dry density. Figure 2.6 indicates that the initial trial fly ash content for the studied soil is 15%.

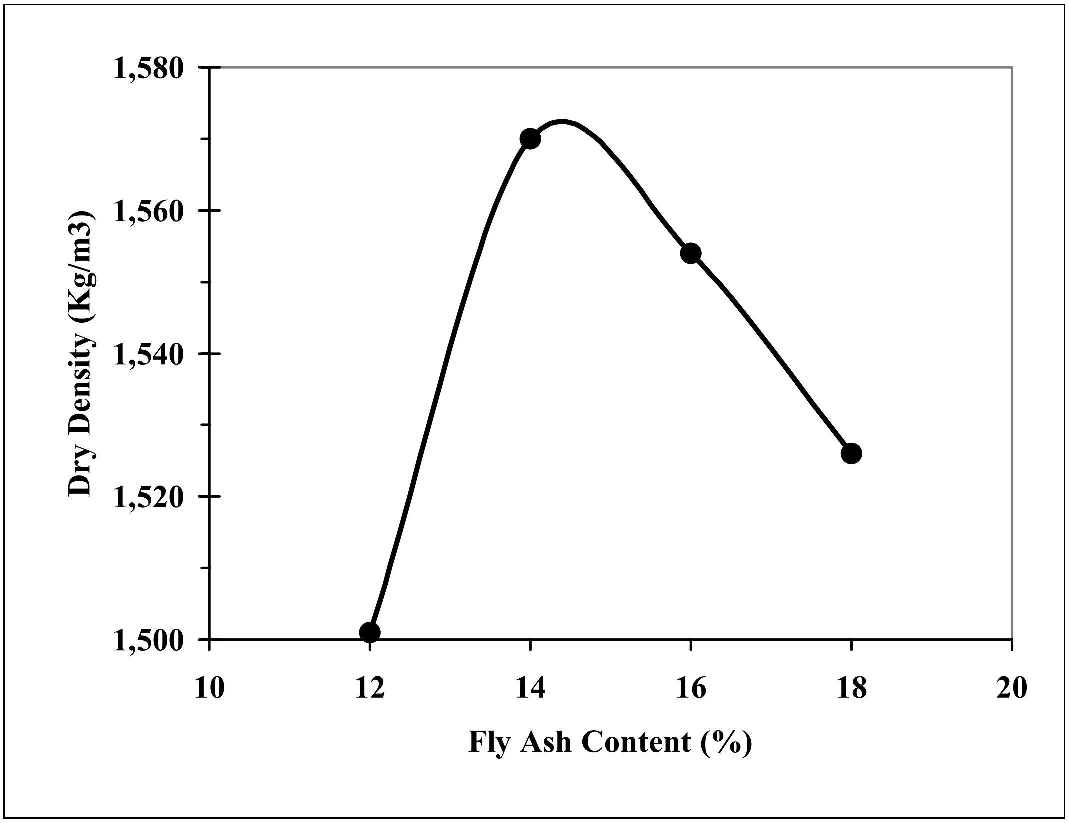


Figure 2.5: Moisture-Density tests Conducted on Soil Treated with Fly Ash

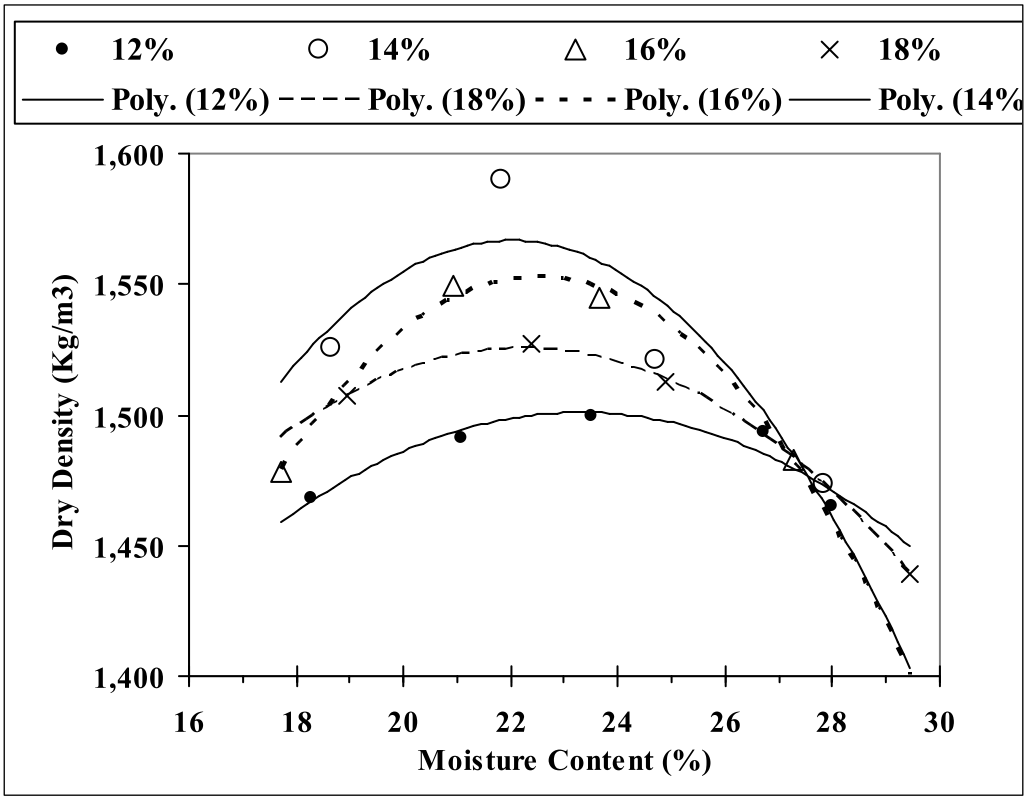


Figure 2.6: Dry density vs Fly-Ash Content

<u>EMC SQUARED® Stabilizer (1000)</u>: The application rate of EMC SQUARED® Stabilizer (1000) was provided by the manufacturer: 1 liter per 3 cubic meters of compacted soil [8]. The application rate does not change with the type of the soil to be stabilized. The amount of the stabilizer required was added to the water and then mixed thoroughly to ensure the uniform distribution of the stabilizer in the solution. The water-stabilizer mix was then added to the soil and then mixed for 3 minutes in the mechanical mixer. Moisture-density tests were conducted on the final mix following the same procedure described for untreated soil. The results of these tests are shown in Figure 2.7.

Moisture-Density relation for the soil treated with EMC SQUARED® Stabilizer (1000) was also determined using modified Proctor test according to ASTM D 1557 [10]. The results of this test are shown in Figure 2.8.

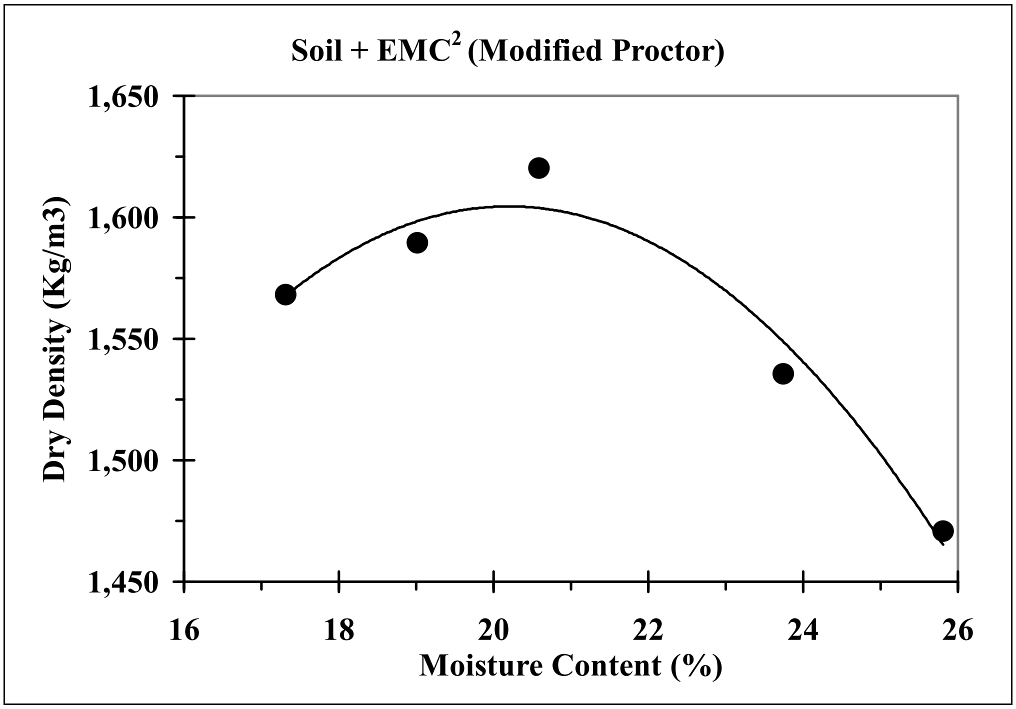


Figure 2.7 Standard Proctor tests results – EMC-Squared treated Soil

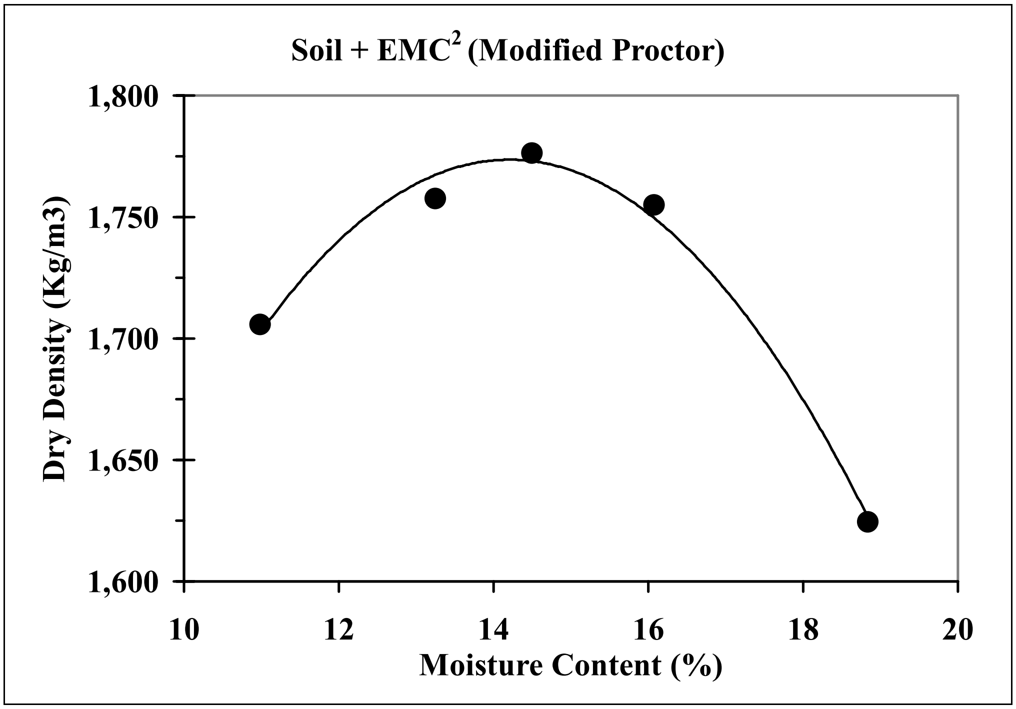


Figure 2.8 Modified-Proctor test results - EMC-Squared treated Soil

2.2.1 Effect of stabilization on OMC and MDD

Results of the standard Proctor tests conducted on untreated and treated soils according to ASTM D-698 [10] are presented in Table 2.9. According to ASTM D 4609, the single operator precision (due to experimental error) for ASTM D 698 [10] is 1.9% for MDD and 9.5% for OMC. Therefore, the change in Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) values beyond this precision may be interpreted as a result of the chemical treatment.

The results showed that the addition of cement and lime decreased the MDD of the soil. Addition of 12% fly ash reduced the MDD of the soil while the addition of EMC SQUARED® Stabilizer (1000) increased the MDD of the soil. Addition of cement and lime or 14% and 18% fly-ash did not change significantly the OMC of the soil. The addition of EMC SQUARED® Stabilizer (1000) increased the modified Proctor density from 1,752 Kg/m^3 for untreated soil to 1,778 Kg/m^3 for treated soil.

Table 2.9: MDD and OMC of the Stabilized Soil

Soil + Stabilizer	MDD (Kg/m^3)	OMC (%)	Is the change beyond the precision?	
			MDD	OMC
Untreated	1553	22.5	-	-
7% Cement	1512	22.5	YES	NO
4% Lime	1515	22.5	YES	NO
12% Fly ash	1501	23.5	YES	NO
14% Fly ash	1570	22.2	NO	NO
16% Fly ash	1554	22.5	NO	NO
18% Fly ash	1526	22.5	NO	NO
EMC[2]	1608	20.5	YES	NO
1.9% MDD = 29.5 Kg/m^3 and 9.5% OMC = 2.1%				

2.3 Unconfined Compression Strength Test

2.3.1 Mix preparation

Untreated Soil: The untreated soil was oven dried at 50^0C to make it break easily. Soil grinder was used to break the soil into smaller grains. The soil was then sieved through No. 4 sieve and the portion of the soil passing the sieve was used to prepare the soil-stabilizer mixtures. For untreated soil, the samples for UCS tests were prepared by mixing the soil with water to bring the moisture content to the OMC. The wet soil mass was then compacted.

Soil-cement mixture: Soil cement samples for unconfined compression test were prepared at three cement contents: 5%, 7% and 9%. The amount of each material required to make the required number of samples for UCS tests was calculated from the values of the density required, volume of the sample, OMC, and the cement content.

The MDD and OMC value of the soil treated with the initial trial cement content was considered applicable to the other cement contents in these calculations. This was done because it was assumed that a 2% variation of cement content would not change significantly the MDD and OMC values.

Prior to adding water, the required amounts of soil and cement were initially dry mixed for one minute in a mechanical mixer to ensure the uniform distribution of cement. After dry mixing, water required to bring the mix to OMC was added and the materials were mixed again for 3 minutes. The mix thus prepared was compacted. Compaction was completed within 2 hours after initial mixing.

Soil-lime mixture: Soil-lime samples for unconfined compression test were prepared at three lime contents: 4%, 6% and 8%. The amount of each material required to make the samples for UCS tests was calculated from the values of density required, volume of the sample, OMC, and the lime content. The MDD and OMC values of the soil treated with 4% lime were considered applicable for other lime contents. This was done because it was assumed that a small variation of lime content would not change significantly the MDD and OMC values.

Prior to adding water, the required amounts of soil and lime were initially dry mixed for one minute in a mechanical mixer to ensure the uniform distribution of lime. After dry mixing, water required to bring the mix to OMC was added to the dry mix and the materials were mixed for 3 minutes. The mix thus prepared was placed in an airtight plastic container to allow it to mellow for 1 hour. The mix was compacted; compaction was completed within two hours after initial mixing.

Soil-fly ash mixture: Soil-fly ash samples for unconfined compression test were prepared at the initial trial fly ash content and at the fly ash contents 3 % below and 3 % above initial trial fly ash content. Thus, fly ash contents of 12%, 15% and 18% were used. The amount of each material required to make the required number of samples for UCS tests was determined by using the values of the density required, volume of the sample, OMC, and the fly ash content. MDD and OMC determined for the soils treated with their corresponding fly ash contents were used in the mix design of soil-fly ash mixtures.

Prior to adding water, the required amounts of soil and fly ash were initially dry mixed for one minute in a mechanical mixer to ensure the uniform distribution of fly ash. The water required to bring the mix to OMC was then added and the materials were mixed again for 3 minutes. The mix thus prepared was compacted; compaction was completed within 2 hours of mixing.

Soil-EMC SQUARED mixture: The application rate of EMC SQUARED System was provided by the manufacturer: 1 liter per 3 cubic meters of compacted soil [8]. For unconfined compression test, two different mixtures were prepared for soils treated with EMC SQUARED® Stabilizer (1000).

For the first mixture, the amount of soil, stabilizer and water required were calculated using the results of standard Proctor tests conducted on corresponding untreated soils. The mixture for each soil was prepared at three different moisture contents and densities (95% & 100%). The moisture contents selected were 18%, 21% and 23%. The mixture was prepared at three moisture contents and densities (95%) obtained from modified Proctor test (Figure 3.2) in addition to standard Proctor test.

For the second mixture, the materials required were calculated using the results of standard Proctor tests. The mixture was prepared at MDD (95% & 100%) and OMC values obtained from both the Standard and Modified Proctor test.

For all mixtures, the required amount of stabilizer was added to the water required to bring the final mix to the required moisture content and then mixed thoroughly to ensure the uniform distribution of the stabilizer in the solution. The water-stabilizer solution was then added to the soil and mixed again for 3 minutes. The mix thus prepared was then compacted.

2.3.2 Specimen compaction

The samples for the UCS test were prepared in the cylindrical molds, shown in Figure 2.9, having an inside diameter of 2.8 inches and a height of 5.7 inches. Steel pistons were used at the top and bottom of the cylinder during compaction. The length of these pistons was designed to obtain samples of 5.7 inches in height. With these molds, samples with height to diameter ratio of approximately 2:1 were obtained. This height to diameter ratio gives better measure of compressive strength since it reduces the complex stress conditions that may occur during shearing of samples with smaller height to diameter ratio (ASTM D 1633-00) [10].

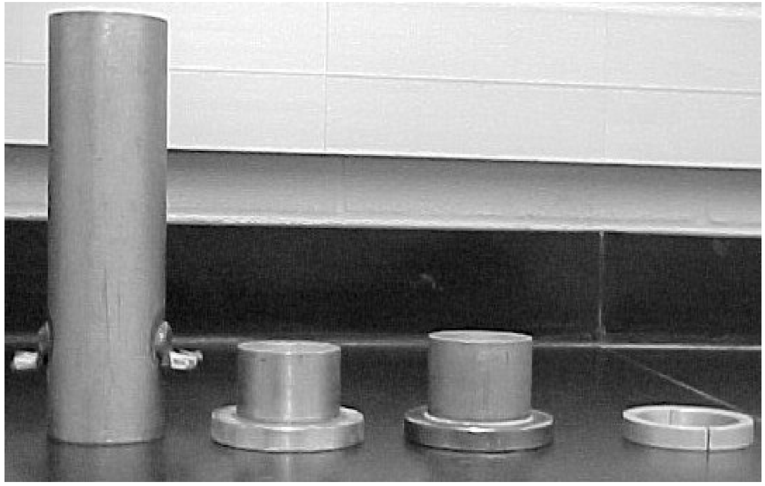


Figure 2.9: Mold and Pistons

The mold was then placed on the bottom piston. The predetermined amount of the soil-stabilizer mixture required to make 2.8-inch diameter by 5.7-inch high samples and to obtain required density (95% or 100% of standard Proctor density) was then poured in the mold using a funnel. The top piston was then placed on top of the mold and the mixture in the mold was compacted by applying a static load until the specimen height is 5.7 inches. The soil-stabilizer mixture in the mold was statically compacted to 95% and 100% of the required standard Proctor density and OMC. The samples of untreated and EMC treated soil were compacted at modified Proctor density in addition to standard Proctor density. The SATEC Model T5000 Electro-Mechanical Universal Testing System, shown in Figure 2.10 was used to apply the static load to compact the samples.

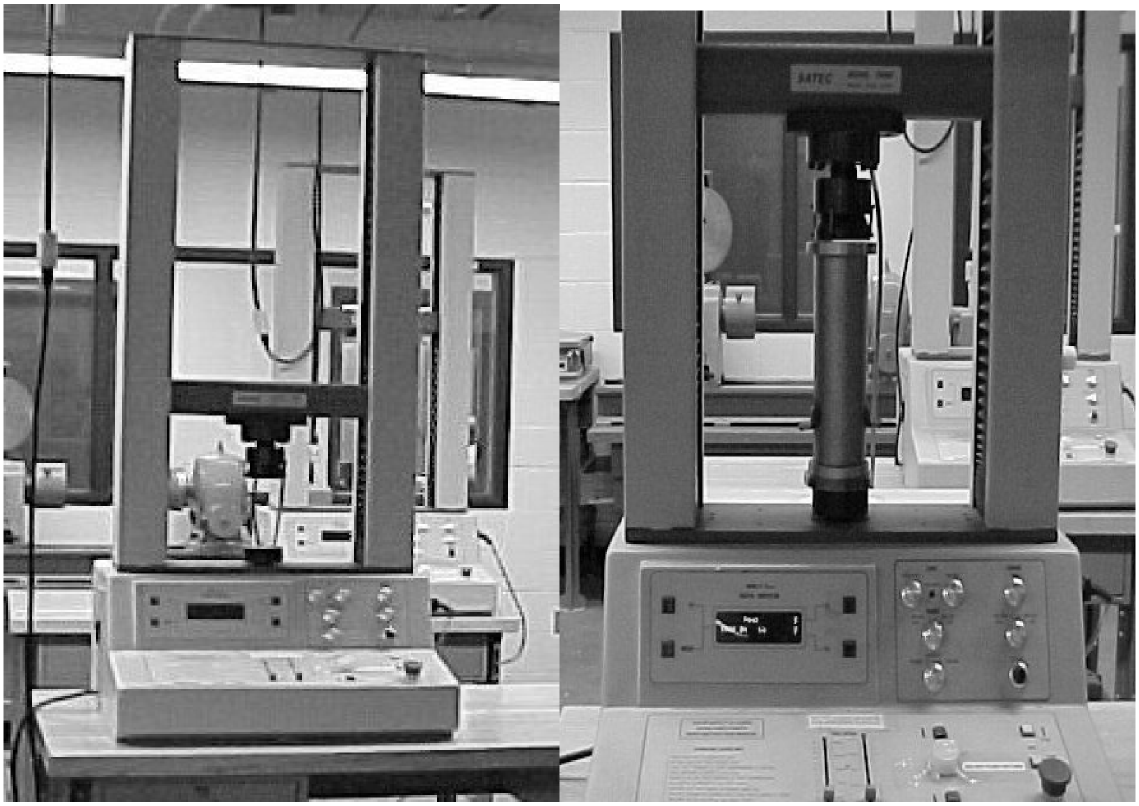


Figure 2.10: SATEC Model T5000 Electro-Mechanical Universal Testing System

2.3.3 Sample curing

The samples were extracted using a hydraulic sample extruder. After extraction, the samples were placed in the plastic bags and then cured in a moist curing room at 25^0C for 2, 7, 14, 28, 90 and 150 days. The EMC-Squared treated soils that were compacted to MDD (95% and 100%) and OMC values of treated soils were dried in the air before they were cured in the moist curing room. The drying period for the soil samples with EMC-Squared was reduced to 24 hours, because they showed cracks, as shown in Figure 2.11.

Figure 2.11: Samples of Soil Treated with EMC-Squared after 24 Hours of Drying

2.3.4 Unconfined compression test procedure

At the end of each curing period, the UCS of lime stabilized soil samples was determined according to ASTM 5102-96 [10]. Strength of soil samples stabilized with other stabilizers was determined according to ASTM 1633-00 [10]. The SATEC machine that was used to compact the samples was used for UCS test (Figure 2.10).

Two samples were tested for UCS at the end of each curing period, at the displacement rate of 0.05 inch/min. The samples were tested immediately after they were removed from the curing room. The load at which the two samples failed was recorded and UCS of each specimen was computed by dividing the load at failure by the initial cross sectional area of the specimen. The average of the strength of two samples was reported as the UCS.

2.3.5 Unconfined Compressive Strength (UCS) Test Results

The 7 days UCS strength of untreated soils was quite low: 24 psi at 95% MDD and 32.3 at 100% MDD. According to AASHTO soil classification [9], these soils would be fair to poor subgrade materials.

To find the effect of chemical stabilization on UCS, the soils were treated with the four stabilizers: cement, lime, fly ash and EMC SQUARED® Stabilizer (1000. The treated soils were then cured and UCS tests were performed. The results of all UCS tests are given in Appendix B.

Table 2.10 shows the results of these tests performed on the soil compacted at 95% and 100% of standard Proctor density. The values represent the average of the UCS of two specimens tested. The results indicate that cement produced higher strengths than did other stabilizers. UCS values for 150 days of curing ranged from 242 psi to 565 psi for cement treated soil. Lime treatment produced substantial improvement in the strength of the soil at 150 days of curing, with UCS values ranging from 212 psi to 434 psi. UCS values at 90 days curing ranged from 95 psi to 209 psi for fly ash treated soils.

Table 3.11 shows the strength of untreated and EMC SQUARED® Stabilizer (1000) treated soils at different conditions. The strengths of treated soils ranged from 13.8 psi to 45 psi for the samples with no drying period. The strength of the treated soil compacted at standard and modified Proctor densities increased significantly when the samples were dried in air for one day before moist curing.

2.3.6 Effect of stabilizer content on the UCS of stabilized soil

The results of the UCS tests indicate the following:

- Figure 2.12 shows that the UCS increased with the increase in the amount of cement. For 90 day curing period, the increase in the UCS ranged from 162 psi to 213 psi at 95% MDD and 178 psi to 395 psi at 100% MDD, with the increase in the cement content from 5% to 9%. With the increase in the cement content from 7% to 9%, the 7 days strength of the treated soil increased by 61 psi and 77 psi at 95% MDD and 100% MDD, respectively. The 7 days strength of 9% cement treated soil was twice the 7 days strength of 5% cement treated soil, at both compaction levels.
- Figure 2.13 shows that the UCS increased with the increase in the amount of lime. For different curing periods, the increase in UCS ranged from 40 psi to 141 psi at 95% MDD and 21 psi to 119 psi at 100% MDD with the increase in the lime content from 4% to 8%.
- Figure 2.14 shows the variation of UCS with fly ash content. The increase in the amount of fly ash did not produce a significant change in the strength of the soil at 95% MDD. At 100% MDD, an increase in the fly ash content from 12% to 18% produced an increase of 61 psi in 28 days strength.
- Figure 2.15 shows that the strength of EMC SQUARED® Stabilizer (1000) treated soil samples with no drying period did not change significantly with the change in the moisture content for all compaction levels.

Table 2.10: UCS of the Stabilized Soil Compacted at Standard Proctor Density

Stabilizer Content (%)	95% MDD Compaction						100% MDD Compaction					
	Curing Period-Days						Curing Period-Days					
	2	7	14	28	90	150	2	7	14	28	90	150
Cement												
5%	135.6	147.3	174.2	199.0	223.2	242.3	171.7	211.2	231.4	252.9	271.3	302.4
7%	174.7	234.9	296.9	327.3	385.3	320.7	276.0	361.0	367.7	389.8	450.2	449.0
9%	244.0	295.9	318.2	411.5	428.4	450.5	384.6	437.8	499.0	541.0	665.4	564.8
Lime												
4%	70.7	99	108.4	140.6	231.9	276.2	115.3	133	152.1	183.9	285	354.3
6%	96.3	109.1	114.4	146.9	270.3	211.9	139.7	149.8	187.7	194	315.8	434.2
8%	111.1	147.8	159	200.5	302.8	417.4	181.8	168.3	188.9	205.2	404.2	-
Fly Ash												
12%	69.7	76.2	95	95.3	95.7	92.5	93.5	106.7	115.5	120.6	146.8	-
15%	78.2	91.1	96.9	102.6	111.5	105.7	129.6	133.1	148.1	146.2	190.3	208.9
18%	79.8	99.5	108.2	111.8	118.5	-	110.7	142.9	151.7	182.1	207.8	-
EMC SQUARED (Moist Curing)												
18%	13.8	13.1	14.3	15.9	-	-	26.9	26.9	22.2	26.6	27.7	27.8
20%	27.6	31.6	32.3	30.6	28.1	-	34.5	27.8	34.0	28.3	35.3	23.6
23%	18.8	24.9	22.9	24.2	20.4	25.7	38.7	35.3	34.2	34.0	44.3	40.1

Table 2.11: UCS of the Soil Stabilized with EMC-SQUARED

Soil	Proctor	MC (%)	Density (%MDD)	Curing Period (days)					
				2	7	14	28	90	150
Untreated (Moist curing)	Std.	23%	95	-	24	-	-	-	-
		23%	100	-	32.3	-	-	-	-
	Mod.	14%	95	-	44.5	-	-	-	-
		14%	100	-	83.3	-	-	-	-
Untreated (Dry curing for the first day)	Std.	23%	95	-	67.5	-	-	-	-
		23%	100	-	106.4	-	-	-	-
	Mod.	14%	95	-	165.9	-	-	-	-
		14%	100	-	245	-	-	-	-
Mixed with EMC^2 (Moist curing)	Std.	18%	95	13.8	13.1	14.3	15.9	-	-
			100	26.9	26.9	22.2	26.6	27.7	27.8
		20%	95	27.6	31.6	32.3	30.6	28.1	
			100	34.5	27.8	34.0	28.3	35.3	23.6
		23%	95	18.8	24.9	22.9	24.2	20.4	25.7
			100	38.7	35.3	34.2	34.0	44.3	40.1
	Mod.	11%	95	123.7	132.6	119.2	133.2	118.4	115.7
		14%	95	106.4	104.3	94.4	90.3	107.0	97.0
		17%	95	75.1	74.0	77.0	67.0	79.4	77.7
Mixed with EMC^2 (Dry curing for the first day)	Std.	23%	95	-	109.0	112.2	-	-	-
			100	-	182.8	167.1	-	-	-
	Mod.	14%	95	-	201.8	172.4	-	-	-
			100	-	306.6	210.1	-	-	-

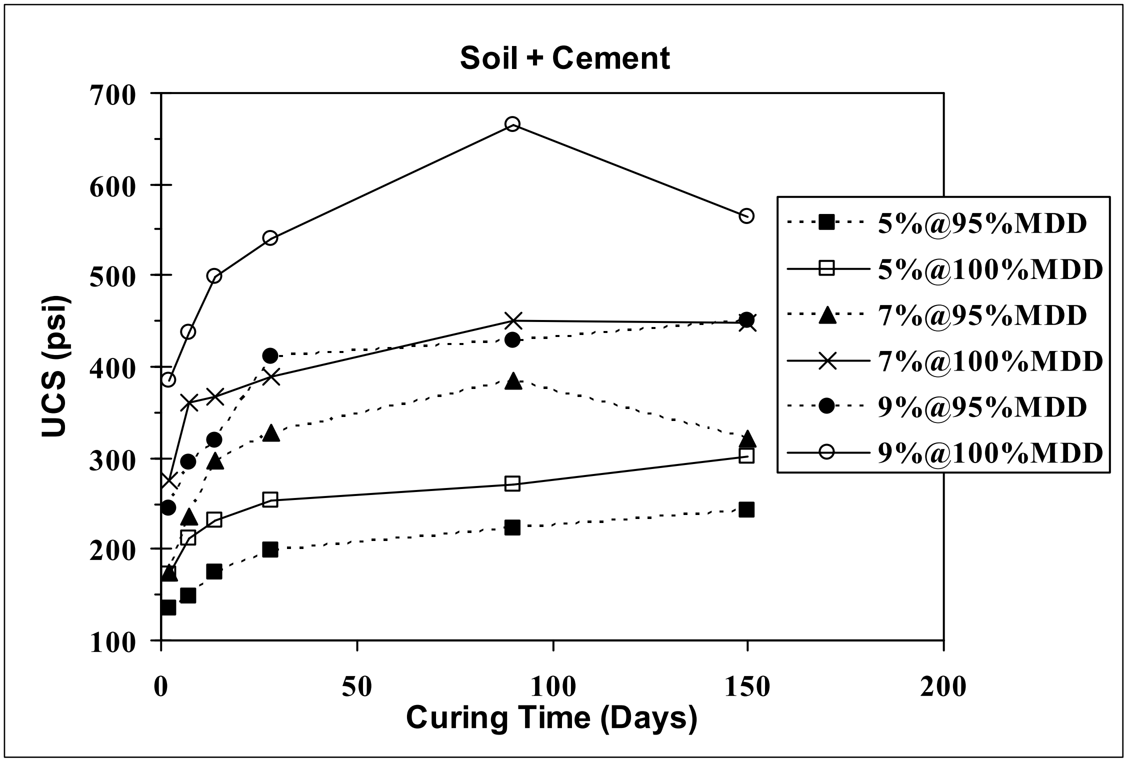


Figure 2.12: UCS of Cement Stabilized Soil

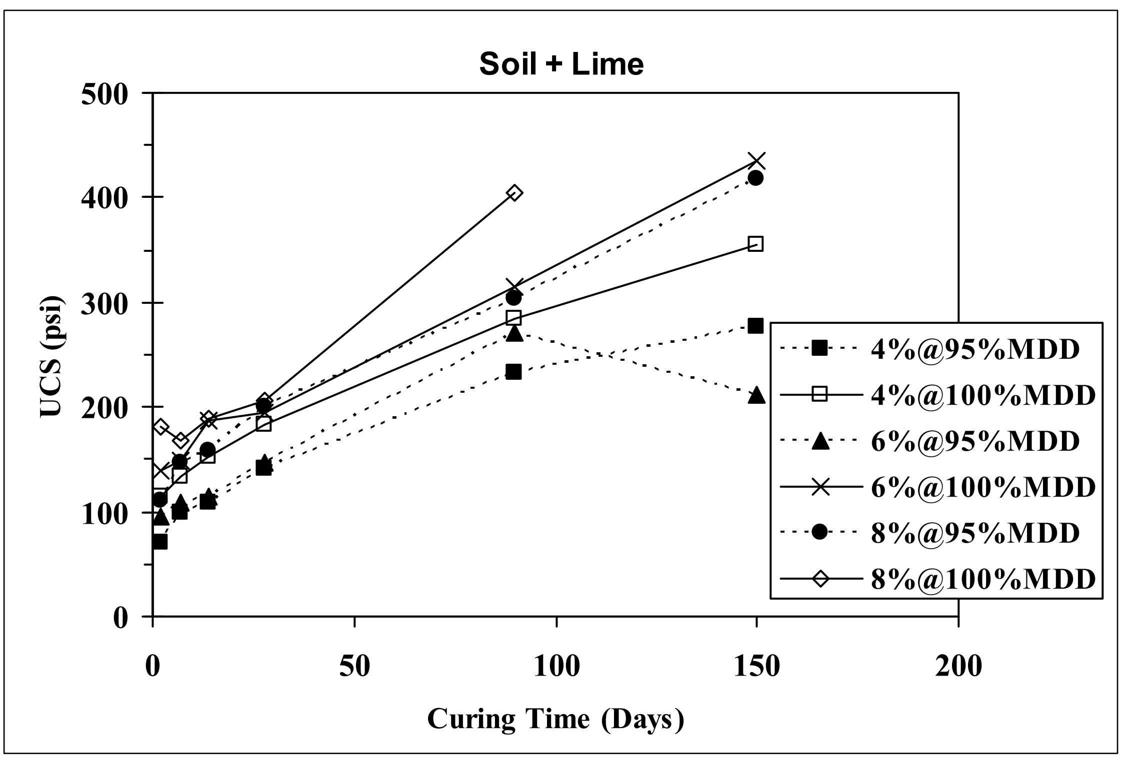


Figure 2.13: UCS of Lime Stabilized Soil

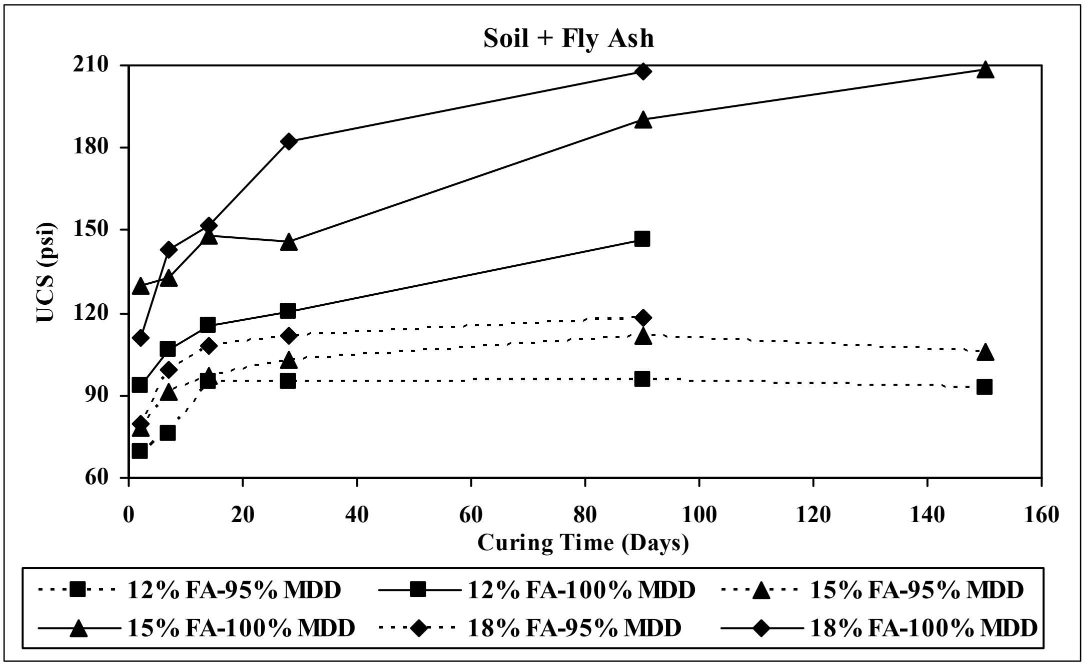


Figure 2.14: UCS of Fly Ash Stabilized Soil

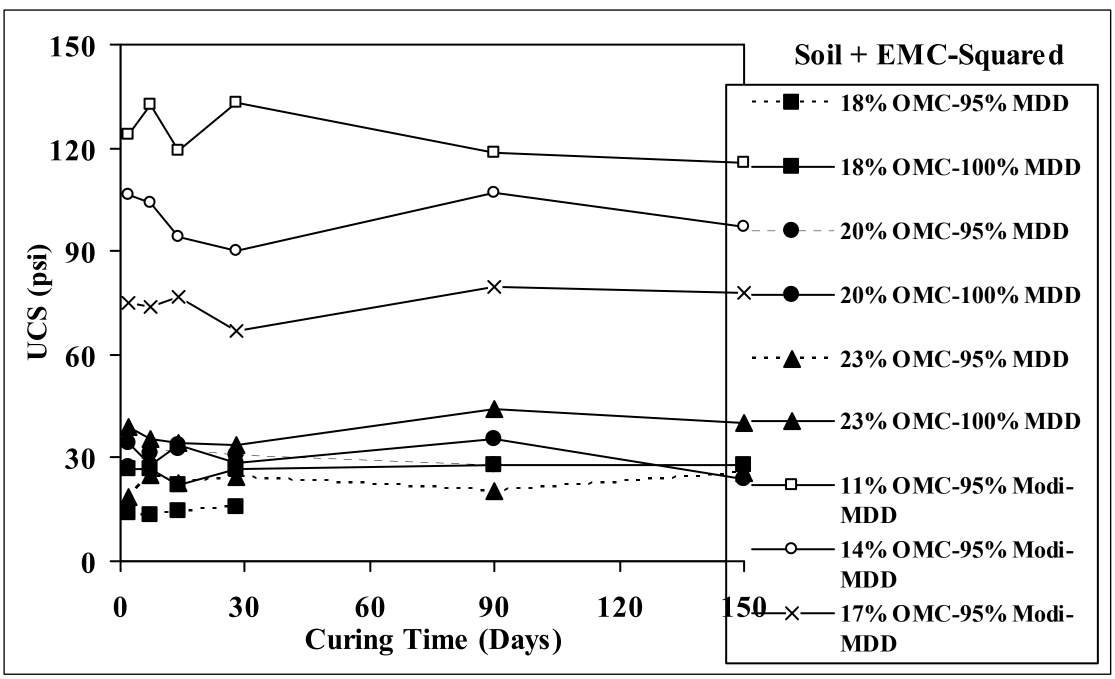


Figure 2.15 UCS of EMC Stabilized Soil

2.3.7 Effect of curing time on UCS of stabilized soils

UCS of all the soils treated with different amounts of cement, lime and fly ash increased with the curing time.

- With the increase in the curing period from 7 days to 28 days, the increase in the UCS ranged from 51 psi to 116 psi at 95% MDD and 28 psi to 103 psi at 100% MDD for cement treated soil (Figure 2.12). The strength was further increased about 20% with the increase in the curing period from 28 days and 150 days, for both compaction levels and for 5% cement content. For the same increase in curing period but for 7% cement content, UCS decreased slightly (2%) for the 95% MDD and increased by 15% for 100% MDD.
- Figure 2.13 shows that the strength of lime treated soil increased with the curing period for both compaction levels. The 90 days strength was more than 2 to 2.5 times higher than the 7 days strength. The strength further increased with the increase in the curing period to 150 days.
- The strength of fly ash treated soil slightly increased with the increase in curing period for both compaction levels (Figure 2.14).
- The strength of EMC SQUARED® Stabilizer (1000) treated soil samples at 23% moisture content with no drying period did not change significantly with the curing time for both compaction levels (Figure 2.15). The strength increased when the samples were dried for 24 hours before moist curing. For the samples compacted at 100% standard Proctor density, the 7-days strength increased from 35.3 psi to 182.8 psi (Table 2.11). For the samples compacted at 95% modified Proctor

density, the 7-days strength increased from 104.3 psi for untreated soil to 306.6 psi for treated soil.

Above results indicate that high early strengths were obtained with the addition of cement. This shows that the strength development of cement treated soils is mainly due to primary products formed during hydration reaction of cement.

Higher strengths were developed for lime and fly ash treated soils at longer curing periods. This shows that the strength development of these treated soils is mainly due to long-term pozzolanic reactions. The results also indicate that high strengths were achieved for cement in the first 7 days, than for other stabilized soils.

EMC SQUARED® Stabilizer (1000) produced an improvement in strength when the treated soil was allowed to dry in air before it was moist cured. However, this is difficult to achieve for the entire mass of the treated soil layer during field construction; the soil dries only at the top part of the layer. Therefore, the increased UCS obtained in the laboratory may not represent well the values the material would achieve in the field. It is also important to note that a high increase in UCS due to one day air curing was also observed for the untreated soil.

2.3.8 Effect of compaction level on UCS of stabilized soils

The samples for UCS test were compacted at 95% and 100% of standard Proctor density to find the effect of compaction level on the compressive strength of the soils. The UCS of the treated soil increased with the compaction level, for all the stabilizer contents at all the curing periods. The 7 days strength for cement treated soil and 28 days strength for soil treated with other stabilizers is generally considered during the evaluation of UCS of stabilized soils. Therefore, the strength increase with compaction level for these curing periods is further discussed.

With the increase in the compaction level of stabilized soil from 95% MDD to 100% MDD (Figures 2.12 to 2.15):

- The increase in 7 days strength of cement treated soil ranged from 63.9 psi to 142 psi for different cement contents.
- The increase in 28 days strength of 4% and 6% lime was 43 psi and 47 psi, respectively. The increase in the compaction level did not produce a significant change in the 28 days strength of the 8% lime treated soil.
- The 28 days strength of 18% fly ash treated soil increased from 111.8 psi to 182.1 psi.
- The strength of EMC SQUARED® Stabilizer (1000) treated soil increased with the compaction level. For the samples with one-day drying period, the 7-days strength increased from 109 psi to 182.8 with the increase in the compaction level from 95% to 100% of standard Proctor density. The 7-days strength increased from 201.8 psi to 306.6 with the increase in the compaction level from 95% to 100% of modified Proctor density.

2.4 Swelling Potential

Swell tests were conducted by KDOT personnel, according to the KDOT specification, “Method of Test for Determination of Volume Change of Soils” [14]. The method is used to determine the volume change of soil, soil mixed with admixtures, soil-aggregate mixtures or any desired fraction of soil-aggregate mixtures caused by the absorption of water. Tests were conducted on untreated soil and the soil treated with different contents of admixtures. The soil was mixed with different amounts of admixtures and the samples, 4 inches in diameter and 2 inches in height, were prepared for swell test. The samples were compacted at 92% of standard Proctor density and two moisture contents: OMC plus 3% and, OMC minus 3%. The required amount of material was carefully placed in the mold. The molding piston and filter paper were then placed in the mold and the material was compacted to the required height using a Carver Laboratory Press. The samples prepared were placed in a galvanized iron pan and the initial height of the sample was recorded using dial gauge. The pan was then filled with water up to a height equivalent to the height of the top of the sample inside the mold. The height of the sample at the end of 96 hours was determined. The percent of volume increase due to the absorption of water was determined by dividing the increase in height of the sample at 96 hours by the initial height of the sample.

A graph was drawn between the percent volume change and the moisture content, as shown in Figure 2.16. The percent of volume change for two moisture contents were joined by a straight line and the percent volume change at OMC was interpolated. In this example, the swelling potential of the untreated soil at OMC of 23% is 1.9%.

According to KDOT, a swelling potential of 2% is considered to be the threshold of concern for samples at OMC and compacted at 92% MDD [15]. Table 2.12 indicates that the swelling potential of the untreated soil was 1.9%, near to this 2% threshold level. Chemical stabilizers were added to the soils and swell tests were conducted to find their effect on the swell potential on the soils after treatment. The results of these tests, presented in Table 2.12, show that the swell potential of the treated samples was very low.

The increase in strength and the decrease in swelling potential are the indicators of the improvement in the engineering properties of the soil. From the results of the laboratory investigation, it is evident that the addition of cement and lime significantly increased the strength and decreased the swelling potential of the soil. Therefore these stabilizers may be considered effective for the stabilization of the studied soil.

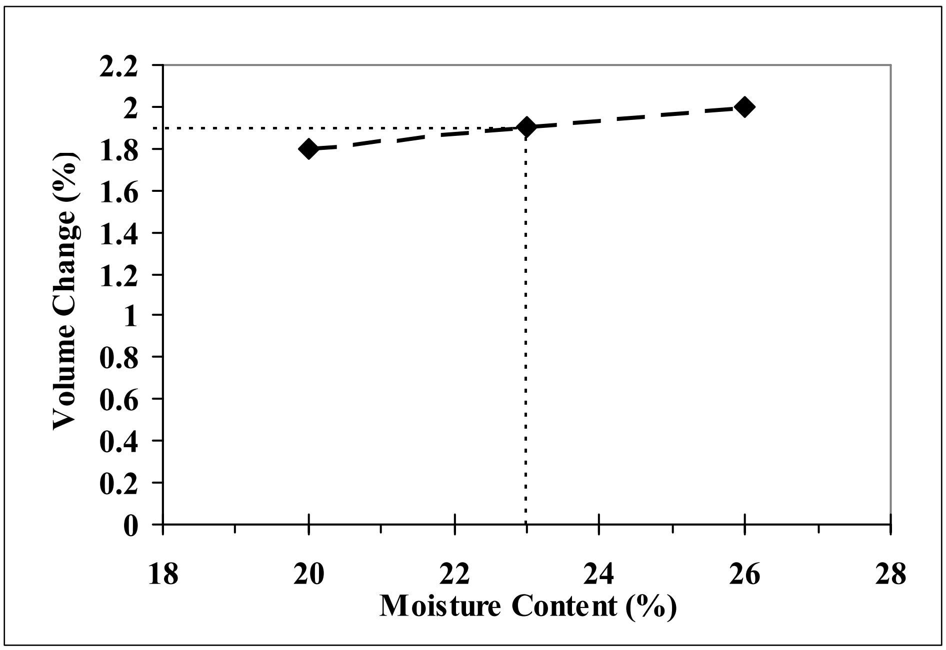


Figure 2.16: Linear Interpolation of Volume Change

Table 2.12: Results of the Swelling Potential Test

Sample Description	% Swell
Raw Soil	1.90
5% Portland Cement	0.1
7% Portland Cement	0.1
9% Portland Cement	0.1
4% Lime	0.05
6% Lime	0.10
8% Lime	0.05
12% Fly Ash	0.10
15% Fly Ash	0.10
18% Fly Ash	0.05

2.5 Selection of Optimum Stabilizer Content

The optimum stabilizer content of each product was selected based on the results of the unconfined compression strength tests, which were performed following ASTM 5102-96 and ASTM 1633-00 [10] at different stabilizer contents. The following stabilizer contents were selected as the optimum contents:

- 7% for Type I Portland cement. With the increase in the cement content from 7% to 9%, the 7 days strength of the treated soil increased only by 61 psi and 77 psi at 95% MDD and 100% MDD.
- 18% for Class C fly ash. The highest strengths were reached for 18% fly-ash content for both 95% and 100% MDD.
- 6% for quick lime. The increase in the amount of lime beyond 6% did not lead to an increase pH of the soil-lime slurry.

Figure 2.17 illustrates the unconfined compressive strengths of the stabilized soil at the optimum stabilizer and moisture contents. At a given compaction level, Type I

Portland *cement* resulted in the highest unconfined compressive strength over all curing periods, followed by lime and fly ash. The commercial stabilizer did not appear to be effective in strength gain over time.

The application rate of the commercial stabilizer, as recommended by the manufacturer, was 1 liter per 3 cubic meters of compacted soil. The stabilized samples for unconfined compression tests were prepared at three different moisture contents: 18%, 21% and 23%. The UCS test results showed that higher compressive strength was obtained at 23% moisture content. In agreement with the recommendations of the manufacturer, the commercial stabilizer-soil samples were compacted at 95% and 100% of the standard Proctor density and modified Proctor density. However, when the pavements were constructed, the compaction level and the optimum moisture content were selected based on the standard Proctor maximum dry density and the corresponding optimum moisture content. This was done so that the performance comparison with the other three stabilizers could be done at the similar compaction energy. It is to be noted that for paving projects, the commercial stabilizer manufacturer recommends that the mix design and compaction be done based on the modified Proctor test results.

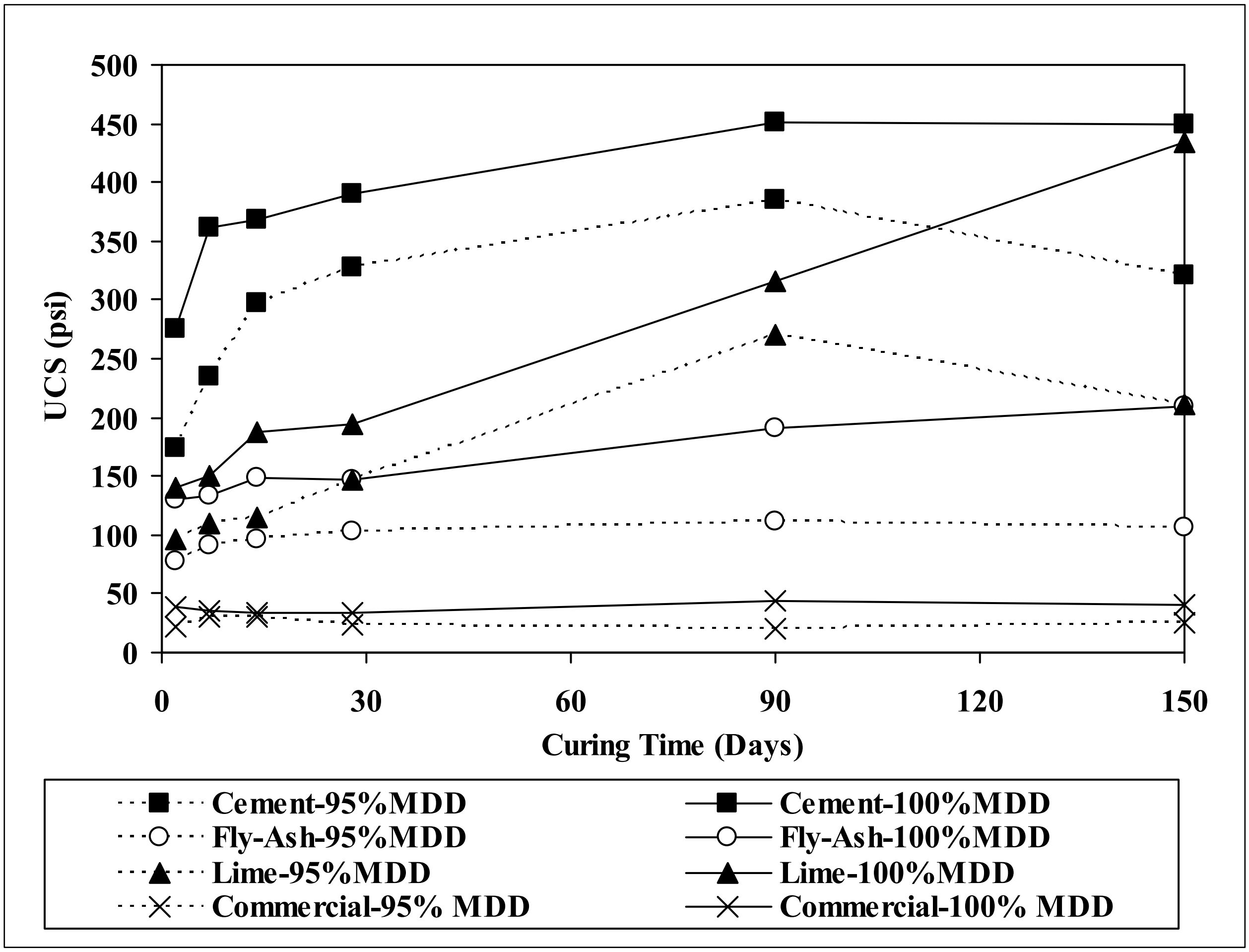


Figure 2.17: UCS of the stabilized soil at the optimum chemical content

CHAPTER 3 - DESCRIPTION OF THE TEST EXPERIMENT

This section gives a detailed description of the test, CISL experiment #12 including the determination of the design contents for the four stabilizers, pavement construction, loading conditions, sensor installation and data acquisition, and the performance monitoring plan.

3.1 The Accelerated Pavement Testing (APT) Facility at Kansas State University

The APT facility at the Civil Infrastructure Systems Laboratory (CISL) of Kansas State University is an in-door facility with about 7,000 ft^2 (651 m^2) floor space. The test pavements are constructed in two 6-ft (1.83-m) deep test pits of varying width and 20 ft (6.1 m) length. The accelerated loading is provided by the ATL machine. The main component of the machine are the steel frame, which has two main girders with 42 ft (12.8 m) center-to-center span, and the bogie, that is supported by the frame. The bogie is pulled back and forth by a rubber belt attached to an electric motor fixed on the frame. The wheel load assembly consists of a tandem axle mounted on the bogie. Loading of the axle assembly is accomplished with a hydraulic pump mounted on the bogie, above the axle, and connected to two hydraulic cylinders mounted on top of a single axle. The hydraulic pump pressurizes the oil in the hydraulic circuit and thus, the two cylinders push the bogie into the steel frame and the axle on the top of the test pavement. The hydraulic pump is also used to raise the bogie when uni-directional loading is applied. The axle load is controlled by the pressure in the hydraulic circuit. Load cells mounted on each wheel are used to measure the instantaneous wheel loads.

The bogie moves with the constant speed of 7 mph (11 km/h) above the test pavement; the acceleration and deceleration is done outside the test area. The bogie takes approximately 5.8 seconds to complete its travel distance in one direction. In bi-directional loading mode, approximately 650 passes of the bogie are applied in one hour of operation, and about 100,000 passes in one week. The operation is typically stopped for several hours weekly for the maintenance of the loading machine and the measurement of pavement response and performance. Typically, two test pavements are constructed in each pit and loaded simultaneously with one wheel of the axle passing above each pavement.

3.2 Test Bed and Construction

The test bed consists of two six feet deep pits, the North Pit (approx. 15 x 20 feet square) and the South pit (approx. 20 x 20 feet square). The pits are surrounded by the reinforced concrete walls. There is no integral drainage system for the pits. An 8 -12 in. layer of pea gravel was placed at the bottom of the pits and was covered by geotextiles for intrusion of fines from the subgrade layer.

In this study, four pavement sections were constructed in the pits, two in the North pit ((NN & NS) and two in the South pit (SN and SS). Figure 3.1 shows the schematic of the pavement cross sections. The subgrade and the base layers were placed in the second part of November in 2002. The asphalt concrete surface layer was constructed on the same day in both pits during the second week of December 2002.

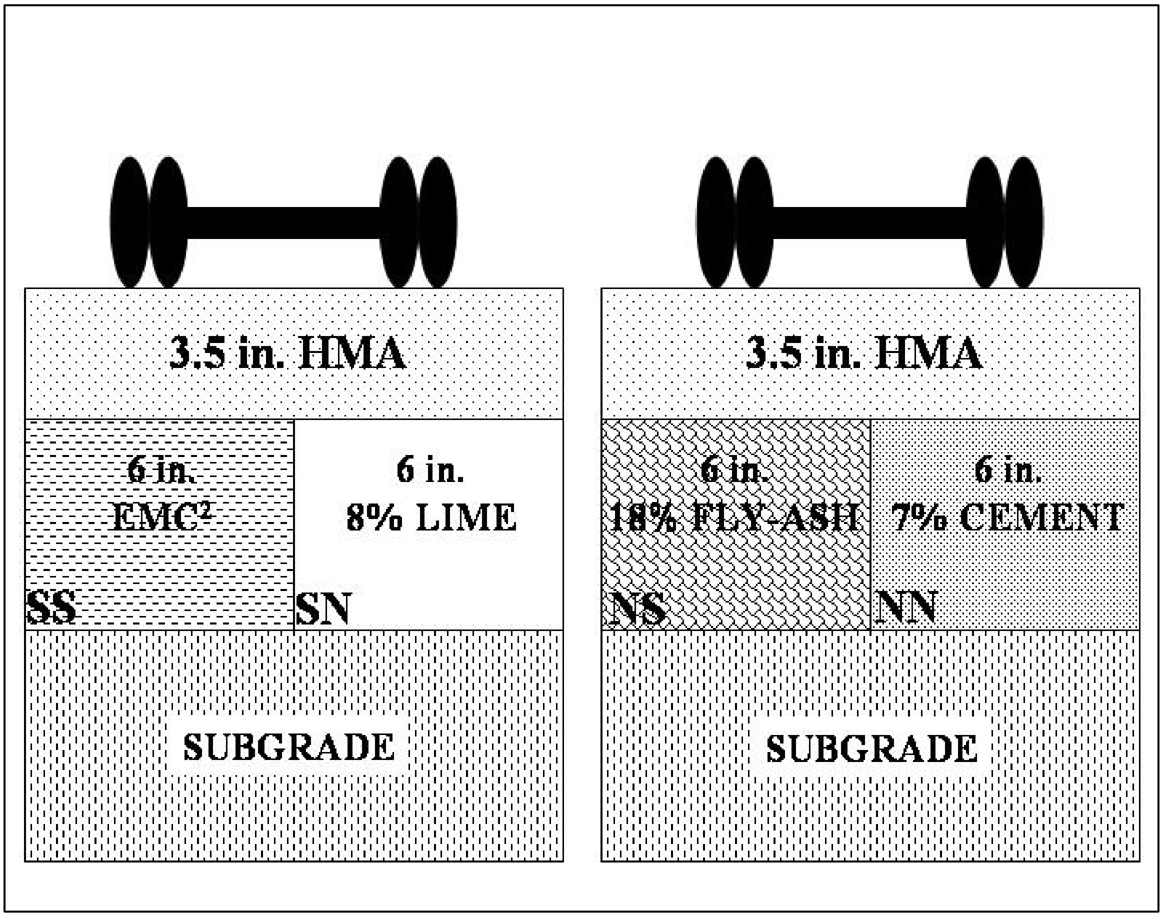


Figure 3.1: Cross Section of the Pavement Sections

3.2.1 Subgrade Soil

The engineering properties of the soil used in the subgrade layer are given in Section 2.12. The top two feet of the soil existing in the pit was removed. Then, the new soils was placed in the pit and compacted to a density greater than 90% of the maximum dry density (MDD) (Table 3.1 and Figure 3.2), at near optimum moisture content. The compaction was done manually with a 'jumping jack' type vibratory compactor. This subgrade was brought up to the required depth in two inch lifts.

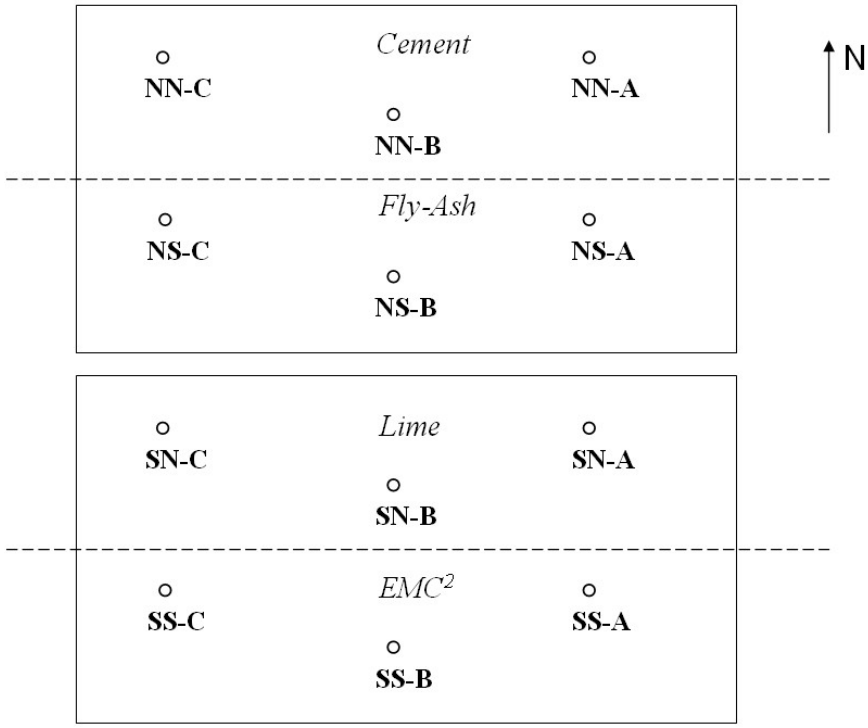


Figure 3.2: Locations of the Nuclear Density Measurement on Soil

3.2.2 Construction of Stabilized Soil Embankment Layer

Loose soil, of the same type as that placed at the top of the subgrade layer, was placed first in the pits. After the moisture content of the loose soil was measured, appropriate quantity of water and stabilizer were added. The mixture was then roto-tilled several times to ensure uniformity. The compaction was done with a rammer-type compactor in two-inch (50-mm) lifts (Figure 3.3). The as compacted density for the granular base, measured with the Troxler Nuclear Density gauge, is given in Figures 3.2 and Table 3.2.

Table 3.1: As-Constructed Densities on the Top 12 inches of Untreated Subgrade Layer

Lift	Point	Cement		Fly-Ash		Lime		EMC-Squared	
		DD	MC %	DD	MC %	DD	MC %	DD	MC %
12 in.	A	98.8	22.0	100.6	21.8	123.8	21.8	123.2	23.0
	B	99.1	21.7	99.8	22.0	122.9	21.2	123.5	21.5
	C	100.7	20.5	98.7	23.4	123.6	21.1	123.0	21.4
	Average	99.5	21.4	99.7	22.4	123.4	21.4	123.2	22.0
11 in.	A	100.9	20.6	99.3	23.2	124.1	20.0	123.2	20.8
	B	100.6	21.3	99.1	24.0	124.5	20.8	123.4	23.2
	C	101.4	20.6	98.9	24.4	123.7	20.3	124.7	20.7
	Average	101.0	20.8	99.1	23.9	124.1	20.4	123.8	21.6
10 in.	A	101.0	21.8	100.8	22.3	124.5	20.8	124.6	21.1
	B	103.5	19.3	99.2	23.5	124.5	21.3	124.2	23.0
	C	101.2	21.6	99.8	23.5	123.8	21.4	124.2	21.8
	Average	101.9	20.9	99.9	23.1	124.3	21.2	124.3	22.0
9 in.	A	103.8	19.3	100.5	23.3	123.5	21.0	124.2	20.6
	B	102.6	20.7	100.2	23.6	124.3	19.9	123.9	21.4
	C	102.0	21.1	100.2	23.3	124.7	20.4	124.1	21.7
	Average	102.8	20.4	100.3	23.4	124.2	20.4	124.1	21.2
8 in.	A	102.5	20.0	98.2	24.3	125.2	21.3	124.3	22.0
	B	102.3	21.6	100.6	22.8	125.1	19.9	124.3	22.6
	C	101.9	20.9	99.5	22.8	125.2	19.7	124.2	21.5
	Average	102.2	20.8	99.4	23.3	125.2	20.3	124.3	22.0
7 in.	A	102.5	20.7	98.3	24.8	125.0	20.4	125.0	22.8
	B	104.6	19.1	101.5	21.7	125.5	21.2	124.6	20.9
	C	101.6	22.0	99.0	23.7	125.6	19.6	124.7	21.5
	Average	102.9	20.6	99.6	23.4	125.4	20.4	124.7	21.7
6 in.	A	102.4	20.4	99.1	23.8	125.0	21.5	124.7	21.2
	B	101.1	22.1	100.0	22.4	125.0	20.6	124.4	22.0
	C	102.1	20.9	98.7	24.1	125.3	19.5	124.3	21.4
	Average	101.9	21.1	99.3	23.4	125.1	20.5	124.5	21.5
5 in.	A	101.1	20.9	99.6	23.0	125.3	21.3	124.9	21.7
	B	102.2	20.2	98.7	23.7	124.7	20.3	124.7	21.2
	C	101.2	21.5	98.9	23.7	125.0	20.5	125.3	21.0
	Average	101.5	20.9	99.1	23.5	125.0	20.7	125.0	21.3
4 in.	A	101.9	20.5	98.0	24.4	126.7	19.4	124.5	22.1
	B	101.2	21.5	98.7	23.8	125.1	21.4	124.4	22.8
	C	102.1	20.7	99.2	22.7	125.8	19.8	124.4	20.3
	Average	101.7	20.9	98.6	23.6	125.9	20.2	124.4	21.7
3 in.	A	103.3	20.1	98.8	22.8	125.7	20.1	123.4	22.4
	B	103.2	20.4	98.0	24.1	127.0	19.1	124.5	20.8
	C	102.8	21.1	99.7	22.3	126.2	20.0	124.3	22.0
	Average	103.1	20.5	98.8	23.1	126.3	19.7	124.1	21.7
2 in.	A	104.7	19.5	99.6	22.0	126.5	21.5	122.9	20.6
	B	105.5	18.6	98.4	25.1	126.2	21.4	123.1	20.9
	C	103.3	21.1	98.7	23.9	125.9	20.7	123.7	21.0
	Average	104.5	19.7	98.9	23.7	126.2	21.2	123.2	20.8
AVERAGE		102.1	20.7	99.3	23.3	125.0	20.6	124.1	21.6

DD - In-Situ Dry Density (pcf); MC – In-situ Moisture Content (%)

Table 3.2: Measured As-Constructed Densities on the Stabilized Embankment Layer

Depth	Point	Cement Section				Fly-Ash Section			
		West End		East End		West End		East End	
		DD (pcf)	MC (%)	DD (pcf)	MC (%)	DD (pcf)	MC (%)	DD (pcf)	MC (%)
6 in.	A	96.5	24.1	98.0	22.7	97.3	15.5	98.8	17.0
	B	98.1	22.9	98.0	21.7	97.2	15.8	97.5	18.3
	C	97.4	23.3	96.7	23.4	98.5	14.8	97.6	19.0
	Average	97.3	23.4	97.6	22.6	97.7	15.4	98.0	18.1
5 in.	A	96.2	24.0	94.8	23.7	96.8	15.5	98.8	16.9
	B	95.8	23.3	95.2	23.9	96.1	16.1	97.8	17.8
	C	96.5	23.2	94.0	25.0	96.6	15.2	97.8	17.7
	Average	96.2	23.5	94.7	24.2	96.5	15.6	98.1	17.5
4 in.	A	94.8	24.3	93.7	23.5	99.3	14.2	96.8	18.1
	B	95.5	24.7	93.4	24.4	98.3	15.0	97.2	18.0
	C	95.6	24.6	91.5	25.6	96.7	16.3	96.8	17.9
	Average	95.3	24.5	92.9	24.5	98.1	15.2	96.9	18.0
3 in.	A	94.9	24.8	90.2	25.4	99.2	15.3	98.2	18.1
	B	96.4	22.7	91.4	24.3	99.6	14.4	99.1	17.2
	C	94.9	24.9	90.8	24.9	99.3	15.0	98.8	17.3
	Average	95.4	24.1	90.8	24.9	99.3	14.9	98.7	17.5
2 in.	A	93.3	24.6	90.5	24.4	102.5	15.1	99.7	18.5
	B	93.9	25.2	88.9	26.0	104.6	14.2	99.6	18.4
	C	93.9	25.0	90.1	23.9	104.7	13.3	100.2	17.1
	Average	93.7	24.9	89.8	24.8	103.9	14.2	99.8	18.0
Depth	Point	Lime Section				EMC-Squared Section			
		West End		East End		West End		East End	
		DD	M %	DD	M %	DD	M %	DD	M %
6 in.	A	90.4	28.1	92.0	27.4	98.0	23.2	96.1	24.9
	B	89.9	28.0	92.6	27.4	96.9	25.1	96.6	24.7
	C	90.0	28.0	91.0	29.1	97.0	25.3	95.4	25.9
	Average	90.1	28.0	91.9	28.0	97.3	24.5	96.0	25.2
5 in.	A	91.5	25.8	92.0	27.1	96.0	25.6	94.8	26.0
	B	90.5	27.4	93.3	26.2	96.5	24.5	94.8	25.6
	C	89.9	27.6	91.3	27.5	96.9	25.1	94.5	26.2
	Average	90.6	26.9	92.2	26.9	96.5	25.1	94.7	25.9
4 in.	A	90.7	26.6	91.2	26.9	95.8	25.9	94.4	25.0
	B	89.7	28.1	89.7	28.2	96.0	25.6	94.7	25.0
	C	87.7	30.5	89.9	28.2	97.6	24.3	94.8	25.0
	Average	89.3	28.4	90.2	27.8	96.5	25.3	94.6	25.0
3 in.	A	88.4	28.3	89.3	28.8	95.7	26.3	93.2	26.6
	B	88.0	28.9	89.5	28.2	96.6	25.6	94.2	25.3
	C	89.0	27.8	88.7	29.9	96.0	24.3	94.0	25.6
	Average	88.5	28.3	89.2	29.0	96.1	25.4	93.8	25.8
2 in.	A	86.3	29.1	88.9	28.7	97.6	24.3	92.2	26.1
	B	88.3	28.5	89.5	28.3	97.7	23.9	92.3	26.8
	C	86.1	29.7	90.0	28.1	97.2	24.9	92.9	25.6
	Average	86.9	29.1	89.4	28.4	97.5	24.4	92.5	26.2

Figure 3.3: Compaction of the Lime-Treated Soil

3.2.3 Construction of the Asphalt Concrete Surface Layer

The 3-inch asphalt layer above the base was placed in one lift on all lanes in two pits. During construction, paving was done by Shilling Construction Co. of Manhattan, Kansas, and the compaction was done with a steel-wheeled vibratory roller. The asphalt layer consisted of a 9.5 mm nominal maximum size Superpave mixture. The combined aggregate gradation of this mixture, designated as SM-9.5B in Kansas, passes below the maximum density line in the sand sizes. At least 50 tons of mixture were produced before building the test sections at the CISL. The mixture consisted of 18% KS Falls crushed limestone (CA-5), 38% Zeandale 1/4” chips, 15% Onaga ½” CS-2 and 29% Onaga Manufactured Sand. The asphalt binder was a PG 64-22. The gradation data of the aggregate material are given in Table 3.5. The gradation curve for the combined aggregate is given in Figure 3.4.

The mix was placed on all pavements in the same day using conventional paving and compaction equipment. Figures 3.6 to 3.8 show the paving and the compaction operation. The as-constructed density of the HMA was measured with a nuclear density gage, in three locations in each section (West, Middle and East), as shown in Figure 3.5. Three repeated measurements (numbered 1, 2 and 3) were performed in each location. Table 3.6 gives the results of the density measurements for the as-constructed hot-mix asphalt.

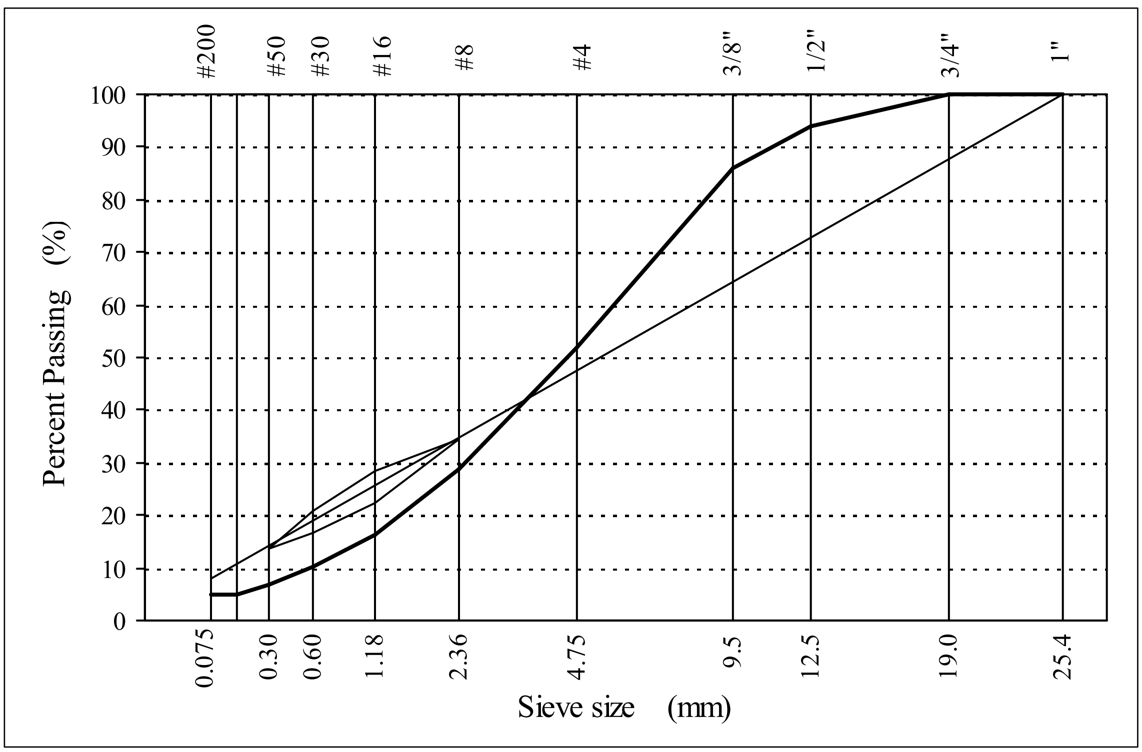


Figure 3.4: Gradation Curve of the Aggregate in the HMA

Table 3.3: Gradation data for the aggregate from stockpiles (percent retained)

Sieve	Zeandale ¼" chips	KS Falls CA-5	Onaga CS-2	Onaga ManSand	Combined
1"	0	0	0	0	0
¾"	0	0	0	0	0
½"	0	33	0	0	5.9
3/8"	0	72	7	0	14.0
#4	65	97	37	1	48
#8	95	97	57	32	71.4
#16	98	97	67	65	83.6
#30	98	97	74	82	89.6
#50	98	97	79	92	93.2
#100	98	97	85	95	95
#200	99	98	87	96	96.1
G_{sb}	2.581	2.514	2.574	2.39	2.510

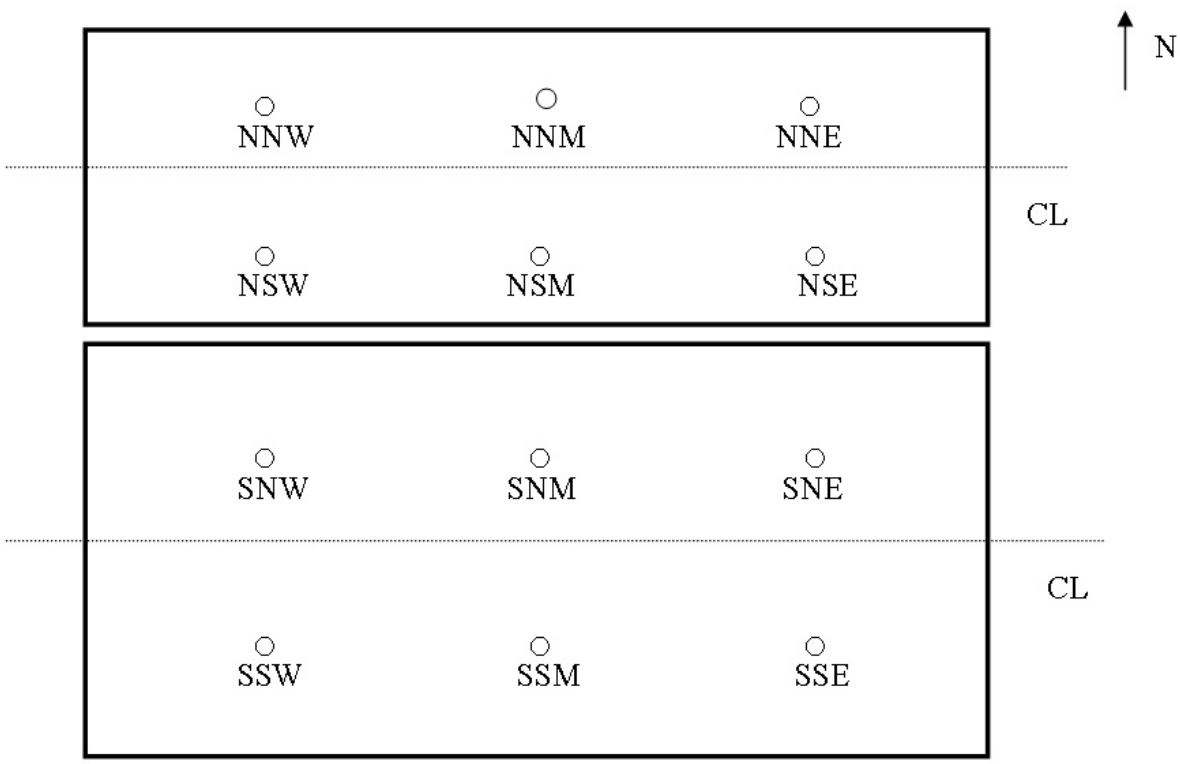


Figure 3.5: Location of Nuclear Density Measurements on the Asphalt Surface Layer

Table 3.4: Measured As-Constructed Densities on the Asphalt Surface Layer

Location		Section			
		Cement NN	Fly-Ash NS	Lime SN	EMC-Squared SS
W	1	126.5	127.6	125.9	129.9
	2	127.1	127.5	127.7	128.4
	3	129.0	127.4	126.9	130.0
	Average	127.5	127.5	126.8	129.4
M	1	122.6	128.3	125.3	127.0
	2	123.2	126.5	125.4	127.8
	3	124.4	127.0	123.6	127.2
	Average	123.4	127.3	124.8	127.3
E	1	120.2	130.2	128.7	127.4
	2	122.2	130.2	125.8	127.0
	3	120.4	127.0	126.2	127.3
	Average	120.9	129.2	126.9	127.2

Figure 3.6: Hot Mix Asphalt Paving on Lime Treated Section

Figure 3.7: Asphalt Concrete Paving

Figure 3.8: Compaction of Asphalt Concrete

3.2.4 As-constructed Layer Thicknesses

The thickness of the as-constructed layers was determined by measuring with surveying equipment on top of each constructed layer the elevation of 19 points spaced at one-foot intervals along a straight line corresponding to the position of the outside wheel path. The points were numbered from east to west, with the first point being at one foot west of the east wall of the pit. A fixed point at the base of a steel pole near the east gate of the CISL laboratory was used as reference. The elevations recorded at the top of the compacted subgrade soil, at the top of the stabilized soil and on top of the pavement surface are given in Appendix C. The thickness of the as-constructed layers, computed as the difference between the elevations recorded in the same point are given in Table 3.5 and plotted in Figures 3.9 and 3.10.

Table 3.5: As-constructed Layer Thickness (inches)

Point	Stabilized Soil Layer				Asphalt Concrete Layer			
	Test Section				Test Section			
	NN	NS	SN	SS	NN	NS	SN	SS
1	5.784	5.436	6.624	6.864	3.84	4.632	2.724	3.204
2	6.252	5.484	6.456	5.916	3.876	4.620	2.640	3.144
3	6.108	5.688	7.032	5.688	3.804	4.308	2.484	3.048
4	6.528	5.748	6.432	5.700	3.888	3.996	2.196	3.024
5	6.468	6.576	6.816	5.952	3.996	3.660	2.124	3.372
6	6.192	6.132	6.600	5.556	3.768	3.828	2.388	3.60
7	6.660	5.688	6.300	5.232	3.732	3.888	2.460	3.48
8	6.564	6.144	6.204	4.824	3.624	3.780	2.424	3.744
9	6.780	6.288	5.868	5.196	3.228	4.020	2.724	3.432
10	6.672	6.252	5.748	4.668	2.976	3.888	2.640	3.288
11	6.792	6.108	5.832	4.560	2.976	3.744	2.640	3.396
12	6.888	5.844	5.928	5.148	2.664	3.756	2.508	3.384
13	6.768	6.828	5.988	4.668	2.808	3.276	2.448	3.468
14	6.384	6.816	6.000	4.452	3.432	3.516	2.484	3.24
15	6.372	6.804	6.072	5.412	3.768	3.348	2.640	2.928
16	6.300	6.600	5.712	5.340	3.456	3.276	2.844	3.204
17	6.156	6.648	5.256	5.412	3.612	3.312	2.808	3.54
18	6.048	6.468	5.484	5.436	3.888	3.312	3.204	3.84
19	5.460	6.048	5.544	5.88	4.188	3.684	3.768	3.588
AVERAGE	6.38	6.19	6.10	5.36	3.55	3.78	2.64	3.36
Standard Deviation	0.370	0.454	0.474	0.590	0.431	0.413	0.364	0.243
CV (%)	5.8	7.3	7.8	11.0	12.1	10.9	13.8	7.2

Figure 3.9 indicates that the thickness of the stabilized soil layer was relatively close to 6.0 inches, the nominal thickness, for the section NN, NS and SN. However, the thickness was less than 6.0 inches for the section with embankment layer treated with EMC-Squared. Figure 3.10 suggests that the thickness of the asphalt concrete layer was relatively close to 3.5 inches, the nominal thickness, for the section NN, NS and SS. However, the thickness was less than 3.5 inches for the section with lime treated embankment layer.

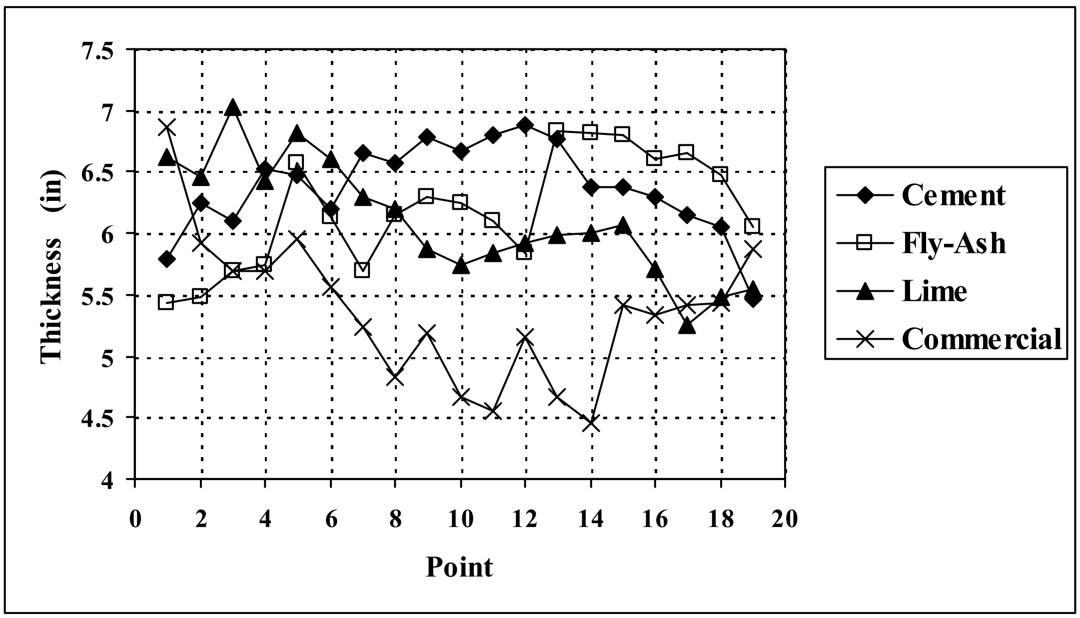


Figure 3.9: As-constructed Thickness of the Stabilized Soil Layer

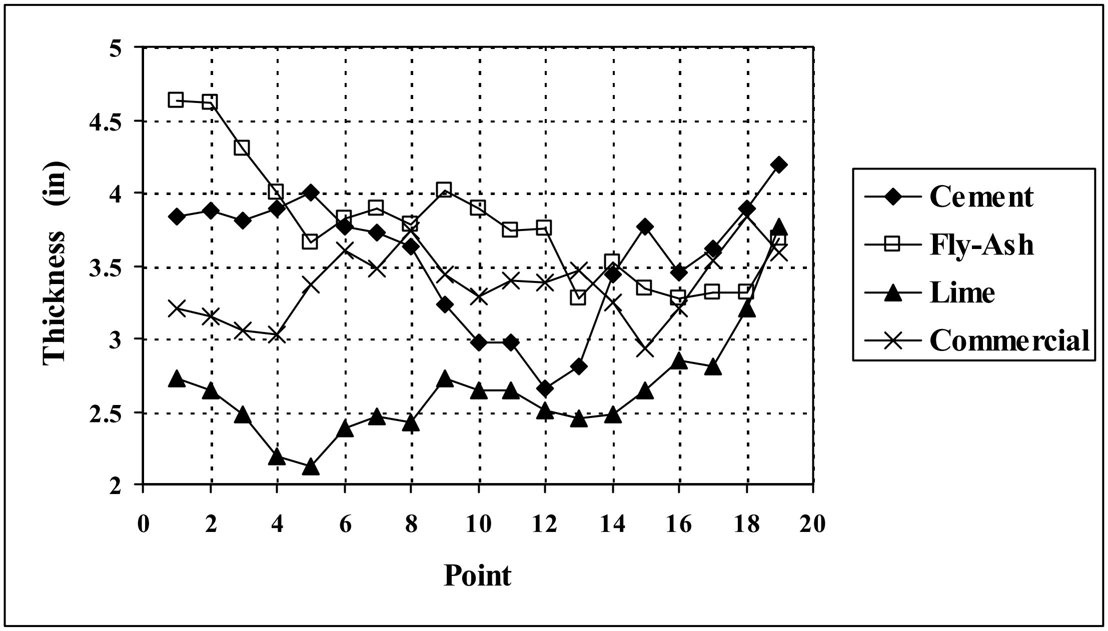


Figure 3.10: As-constructed Thickness of the Asphalt Concrete Layer

3.3 Instrumentation and Pavement Condition and Response Monitoring

Several sensors were placed in the test sections to monitor pavement behavior. In addition to complement measurements obtained from these sensors, Falling Weight Deflectometer (FWD) and weight drop deflections were also recorded.

3.3.1 Pressure Cells

Two Pressure cells (Geokon) were placed at the bottom of the base layer in the centerline of each pavement section to measure the vertical compressive stress at the top of the soil subgrade. The relative locations of the pressure cells are shown in Figure 3.11. One cell was placed in the western part of the lane and the other one in the eastern part. These 6-inch diameter Geokon pressure cells were successfully used in previous projects and have shown good performance and acceptable results. These sensors were installed according to the manufacture's guidelines. After the subgrade was compacted, holes were drilled to place the pressure cells. After the horizontal alignment was checked with a level, the cells were covered with a thin layer of sand.

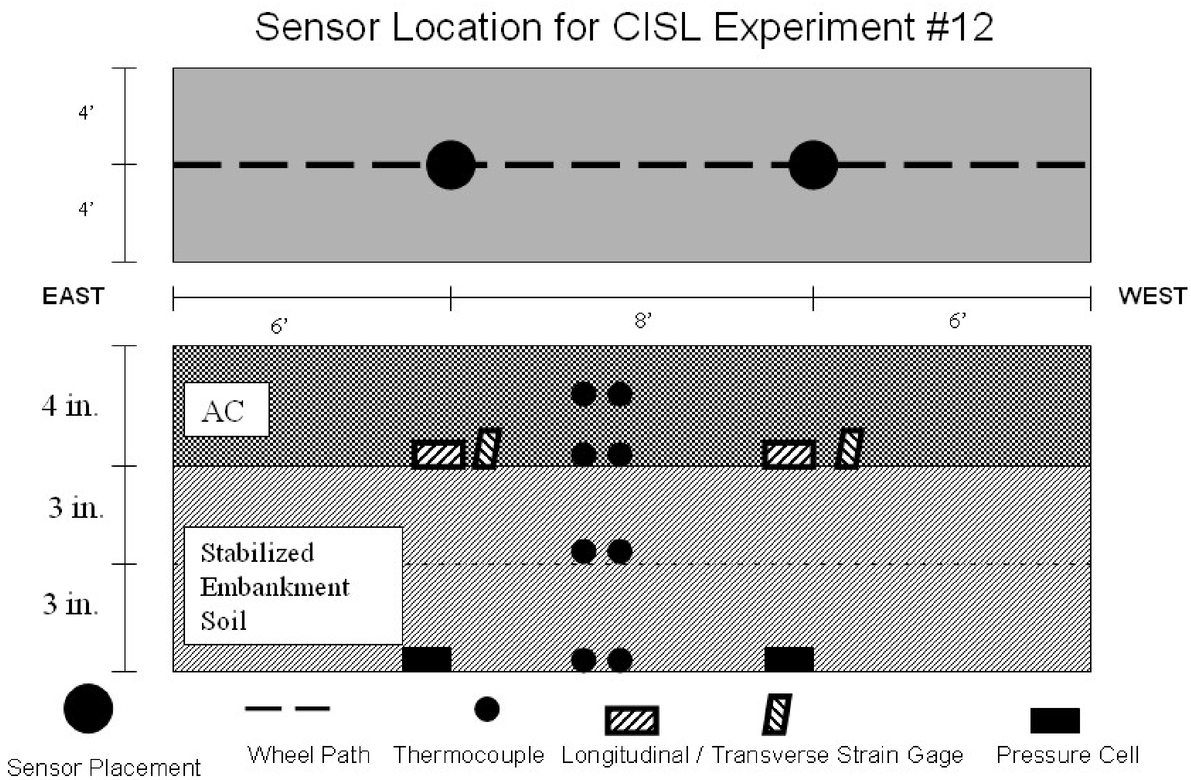


Figure 3.11: Location of Sensors Embedded in the Pavement Structure

3.3.2 Strain Gages

Strain gages were installed at the bottom of the asphalt surface layer to measure transverse and longitudinal tensile strains. In each section, four strain gages were installed on the centerline of the lane as shown in Figure 3.11. One gage was placed in the longitudinal direction and one in the transverse direction in the western part of the lane. Similarly one gage was placed in the longitudinal direction and one in the transverse direction in the eastern part of the lane.

The gages were constructed by attaching aluminum bars at the two ends of Tokyo Sokkai Kenkyujo (TML) strain gages. The H-Bars formed this way were fixed with short nails on top of the base layer after the layer was compacted, and before paving the asphalt concrete surface layer. During paving, asphalt mix was shoveled on top of the strain gages and the connection wires, and then lightly compacted to prevent deterioration of gages and wires during the paving operation. Five out of total sixteen gages were lost during construction. It was presumed that the gages became inoperable when the hot asphalt mix melted the connection wires of these gages.

3.3.3 Longitudinal Position of the ATL Load Assembly

A linear positioning gage, fixed to the East-North pole of the frame of the ATL machine, was used to record the longitudinal position of the ATL load assembly when the strain/pressure measurements were performed. The gage provided accurate measurement only when the load assembly was traveling West, away from the gage, since the cable of the linear positioning gage was properly stretched. When the load assembly was traveling toward the gage, the cable was not stretched to its entire length, and the readings were erroneous- somewhat higher than the true position of the load assembly. However, the use of the linear positioning gage for the measurement of longitudinal position of the ATL load assembly was abandoned in this experiment, and a new measuring system was installed.

The ATL load assembly position reading, horizontal strains at the bottom of the asphalt surface layer and vertical stress at the top of the subgrade were taken at a frequency of 100Hz by the same data acquisition system. The use of a single data acquisition system allowed all recording to be recorded on the same time basis in a single file.

3.3.4 Thermocouples

Four thermocouples were placed in each pavement structure, in the center location of each lane as shown in Figure 3.11. Two sensors were placed at the bottom of the asphalt layer (3 inches from the surface) and two at the bottom of the base layer.

The thermocouples were manufactured in-house and their precision was verified before installation. Similar thermocouples were used in previous ATL experiments and produced acceptable results when compared to other conventional temperature measurement devices. Temperature readings were taken monthly.

3.3.5 Falling Weight Deflectometer (FWD) Testing

FWD testing was performed by KDOT personnel at two different time periods:

- On the newly constructed pavements, on January 24, 2003. APT loading was started on the ATL sections in March 2003.
- On three of the four sections on November 17, 2003

The FWD tests were performed at six stations on each test lane as shown in Figure 3.12. For stations 1, 2 and 3 the geophones were oriented toward the East. For stations 4, 5 and 6 the geophones were oriented toward the West. Stations 3 and 4 were at the same location, in the center of the lane, but the geophones were directed to the East for station 3 and to the West for station 4.

Strain and pressure measurements were performed when the FWD loads were dropped at stations 1 and 6 to investigate if predicted strains, computed using the backcalculated elastic moduli, matched the measured strains. Stations 2 and 5 were added to stations 1 and 6 to investigate the effect of load position on the strain magnitudes. Therefore, stations 2 and 5 were selected six inches off the center of the lane from stations 1 and 6, respectively.

The FWD testing sequence consisted of three drops at 6,000 lbs load level followed by five drops at 9,000 lbs load level. The seven geophones were placed at: 0.0, 8.0, 12.0, 18.0, 24.0, 36.0 and 60.0 inches from the center of the FWD loading plate. The deflections recorded for the last drop at 6,000-lb load level and the last two drops at the 9,000 lb-load level were used to back calculate the elastic moduli of the pavement layers. These drops were selected since their deflection measurements are the most reliable because the FWD loading plate has the optimum contact with the pavement surface.

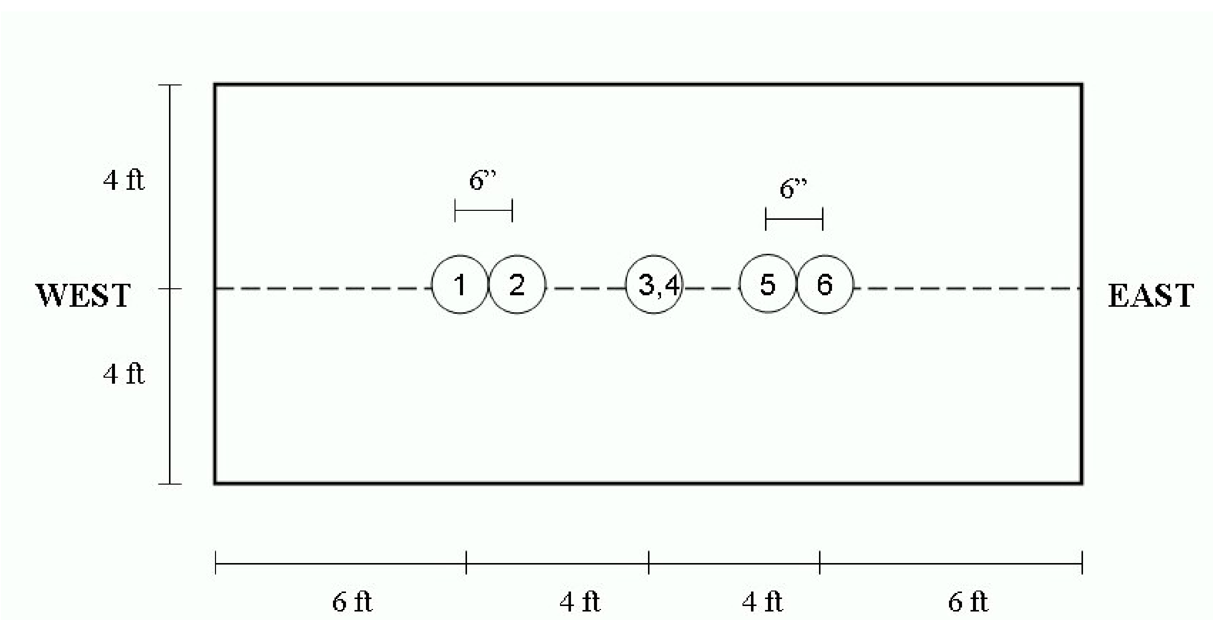


Figure 3.12: Location of the FWD Test Stations

3.3.6 Weight Drop Device

Weight drop tests were performed on the same day profile measurements were made at two stations for each lane. Figure 3.13 shows the station locations. Station W was at the West side of the lane; the tests were done with the Linear Variable Differential Transformer (LVDT) beam directed to the East. Station E was at the East side of the lane; the tests were done with the LVDT beam directed to the West.

The weight drop test consisted of dropping a weight of 60 lbs on a set of rubber plates that transmitted the load to a circular steel plate, nine inches in diameter. The plate was placed at the top of the pavement. The dynamic impact load was measured with a load cell under the rubber plates. The pavement surface deflections were measured by nine LVDTs fixed on a reference plastic beam oriented radially. The first LVDT is located at the center of the loading plate, and the remaining eight the following offsets from the center of the loading plate: 6, 12, 21, 30, 39, 48, 57 and 66 inches. The plastic beam holding the LVDTs was attached to the frame of the ATL machine so that it will not move when the weight was dropped.

The principle of the weight drop device is very similar to that of the FWD but the dropped weight, diameter of the loading plate and the spacing between the geophones are larger for the FWD. A typical load applied by the FWD is between 6,000 and 12,000 lbs, while the load applied by the weight drop device ranges between 2,000 and 2,500 lbs.

The vertical impact force and the seven sensor deflections were measured and recorded at a sampling frequency of 10,000Hz. The time traces of the load and deflections are recorded. For back calculation of layer moduli based on a linear elastic layered theory algorithm, only the maximum load and the maximum deflections were used.

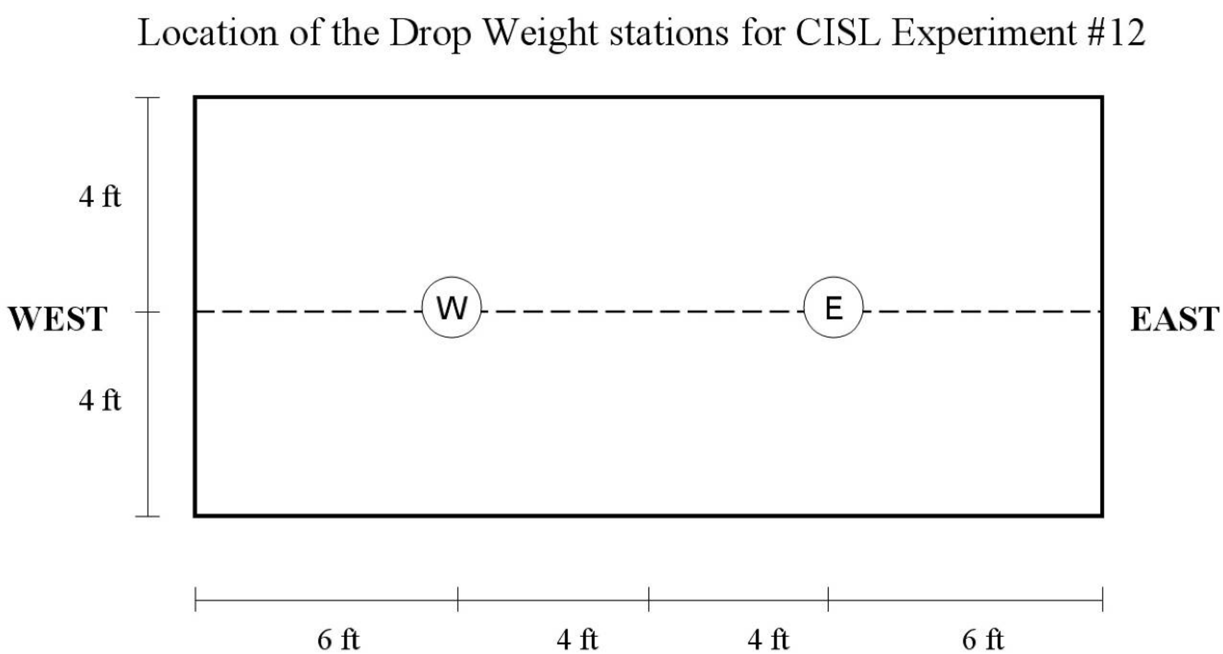


Figure 3.13: Location of Weight Drop Stations

3.4 Accelerated Pavement Testing Conditions

The test pavements were loaded in pairs using a tandem axle with dual wheels and a 30-kip (136.2 kN) load and a single axle with a 26-kip (118 kN). Accelerated loading was done in bi-directional mode, at a speed of about 7 mph (12 km/h). The lateral wander applied in this experiment followed a truncated normal distribution with a standard deviation of six inches (150 mm) and maximum wander of 12 inches (Figure 3.14). The tire inflation pressure was maintained at 100 psi (690 kPa) and was verified weekly. The dynamic wheel load was monitored with load cells installed on each wheel.

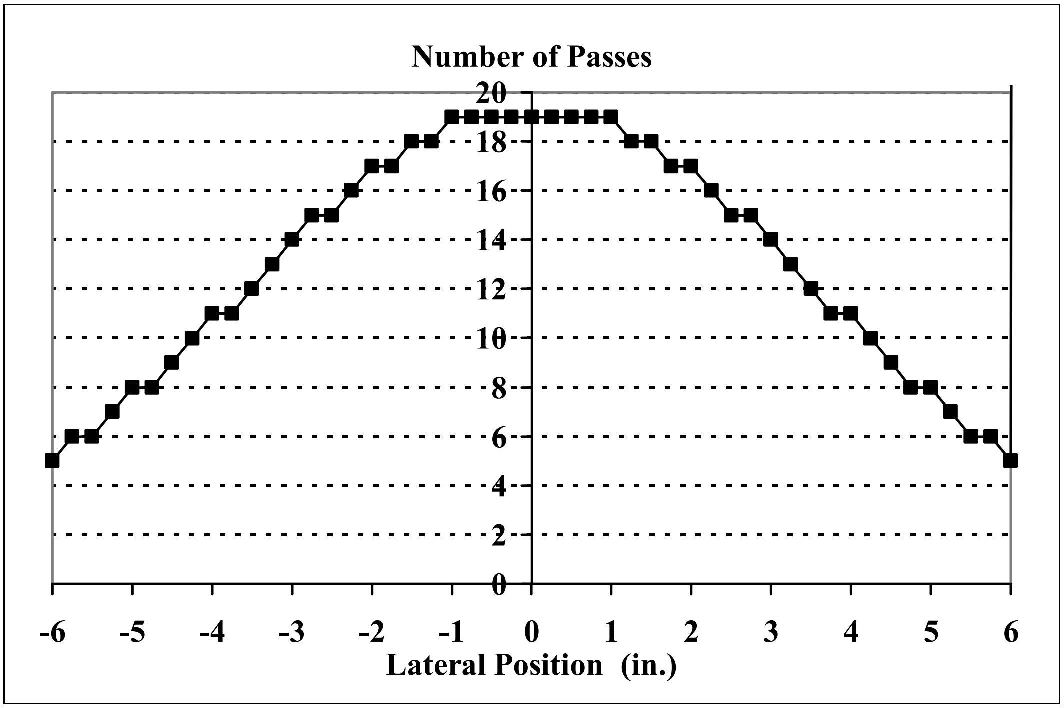


Figure 3.14: Distribution Function for the Lateral Wheel Wander

All testing was performed at ambient temperature. Thermocouples embedded in the pavement structure indicated that the four pavement sections were tested under very similar temperature regimes (Table 3.6, Figures 3.15 and 3.16).

Around 800,000 passes of the 30 kip tandem axle were applied to the pavement sections in the South pit (lime and commercial stabilizer). To the pavement sections in the north pit (Portland cement and Fly ash) 1,300,000 passes of the 30 kip tandem axle were applied followed by 700,000 passes of a 26 kip single axle. In order to be able to continue testing of the adjacent test section with lime, the failed HMA layer and two inches (50 mm) of stabilized embankment soil were removed. The remaining soil was recompacted. Two inches (50 mm) of sand were placed and compacted, and four inches (100 mm) of Portland cement concrete was poured, finished and cured for 28 days.

Loading was continued to a total of 800,000 load repetitions applied to the section with lime-treated soil subgrade while 1.3 million load repetitions were applied to the Portland cement and fly ash- treated sections. At this number of load repetitions, all three sections exhibited only rutting in the wheel path. Because no cracking or other distresses were observed, the performance comparison of these three sections could be done only based on permanent deformation and rut depth computed from the measured transverse profiles. An additional 700,000 load repetitions were applied to the pavements with fly-ash and cement treated embankment soils in January – March 2004 to induce fatigue failure and to compare the fatigue lives of these two pavements.

Table 3.6: Temperature Measured During Testing

DATE	At the Depth of 1.5 inches		At the Depth of 3 inches				At the Depth of 9 inches			
	NN & NS	SN & SS	NN	NS	SN	SS	NN	NS	SN	SS
4/30/2003	75.5	73.9	76.4	76.8	74.2	73.3	74.8	74.3	72.5	72.8
5/5/2003	78.1	75.7	74.3	74.9	80.7	85.2	72.9	72.4	76.6	83.3
5/28/2003	79.8	78.0	77.1	78.5	77.3	-	76.0	75.4	74.8	75.6
6/5/2003	77.7	76.2	78.4	79.0	77.1	-	78.1	78.2	76.0	76.4
6/30/2003	84.6	83.7	80.9	81.8	86.1	-	80.2	79.6	82.4	82.2
6/30/2003	84.5	83.5	80.8	81.8	86.1	-	80.2	79.6	82.4	82.3
7/10/2003	88.1	85.8	84.6	84.9	86.7	-	83.8	83.2	83.9	84.3
7/11/2003	87.7	85.7	85.4	85.5	87.2	-	84.6	83.8	86.0	86.2
7/22/2003	84.7	83.6	87.7	87.6	84.9	-	87.3	87.4	86.2	86.3
8/1/2003	83.2	82.3	87.2	86.3	83.6	-	87.9	87.7	85.7	85.8
8/21/2003	91.6	90.8	91.3	91.2	90.2	-	91.1	91.1	89.8	90.2
8/29/2003	88.7	87.4	89.0	88.7	89.4	-	89.2	88.9	90.8	90.5
9/9/2003	83.2	82.2	83.4	83.2	83.0	-	83.9	84.1	83.7	83.6
9/18/2003	76.3	76.9	81.5	80.6	79.6	-	82.1	82.3	82.8	82.8
9/26/2003	77.4	76.5	77.5	77.5	78.3	-	77.7	78.0	79.2	78.6
10/6/2003	73.4	72.7	73.1	73.0	72.9	-	73.7	74.1	74.3	73.8
10/17/2003	67.0	67.7	70.1	68.2	67.7	-	71.7	72.4	73.3	70.3
11/11/2003	69.1	68.2	68.3	68.0	68.0	-	68.3	68.7	67.5	66.2
11/20/2003	69.1	68.3	70.1	69.8	68.4	-	70.3	70.7	68.4	67.6
12/11/2003	66.4	66.7	69.1	70.3	66.2	-	69.1	70.3	66.6	66.3
1/28/2004	65.8	64.9	68.9	69.4	64.7	-	68.5	68.9	65.2	65.0
2/6/2004	66.7	65.8	67.6	67.2	66.1	-	67.6	67.3	66.0	66.0
2/13/2004	66.6	65.7	69.2	69.0	65.6	-	69.5	69.4	66.2	66.1
3/5/2004	68.4	67.3	69.1	68.6	67.3	-	69.1	69.1	68.1	67.0
3/16/2004	68.4	67.2	71.8	71.4	68.5	-	71.8	71.8	69.3	68.2

No water was added to the pavements. Since the pavements were constructed in pit and the asphalt concrete surface layer was paved wall-to-wall, the moisture content in the subgrade soil remained relatively constant during the accelerated testing. However, the volumetric moisture contents measured by the TDR gages, reported in Table 3.7, indicated that the values were significantly higher for the SN and SS pavements.

Table 3.7: Moisture Content (Volumetric) in the Subgrade Soil During Testing

Date	NN	NS	SN	SS
02/03/2003	16.67		13.7	13.16
05/02/2003	17.75		12.62	14.51
05/28/2003	17.48		13.97	13.43
06/30/2003	16.94		13.16	14.78

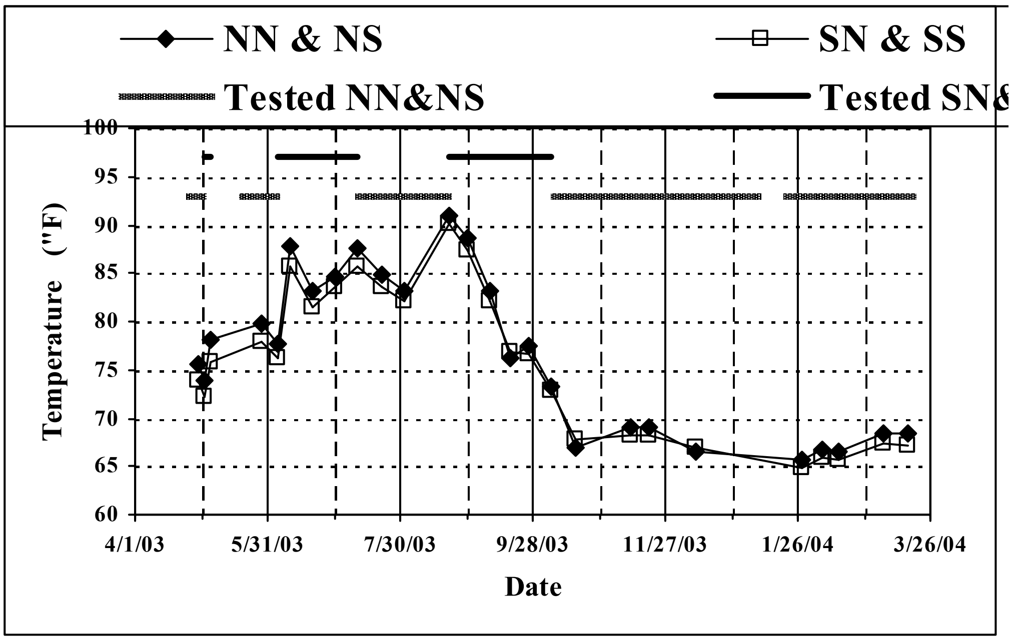


Figure 3.15: Temperature Measured at a Depth of 1.5 inches

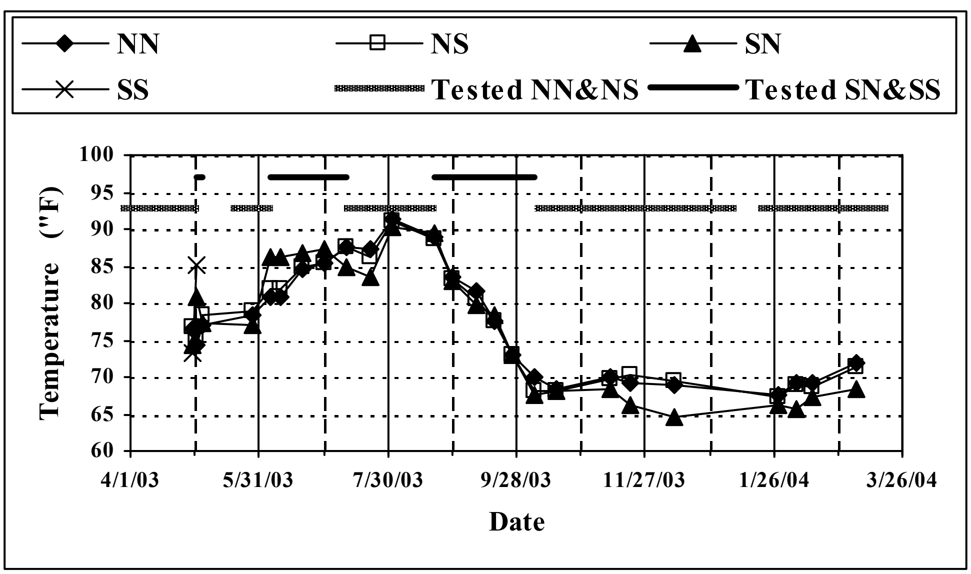


Figure 3.16: Temperature Measured at a Depth of 3.0 inches

3.5 Operating Schedule and Recording of Data

Table 3.8 shows the operating schedule of the project, when test data was collected. In January 2004, upgrades to the ATL machine caused delays in the planned operating schedule.

Table 3.8: Summary of Loading and Data Acquisition dates

DATE	PASSES	PIT	REMARKS
04-25-03	0	North	Start loading with tandem axle
05-02-03	100,000	North	100K data
05-02-03	0	Center	Start loading
05-05-03	45,131	Center	South Lane Failure (EMC^2)
05-19-03	100,000	North	Resume loading
05-27-03	200,000	North	200k data
06-05-03	300,000	North	300k data
06-05-03	45,131	Center	Restart Loading
06-20-03	200,000	Center	200k data
07-11-03	300,000	Center	300k data
07-11-03	300,000	North	Restart loading
07-21-03	400,000	North	400k data
08-01-03	500,000	North	500k data
08-11-03	600,000	North	600k data
08-21-03	700,000	North	700k data
08-21-03	300,000	Center	Restart Loading
08-29-03	400,000	Center	400k data
09-09-03	500,000	Center	500k data
09-18-03	600,000	Center	600k data
09-26-03	700,000	Center	700k data
10-06-03	800,000	Center	800k data
10-07-03	700,000	North	Restart Loading
10-17-03	800,000	North	800k data
10-31-03	900,000	North	900k data
11-11-03	1,000,000	North	1000k data
11-17-03	1,050,000	North	FWD data
11-20-03	1,100,000	North	1100k data
12-01-03	1,200,000	North	1200k data
12-09-03	1,300,000	North	1300k data
01-08-04	1,385,000	North	Single axle upgrade and data
01-20-04	1,385,000	North	Resume loading
01-28-04	1,485,000	North	1485k data
02-04-04	1,585,000	North	1585k data
02-13-04	1,685,000	North	1685k data
02-24-04	1,785,000	North	1785k data
03-05-04	1,885,000	North	1885k data
03-19-04	2,000,000	North	2000k data – End Loading
05-02-04			Post-Mortem Evaluation
05-11-04			Post-Mortem Evaluation

** Data taken consisted of strain gage readings, load, position, soil pressure readings, transverse and longitudinal profiles, and drop weight data.*

CHAPTER 4 - TEST RESULTS AND OBSERVATIONS

4.1 Transverse Profiles

Transverse profile measurements were performed periodically, at the same time with the longitudinal profile, strain/stress and weight drop measurements (Table 3.8). On each pair of pavements, transverse profiles were measured at three different spatial locations: at the middle of the lane, five feet West from the middle, and five feet East of the mid location. Each profile consists of elevation data at 210 points spaced at 0.5 in. intervals. For each profile, two steel balls were glued to the pavement in locations not trafficked by the ATL machine, at transverse position of 36 and 72 inches. The steel balls were used as reference since their elevations did not change during the entire experiment. The movement of these balls was checked every time profile measurements were made using a reference elevation point at the base of the steel pole near the East gate of the CISL laboratory.

Figure 4.1 illustrates two typical transverse profiles obtained from the elevation data on the NN and NS pavements. The initial profile is the profile measured before any ATL load was applied. The profile showing larger elevation variation is the profile after ATL passes have been made on the pavements. The ruts caused by the passage of the ATL load assembly at the pavement surface are clearly visible. Since no lateral wander was applied in this experiment, the ruts formed underneath each tire. Between the tires of the dual wheel, the asphalt concrete surface exhibited some heaving due to upward shoving of the materials.

Two major parameters were derived from the elevation data:

- *Permanent Deformation* at the pavement surface was calculated first in each of the 105 (210/2) points of the profile by subtracting measured elevation after a given number of ATL passes from the initial elevation data. The permanent deformation was positive when the current elevation of the point was lower than the initial elevation. Then, for each pavement, and for a particular transverse profile (West, Middle and East), the permanent deformation (PD) was computed as the maximum value obtained from the 105 points. The permanent deformation data is reported in Tables 4.1 and 4.2.
- *Rut Depth* (RD) for each pavement, and for a particular transverse profile (West, Middle and East), was computed as the difference between the elevation of the highest and lowest points of that profile. The rut depth data is reported in Tables 4.3. and 4.4.

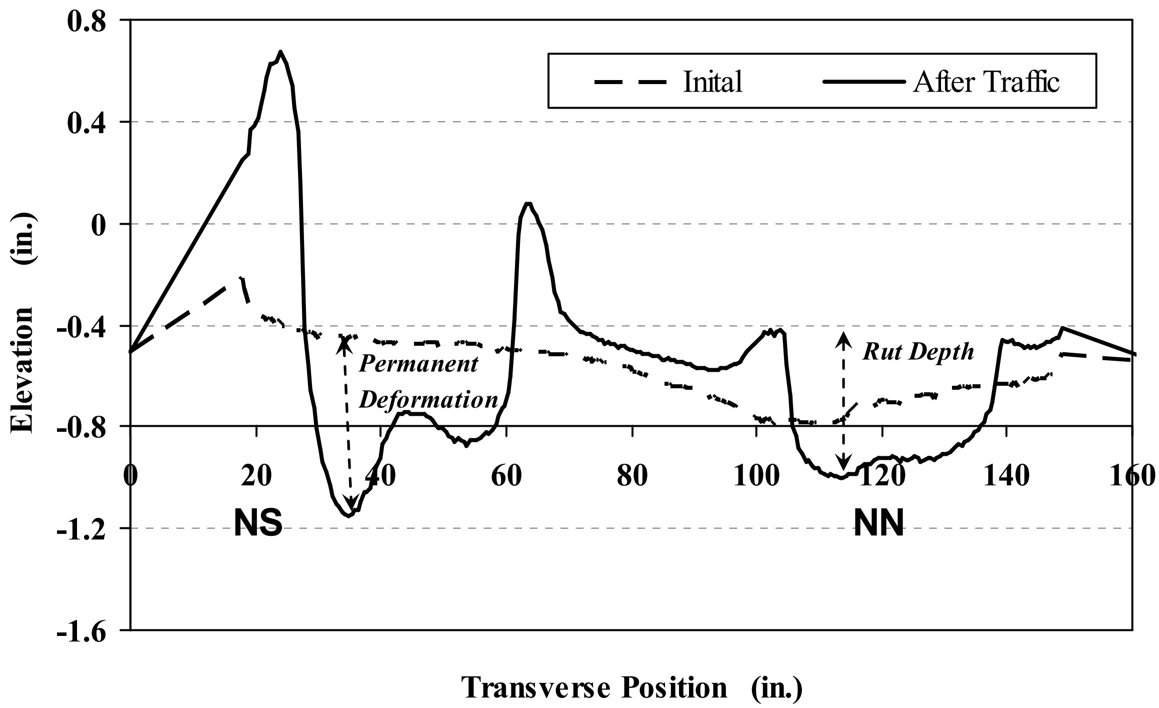


Figure 4.1: Example of Transverse Profile

The evolution of permanent deformation with the number of ATL load assembly passes is plotted in Figure 4.2., while the evolution of rut depth with the number of applied ATL load assembly passes is plotted in Figure 4.3. Figures 4.2 and 4.3 indicate that the highest rut depth and permanent deformation values were recorded for the pavement with fly ash-treated soil. The permanent deformation and rut depth values were similar for the pavements with cement and lime treated-soil subgrades. However, hydrated lime seems to be a more effective stabilizer since the HMA layer thickness determined with the rod-and-level method (Table 3.5), as well as those measured during the post-mortem investigation, was smaller for section SN than for section NN (Figures 4.19 and 4.20).

Table 4.1: Evolution of Permanent Deformation (in.) - Lanes NN and NS

Date	Passes (x 1,000)	NN-W	NN-M	NN-E	NN-Avg	NS-W	NS-M	NS-E	NS-Avg
2/3/2003	0	0.000	0.000	0.000	**0.000**	0.000	0.000	0.000	**0.000**
5/2/2003	100	0.085	0.114	0.099	**0.099**	0.059	0.114	0.111	**0.095**
5/27/2003	200	0.131	0.138	0.118	**0.129**	0.114	0.174	0.132	**0.140**
6/5/2003	300	0.162	0.183	0.148	**0.164**	0.145	0.256	0.205	**0.202**
7/21/2003	400	0.220	0.208	0.208	**0.212**	0.199	0.364	0.262	**0.275**
8/1/2003	500	0.236	0.232	0.246	**0.238**	0.235	0.477	0.328	**0.347**
8/11/2003	600	0.233	0.191	0.258	**0.228**	0.229	0.521	0.324	**0.358**
8/21/2003	700	0.263	0.259	0.270	**0.264**	0.261	0.637	0.365	**0.421**
10/17/2003	800	0.277	0.255	0.267	**0.266**	0.272	0.656	0.354	**0.428**
10/31/2003	900	0.270	0.254	0.304	**0.276**	0.271	0.637	0.376	**0.428**
11/11/2003	1000	0.282	0.263	0.282	**0.276**	0.274	0.662	0.394	**0.443**
11/11/2003	1100	0.290	0.260	0.278	**0.276**	0.274	0.668	0.385	**0.442**
12/1/2003	1200	0.272	0.241	0.279	**0.264**	0.260	0.635	0.379	**0.425**
12/9/2003	1300	0.289	0.257	0.281	**0.276**	0.264	0.647	0.381	**0.431**
1/8/2004	1385	0.257	0.262	0.283	**0.267**	0.253	0.667	0.396	**0.439**
1/28/2004	1485	0.271	0.261	0.243	**0.259**	0.262	0.665	0.377	**0.434**
2/5/2004	1585	0.273	0.265	0.274	**0.271**	0.249	0.672	0.390	**0.437**
2/13/2004	1685	0.296	0.265	0.274	**0.278**	0.266	0.674	0.389	**0.443**
2/24/2004	1785	0.297	0.264	0.275	**0.279**	0.268	0.683	0.396	**0.449**
3/5/2004	1885	0.307	0.264	0.273	**0.281**	0.282	0.682	0.406	**0.457**
3/16/2004	2000	0.287	0.266	0.272	**0.275**	0.263	0.704	0.409	**0.459**

Table 4.2: Evolution of Permanent Deformation (in.) - Lanes SN and SS

Date	Passes (x 1,000)	SN-W	SN-M	SN-E	SN-Avg	SS-W	SS-M	SS-E	SS-Avg
4/25/2003	0	0.000	0.000	0.000	**0.000**	0.000	0.000	0.000	**0.000**
5/7/2003	45	0.115	0.115	0.166	**0.132**	0.502	0.418	0.294	**0.405**
6/20/2003	200	0.167	0.179	0.215	**0.187**				
7/11/2003	300	0.222	0.205	0.226	**0.218**				
8/29/2003	400	0.224	0.208	0.251	**0.228**				
9/9/2003	500	0.234	0.218	0.240	**0.231**				
9/18/2003	600	0.257	0.239	0.263	**0.253**				
9/26/2003	700	0.265	0.233	0.287	**0.262**				
10/6/2003	800	0.259	0.252	0.292	**0.268**				

Table 4.3: Evolution of Rut Depth (in.) - Lanes NN and NS

Date	Passes (x 1,000)	NN-E	NN-M	NN-W	NN-Avg	NS-E	NS-M	NS-W	NS-Avg
2/3/2003	0	0.000	0.000	0.000	**0.000**	0.000	0.000	0.000	**0.000**
5/2/2003	100	0.214	0.204	0.146	**0.188**	0.135	0.234	0.177	**0.182**
5/27/2003	200	0.272	0.228	0.275	**0.259**	0.192	0.328	0.227	**0.249**
6/5/2003	300	0.344	0.315	0.240	**0.299**	0.267	0.480	0.321	**0.356**
7/21/2003	400	0.514	0.462	0.381	**0.452**	0.428	0.843	0.538	**0.603**
8/1/2003	500	0.555	0.524	0.434	**0.504**	0.500	1.102	0.650	**0.751**
8/11/2003	600	0.619	0.535	0.464	**0.539**	0.542	1.332	0.730	**0.868**
8/21/2003	700	0.659	0.598	0.495	**0.584**	0.593	1.660	0.916	**1.057**
10/17/2003	800	0.671	0.595	0.495	**0.587**	0.583	1.675	0.911	**1.056**
10/31/2003	900	0.656	0.603	0.510	**0.590**	0.587	1.672	0.920	**1.060**
11/11/2003	1,000	0.673	0.601	0.502	**0.592**	0.578	1.686	0.938	**1.068**
11/11/2003	1,100	0.685	0.606	0.494	**0.595**	0.590	1.697	0.941	**1.076**
12/1/2003	1,200	0.679	0.609	0.511	**0.600**	0.587	1.692	0.935	**1.071**
12/9/2003	1,300	0.681	0.603	0.495	**0.593**	0.580	1.686	0.931	**1.066**
1/8/2004	1,385	0.694	0.600	0.502	**0.599**	0.591	1.700	0.941	**1.077**
1/28/2004	1,485	0.698	0.605	0.495	**0.599**	0.590	1.710	0.942	**1.081**
2/5/2004	1,585	0.723	0.614	0.502	**0.613**	0.597	1.723	0.955	**1.092**
2/13/2004	1,685	0.722	0.619	0.500	**0.614**	0.595	1.734	0.959	**1.096**
2/24/2004	1,785	0.737	0.619	0.505	**0.620**	0.598	1.736	0.961	**1.098**
3/5/2004	1,885	0.737	0.620	0.506	**0.621**	0.608	1.754	0.971	**1.111**
3/16/2004	2,000	0.761	0.624	0.505	**0.630**	0.622	1.772	0.990	**1.128**

Table 4.4: Evolution of Rut Depth (in.) - Lanes SN and SS

Date	Passes (x 1,000)	SN-E	SN-M	SN-W	SN-Avg	SS-E	SS-M	SS-W	SS-Avg
4/25/2003	0	0.000	0.000	0.000	**0.000**	0.000	0.000	0.000	**0.000**
5/7/2003	45	0.203	0.168	0.148	**0.173**	0.835	0.540	0.299	**0.558**
6/20/2003	200	0.447	0.331	0.258	**0.345**				
7/11/2003	300	0.562	0.393	0.306	**0.420**				
8/29/2003	400	0.737	0.500	0.385	**0.541**				
9/9/2003	500	0.798	0.538	0.395	**0.577**				
9/18/2003	600	0.825	0.527	0.400	**0.584**				
9/26/2003	700	0.841	0.529	0.427	**0.599**				
10/6/2003	800	0.845	0.542	0.419	**0.602**				

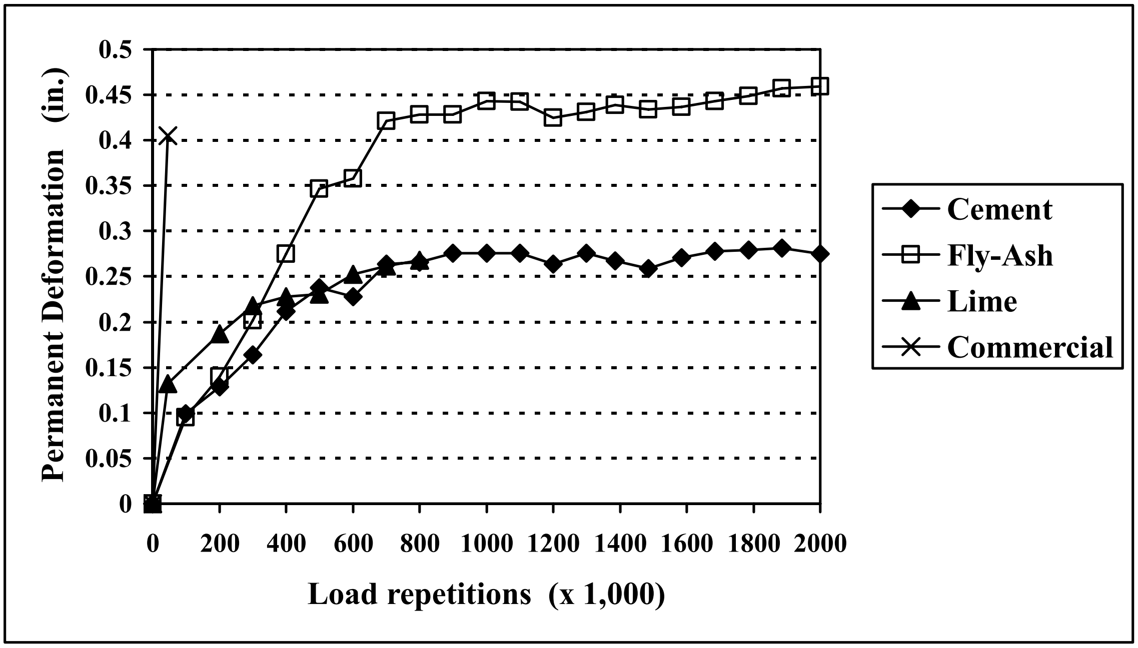


Figure 4.2: Evolution of Permanent Deformation

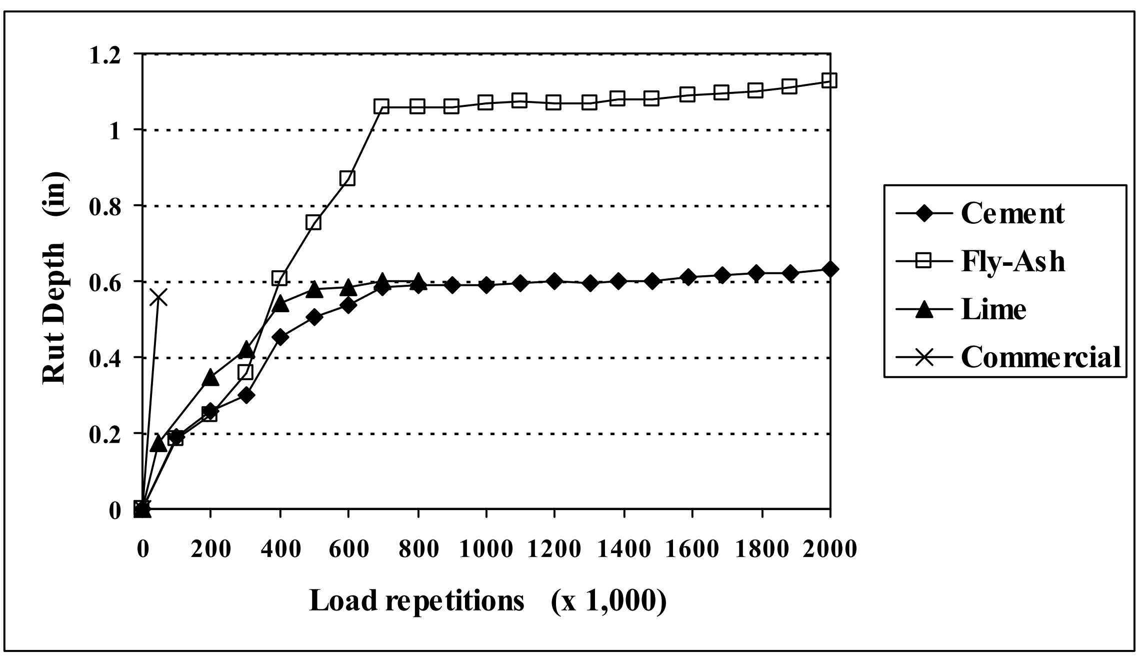


Figure 4.3: Evolution of Rut Depth

4.2 Longitudinal Profiles

The longitudinal profile of a pavement section was recorded by measuring the elevation of 19 points spaced at one-foot intervals on the outside wheel path with surveying equipment. The points were numbered from East to West, with the first point being at one foot West of the East wall of the pit. A fixed point at the base of a steel pole near the East gate of the CISL laboratory was used as reference. The longitudinal profile data is reported in Appendix C.

The roughness of the longitudinal profile was estimated from the elevation data, with the Slope Variance (SV) as the roughness statistic. SV was selected for this project because of its simplicity. Other indexes that are computed based on elevation data require a minimum length of pavement section. For example, to compute the International Roughness Index (IRI), the road section must be at least 33 feet (11 meters) long. The slope variance (SV) can be computed as:

$$SV = [\ SUM\ (S_i - S_{avg})^2\ ] / (N-1) \qquad (4.1)$$

Where :

N – number of segments where the slope is computed (N=18 for the CISL sections);

$S_i = 100*(h_{i+1} - h_i) / d$ - slope in point i, in percent;

h – elevation (in); and

d – spacing between points (d = 12 in).

It is important to note that the roughness statistic derived from the longitudinal profile is not a good indicator of pavement performance for the 20-ft long pavement sections subjected to full scale accelerated testing at CISL, and it does not correlate well with the roughness of in-service pavement. The main reason is that the variability of material properties and layer thickness are different for such a short section than for an in-service pavement. Also, the environmental factors, which are affect pavement performance, are carefully controlled in CISL.

However, the slope variance was computed here only to compare its evolution for the four pavement structures under study. The Slope Variance values are reported in the Appendix C.

Figure 4.4 shows the evolution of Slope Variance and clearly indicates that the SV values did not change with the number of accumulated ATL load assembly passes, with the exception of the section with the embankment soil treated with EMC-Squared. This can be explained by the very uniform dynamic loading provided by the ATL machine to the remaining three sections.

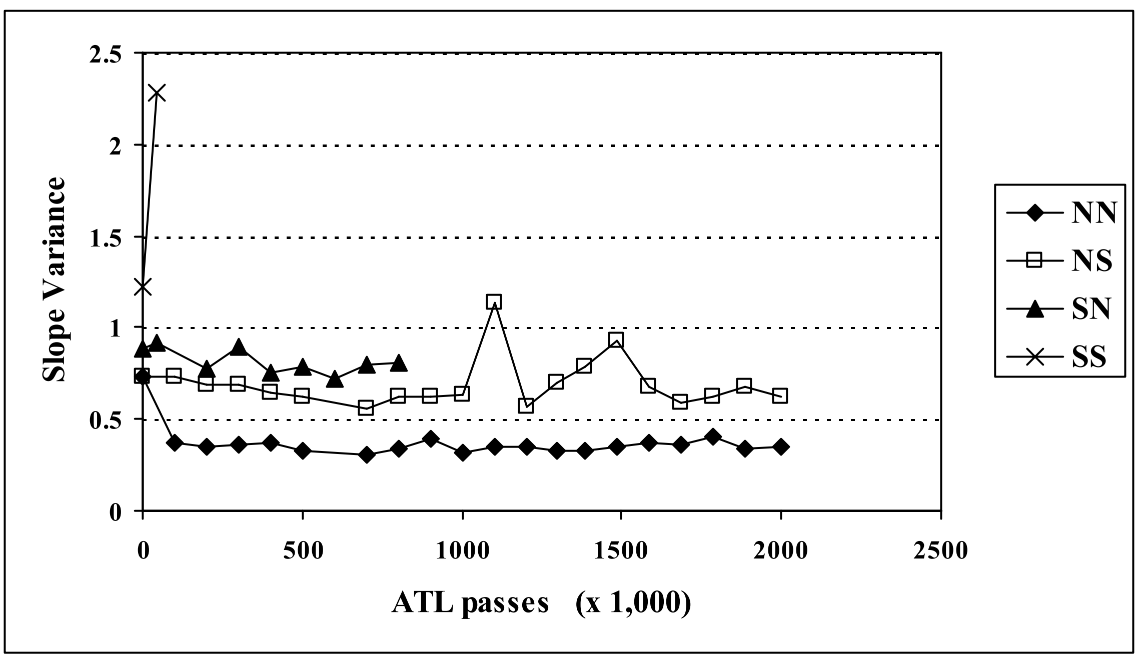


Figure 4.4: Evolution of Roughness

4.3 Fatigue Cracking

The pavement structure with the embankment soil treated with the commercial product, EMC-Squared, failed after 45,000 passes of the 30-kip tandem axle, due to lack of sufficient support underneath the hot-mix asphalt surface layer. Severe cracking and rutting developed, as shown in Figures 4.5 and 4.6. The treated soil was placed in the pit and compacted at a moisture content of 23%, which is the optimum moisture content for the standard Proctor tests. The material was compacted at more than 95% standard proctor maximum density. This was done in agreement with the manufacturer of the commercial product so that the performance comparison with the other three stabilizers could be done at the similar compaction energy. However, it is to be noted that for paving projects, the commercial stabilizer manufacturer recommends that the mix design and compaction be done based on the modified Proctor test results, to achieve a stiffer and denser material.

In order to be able to continue loading on section SN, the severely distressed pavement in section SS was removed. The asphalt concrete layer and about three inches of the stabilized soil were excavated. New soil was placed and compacted in the SS section. A four inch Portland cement concrete was placed on top of the soil (Figure 4.7). The concrete was cured for more than 28 days before accelerated loading was continued on section SN. No cracking was observed on the SN section after a total of 800,000 passes were applied. Loading was not continued on this section due to financial and time constraints.

As previously indicated, after 800,000 load repetitions of the 30-kip (136.2 kN) tandem axle were applied to the pavement with lime-treated soil embankment and 1.3

million load repetitions of the 30-kip (136.2 kN) tandem axle applied to the pavements with Portland cement and fly ash-treated embankments, these three sections exhibited only rutting in the wheel path. An additional 700,000 load repetitions of the 26-kip single axle were applied to the pavements with fly-ash and cement treated embankment soils in January – March 2004 to induce fatigue cracking failure. Figures 4.8 and 4.9 show the crack pattern observed on these two pavements after two million load repetitions of single and tandem axle were applied. The figures indicate that more severe and extensive fatigue cracking developed in the pavement structure with fly-ash treated embankment soil than in the pavement with cement treated soil. This suggests that, for the clayey soil employed in this study, cement is a more effective stabilizer than fly-ash. No conclusion based on comparison of cracking development can be drawn between lime stabilization and cement stabilization of the studied soil, since the lime-treated section was subjected to only 800,000 load repetitions.

Figure 4.5: Severe rutting measured on the SS sections at 45,000 load cycles

Figure 4.6: Severe rutting and cracking on the SS sections at 45,000 load cycles

Figure 4.7: Placement of PCC layer on the distress section SS

Figure 4.8 Surface cracks on the NN and NS sections at 2,000,000 load cycles

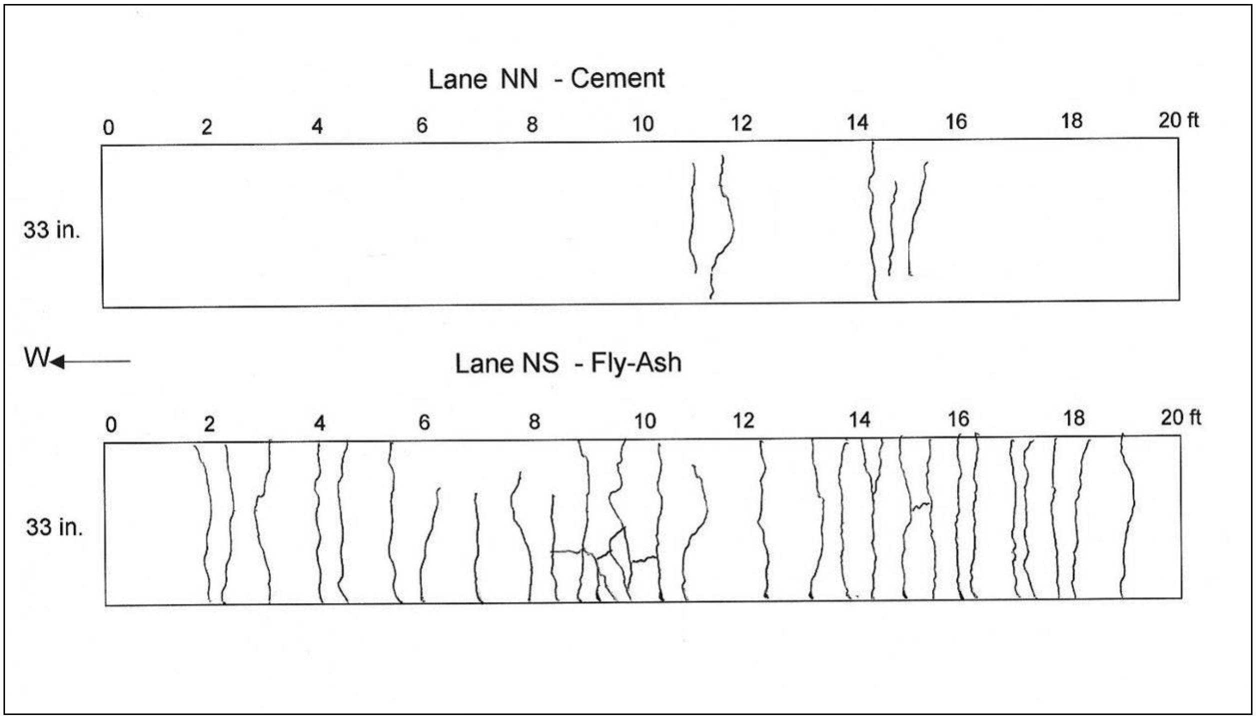


Figure 4.9: Surface crack pattern at 2,000,000 load cycles

4.4 Horizontal Strains at the Bottom of the Asphalt Concrete Layer

The strain and pressure values were recorded for at least four cycles (eight passes) of the ATL load assembly, at a sampling frequency of 100Hz. Recording was started when the ATL load assembly was at the West end of the travel and started traveling East. The strain measurements were performed for two lateral positions of the wheels:

- Position 0" – The symmetry axis of the wheel placed above the gages. In this position, the tires were straddling the gages, as shown in Figure 4.10.
- Position +6" - With one tire passing right above the strain gages. The symmetry axis of the wheel was 6 inches away transversely from the gages (Figure 4.10).

The stress and strain data was stored in the same electronic file, in a spreadsheet format, along with the longitudinal position of the loading bogie. Figure 4.11 presents the six typical shapes of the strain signal that were observed for a complete ATL test cycle (from the time the load assembly leaves and arrives at the West end of travel position). The values A to E recorded on the strain signals are given in Appendix C.

As mentioned earlier, five strain gages failed during placement of the asphalt concrete layer, possibly due to high temperature of the asphalt mixture during placement. The failed gages are:

- Both gages measuring longitudinal strain in section NN; the gages measuring longitudinal strain in section NS- the East side, and SS – the West side.
- Both gages measuring transverse strain in section SS, the gages measuring transverse strain in section NN- the West side, and SN – the East side.

From the recorded strain signals, two typical signal shape types were identified. Typical strain signal shape 1 was observed when the single axle loads were applied to the pavement structures, while typical strain signal type 2 was observed when tandem axle loads were applied to the pavement structures. The strain values (S) were computed with the following formulas:

Shape type 1: $S = (A+C)/2 - B$

Shape type 2: $S = (A + B + D + E)/4 - C$

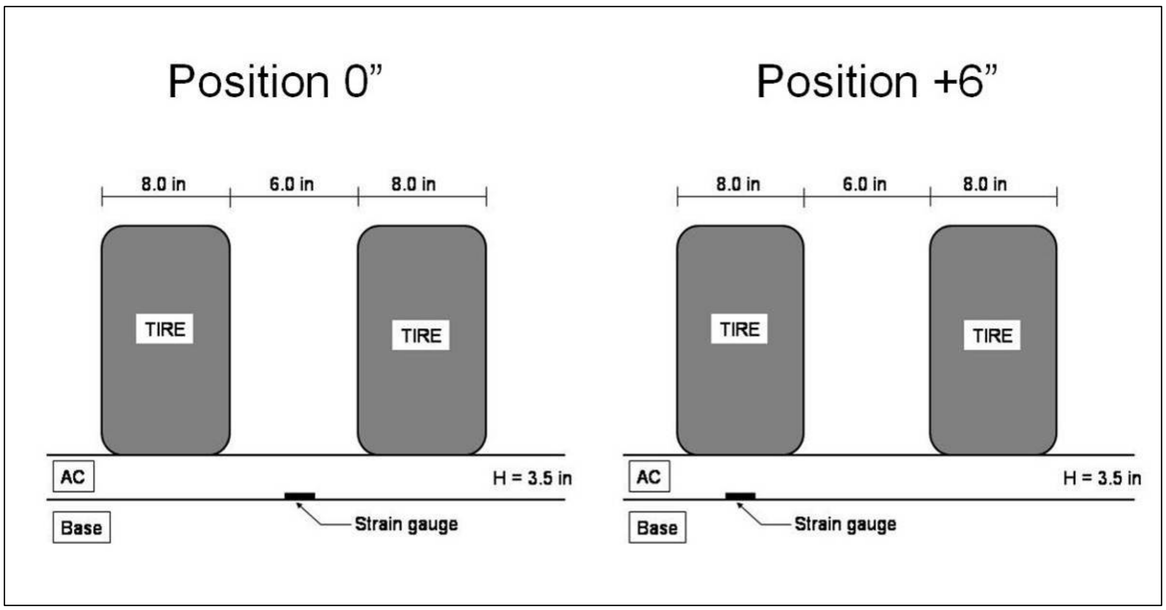


Figure 4.10: Position of the Wheel during Strain Measurements

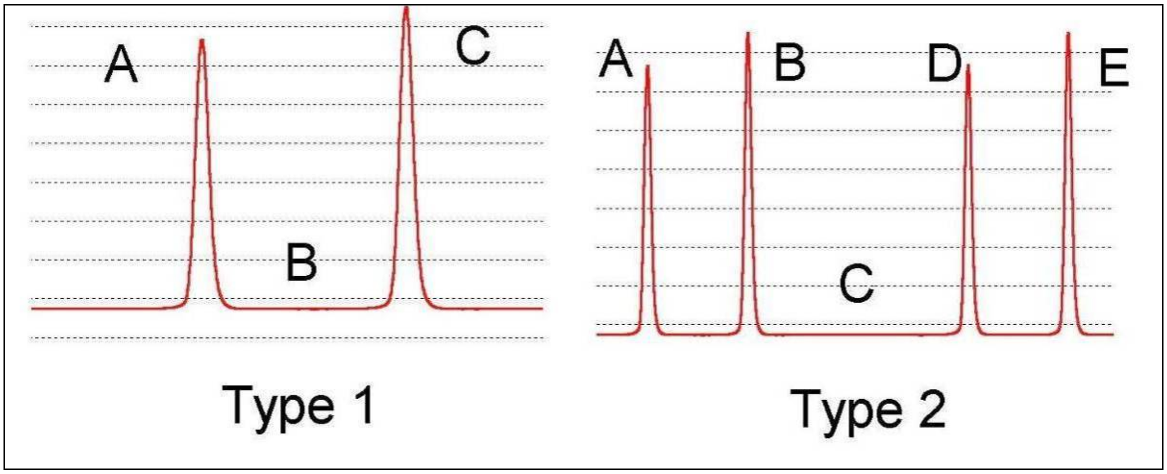


Figure 4.11: Types of Strain Signal Shapes

In many cases, the strain gages provided atypical signals, which were very difficult to analyze, even though the strain gages functioned properly. Most of the signals recorded when the tandem axle passed above the gages in position 0" fell in this group. A possible reason for the occurrence of these atypical signals is that the chemically stabilized embankment layers had stiffnesses comparable and, in many instances higher, than that of the asphalt concrete layer. Therefore, the neutral plane was located close to the strain gages. In addition to this, the thickness of the asphalt concrete layer (3.5") was almost half the distance between the walls of the dual tires. Therefore the strains measured by the gages were small, and changed sign when the wheel was in position0" and +6".

The values of measured horizontal strains at the bottom of the asphalt concrete layer obtained from typical signals are given in Appendix C. Tables 4.5 and 4.6 tabulate the average values of the measured strains, when both gages measuring the same strain (longitudinal or transverse) on the same pavement sections were recorded.

Table 4.5: Longitudinal Strains at the Bottom of the Asphalt Concrete Layer

Test Section	Date	Passes (x 1,000)	Wheel Position	
			0"	+6"
NS	25-Mar-03	0	244.2	
SN	2-May-03	0		22.4
SN	5-May-03	45		100.9
SS	2-May-03	0	154.7	137.1
SS	5-May-03	45	163	215.6

The average values of the measured transverse strains are plotted in Figures 4.12. The figure indicates that the transverse strains decrease with the increasing number of load repetitions. The figure also indicates that the transverse strains measured underneath a tire (wheel in position +6”) are higher than the strains measured when the wheel straddles the gages (wheel in position 0”). Among test sections with the soil stabilized with cement, fly-ash and lime, the highest transverse strain was recorded for the section with the fly-ash stabilized embankment soil. The lowest transverse strain was recorded for the section with the lime stabilized embankment soil.

Table 4.6: Transverse Strains at the Bottom of the Asphalt Concrete Layer

Test Section	Date	Passes (x 1,000)	Wheel Position 0"	+6"
NN	30-Apr-03	100		53.2
NN	28-May-03	200		67
NN	5-Jun-03	300		53
NN	21-Jul-03	400		134
NN	1-Aug-03	500		141
NN	21-Aug-03	700		92.9
NN	17-Oct-03	800	75.5	94.6
NN	31-Oct-03	900	74.5	89
NN	11-Nov-03	1,000		70.2
NN	20-Nov-03	1,100		92.2
NN	9-Dec-03	1,300	87.3	95.4
NN	28-Jan-04	1,485	74	100
NN	6-Feb-04	1,585	82.6	96.5
NN	13-Feb-04	1,685	87.3	94.3
NN	24-Feb-04	1,785	65.7	115.4
NN	5-Mar-04	1,885	89.9	104
NN	16-Mar-04	2,000		100.7
NS	30-Apr-03	100		218.4
NS	28-May-03	200		181.2
NS	5-Jun-03	300		224.5
NS	21-Jul-03	400		189
NS	1-Aug-03	500		164.8
NS	21-Aug-03	700		181
NS	17-Oct-03	800	33	127.4
NS	31-Oct-03	900	33.9	120
NS	11-Nov-03	1,000	27.3	114.8
NS	20-Nov-03	1,100	24.8	140.6
NS	9-Dec-03	1,300	33.1	126.3
NS	28-Jan-04	1,485	37.3	113.8
NS	6-Feb-04	1,585	38.3	129.5
NS	13-Feb-04	1,685	49.3	123.5
NS	24-Feb-04	1,785	31.6	118
NS	5-Mar-04	1,885	31.1	113.5
NS	16-Mar-04	2,000	32.8	104.6
SN	2-May-03	0	-79.2	15
SN	5-May-03	45	-146.1	135.8
SN	20-Jun-03	200	-109.6	122.6
SN	11-Jul-03	300	-59.6	110.5
SN	29-Aug-03	400		127.3
SN	9-Sep-03	500		126
SN	18-Sep-03	600		117.8
SN	26-Sep-03	700		113
SN	6-Oct-03	800		94.5

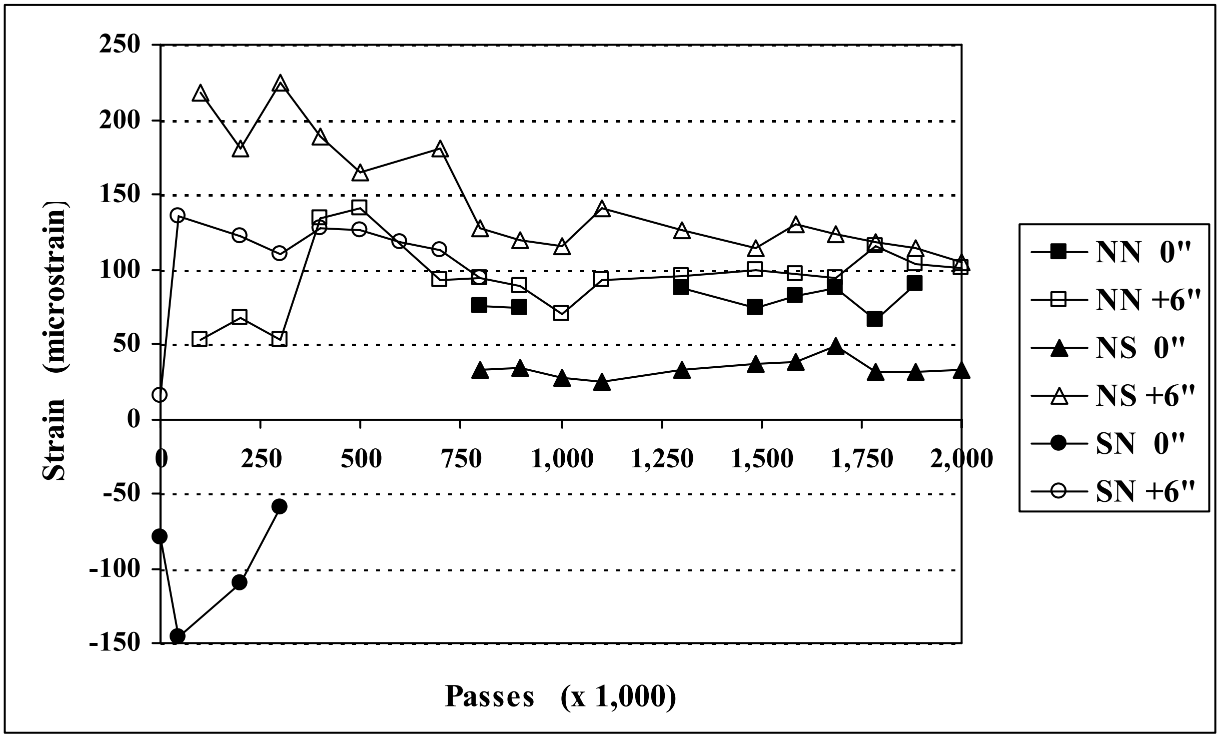


Figure 4.12: Transverse Strains at the Bottom of the Asphalt Concrete Layer

4.5 Vertical Stresses at the Top of the Subgrade

Vertical compressive stresses at the top of the subgrade were measured in each pavement structure at two locations (West and East) as shown in Figure 3.11. The stress measurements were performed on the dates indicated in the pavement monitoring plan given in Table 3.8.

The measured compressive stresses are reported in the Appendix D. The maximum values of the stresses measured by the two pressure cells in the same lane are reported in Table 4.7 and have been plotted in Figure 4.13. The higher of the two values was selected since, if a pressure cell does not make proper contact with the material above it, it records a lower stress than if proper contact is developed.

Figure 4.13 indicates that the highest vertical compressive stresses at the top of the subgrade were recorded at the beginning of the test for the test section with the embankment soil stabilized with EMC-Squared. The measured compressive stresses were higher for the test section with the embankment soil stabilized with fly-ash than for the test sections where the soil was stabilized with cement or lime. The lowest vertical stresses were recorded for the test section with the embankment soil stabilized with lime, suggesting that the lime stabilized layer protects the best the subgrade soil underneath.

The figure also indicates that the compressive stresses at the top of the subgrade measured underneath a tire (wheel in position +6”) are always lower than the stresses recorded measured when the wheel straddles the gages (wheel in position 0”).

Table 4.7: Maximum Vertical Compressive Stresses at the Top of Subgrade (psi)

Date	Passes (x 1,000)	Test Section NN Wheel Position 0”	Test Section NN Wheel Position +6”	Test Section NS Wheel Position 0”	Test Section NS Wheel Position +6”
25-Mar-03	0	5.9	5.9	13.6	13.6
30-Apr-03	100	5.0	4.6	9.7	7.8
28-May-03	200	6.7	6.0	9.5	8.9
5-Jun-03	300	6.8	5.0	9.6	8.1
21-Jul-03	400	7.7	6.9	10.9	10.2
1-Aug-03	500	7.7	6.4	11.4	10.3
21-Aug-03	700	8.5	7.2	13.1	11.7
17-Oct-03	800	7.8	5.7	12.7	10.2
31-Oct-03	900	7.7	6.1	12.9	11.2
11-Nov-03	1000	6.5	4.8	13.0	10.8
20-Nov-03	1100	7.7	5.8	13.0	10.8
9-Dec-03	1300	7.2	5.5	12.6	10.6
28-Jan-04	1485	9.9	8.6	16.2	14.6
6-Feb-04	1585	10.9	9.7	16.7	15.5
13-Feb-04	1685	10.9	10.1	16.6	15.8
24-Feb-04	1785	11.5	9.5	17.0	15.1
5-Mar-04	1885	12.1	11.1	17.7	16.4
16-Mar-04	2000	13.0	11.2	18.5	16.8
Date	Passes (x 1,000)	Test Section SN Wheel Position 0”	Test Section SN Wheel Position +6”	Test Section SS Wheel Position 0”	Test Section SS Wheel Position +6”
2-May-03	0	4.2	5.2	13.5	9.9
5-May-03	45	2.9	3.8	17.0	16.0
20-Jun-03	200	3.2	3.9		
11-Jul-03	300	3.0	3.8		
29-Aug-03	400	2.0	2.5		
9-Sep-03	500	2.4	2.3		
18-Sep-03	600	3.1	4.6		
26-Sep-03	700	3.0	3.4		
6-Oct-03	800	3.0	2.9		

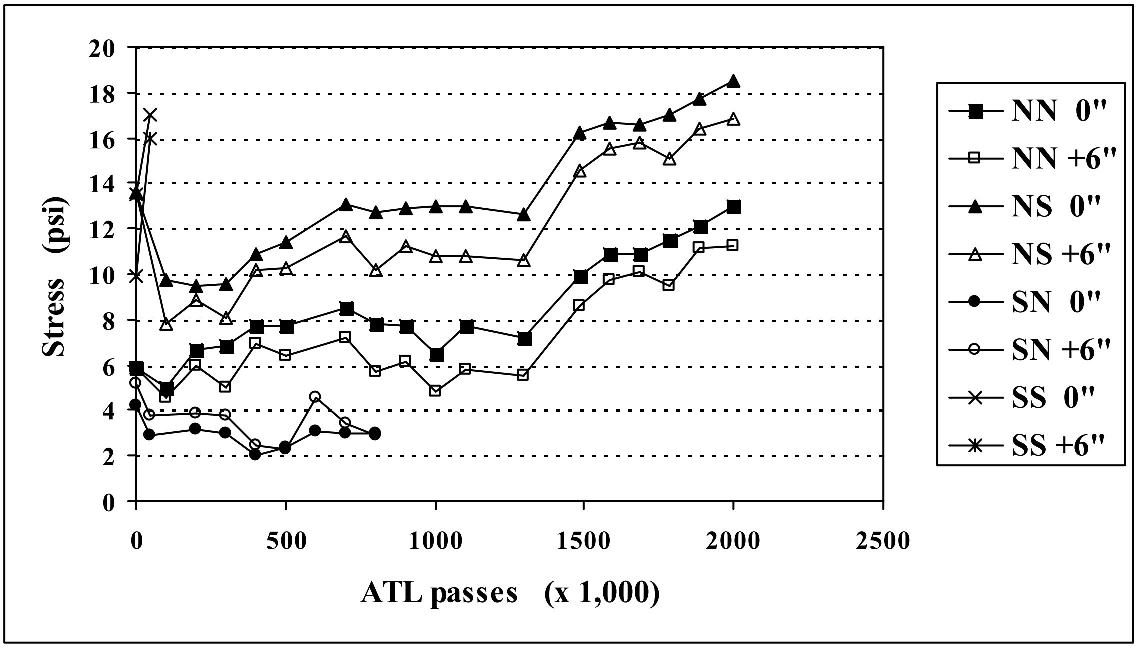


Figure 4.13: Maximum Vertical Stress at the Top of the Subgrade

4.6 Backcalculation of Layer Moduli from the FWD Deflections

The backcalculation analysis was performed using MODULUS 4.0 backcalculation program [16]. The measured FWD deflections along with the backcalculated layer moduli are reported in the Appendix E. In the backcalculation, the thicknesses obtained from the rod-and-level measurements in the same spot where the FWD tests were performed were employed (Table 3.5). The backcalculated asphalt layer moduli were not corrected to the standard temperature of 68°F, because the temperature at the bottom of the asphalt layer varied between 67°F and 72° F during FWD tests, close to the reference temperature of 68°F (Table 3.6). The backcalculated moduli for the last drop at the 9,000 lbs load level (Drop 3) are reported in Table 4.8. The average values of the backcalculated moduli are plotted for each pavement layer in Figures 4.14, 4.15 and 4.16, for both FWD test sessions.

Table 4.8 indicates that for each of the four test sections, the backcalculated asphalt layer moduli for the six FWD test stations exhibited large differences. Moduli are also quite different for the four pavement sections, despite the fact that the same HMA mix was used in paving. This large variation cannot be attributed to the asphalt layer thickness of the constructed pavements since the thicknesses obtained from the rod-and-level measurements in the same spot where the FWD tests were performed were employed in the backcalculation (Table 3.5).

Figure 4.15 and Table 4.8 suggest that the backcalculated modulus for the stabilized soil remained relatively unchanged after the application of the 1,100,000 passes of the ATL bogie. The backcalculated modulus of the lime stabilized soil was higher than the backcalculated moduli for the stabilized soil of the other three pavement sections.

Table 4.8 and Figure 4.16 indicate that, the backcalculated subgrade soil moduli before loading was applied was between to 12,000 and 15,000 psi for sections NN, NS and SN and only about 8,000 psi were obtained for the section SS. Because the same soil was placed in all pavements and using the same compaction process, the low value obtained for section SS must be the result of errors in the backcalculation process, which attributed a too high value to the moduli of the stabilized soil layer for section SS. This may be possible since the moduli backcalculation may not be accurate for flexible pavements with asphalt layer thickness smaller than 4 inches. After the application of the 1,100,000 passes of the ATL bogie, the modulus of the subgrade soil in the NN and NS sections dropped only about 10 percents.

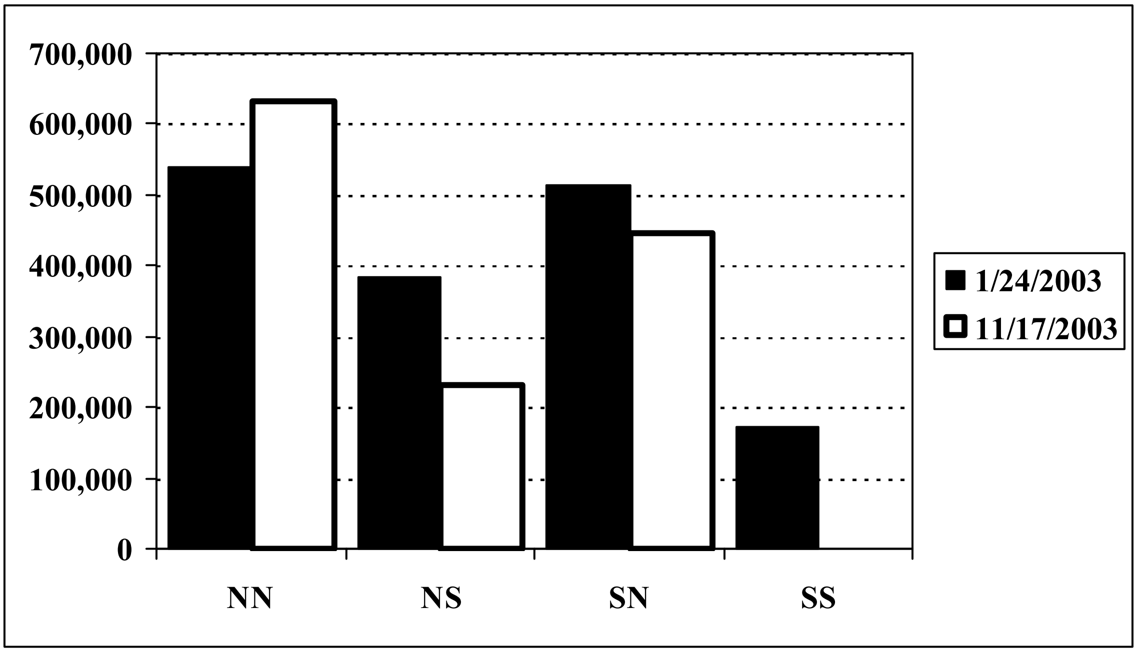


Figure 4.14: Average Backcalculated Asphalt Layer Modulus from FWD Deflections

Table 4.8: Backcalculated Moduli from the FWD deflections

Section	Date	Passes (x1,000)	Station	Drop Nr.	E(AC) (psi)	E(Base) (psi)	Mr subgrade (psi)
NN	1/24/2003		1	3	491,015	38,282	13,212
NN	1/24/2003		2	3	493,200	34,676	13,722
NN	1/24/2003		3	3	1,013,377	31,069	12,822
NN	1/24/2003		4	3	672,500	41,008	13,464
NN	1/24/2003		5	3	269,741	35,868	14,441
NN	1/24/2003		6	3	291,814	31,061	14,280
Average					***538,608***	***35,327***	***13,657***
NN	11/17/2003		1	3	498,990	28,601	11,728
NN	11/17/2003		3	3	975,944	33,655	11,476
NN	11/17/2003		4	3	736,296	48,387	11,352
NN	11/17/2003		6	3	315,545	41,955	12,777
Average					***631,694***	***38,150***	***11,833***
NS	1/24/2003		1	3	462,319	39,048	12,377
NS	1/24/2003		2	3	507,458	36,769	12,315
NS	1/24/2003		3	3	279,722	36,180	11,058
NS	1/24/2003		4	3	451,375	19,445	11,110
NS	1/24/2003		5	3	308,717	32,622	11,765
NS	1/24/2003		6	3	287,775	24,602	11,190
Average					***382,894***	***31,444***	***11,636***
NS	11/17/2003		1	3	335,921	24,665	9,993
NS	11/17/2003		3	3	201,997	19,830	9,875
NS	11/17/2003		4	3	168,563	20,883	10,495
NS	11/17/2003		6	3	219,249	35,632	6,564
Average					***231,433***	***25,253***	***9,232***
SN	1/24/2003		1	3	199,310	37,276	17,030
SN	1/24/2003		2	3	306,558	38,755	16,843
SN	1/24/2003		3	3	509,916	43,975	15,347
SN	1/24/2003		4	3	745,192	34,808	15,310
SN	1/24/2003		5	3	1,134,966	46,147	17,094
SN	1/24/2003		6	3	179,830	60,217	18,005
Average					***512,629***	***43,530***	***16,605***
SN	11/17/2003		1	3	328,930	45,205	12,341
SN	11/17/2003		3	3	505,170	50,008	13,253
SN	11/17/2003		4	3	546,655	41,583	13,747
SN	11/17/2003		6	3	402,210	40,142	15,463
Average					***445,741***	***44,235***	***13,701***
SS	1/24/2003		1	3	215,232	43,762	7,561
SS	1/24/2003		2	3	232,327	44,290	7,571
SS	1/24/2003		3	3	140,168	32,422	7,796
SS	1/24/2003		4	3	142,087	43,956	7,699
SS	1/24/2003		5	3	147,615	33,568	7,199
SS	1/24/2003		6	3	151,393	31,615	7,218
Average					***171,470***	***38,269***	***7,507***

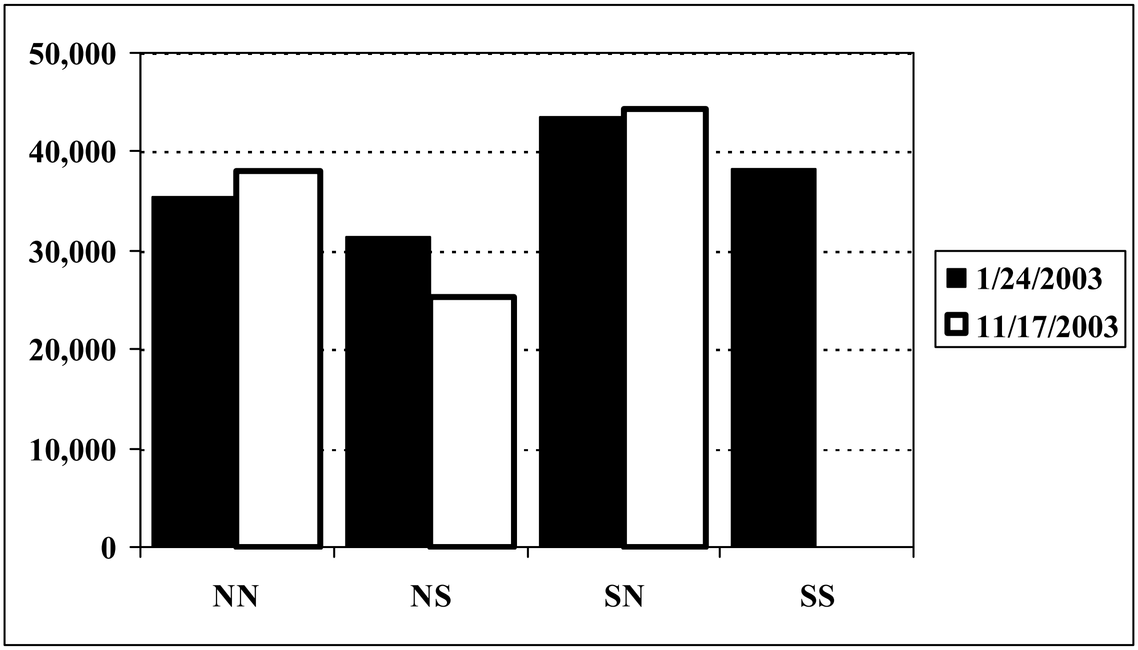


Figure 4.15: Average Stabilized Soil Modulus Backcalculated from FWD Deflections

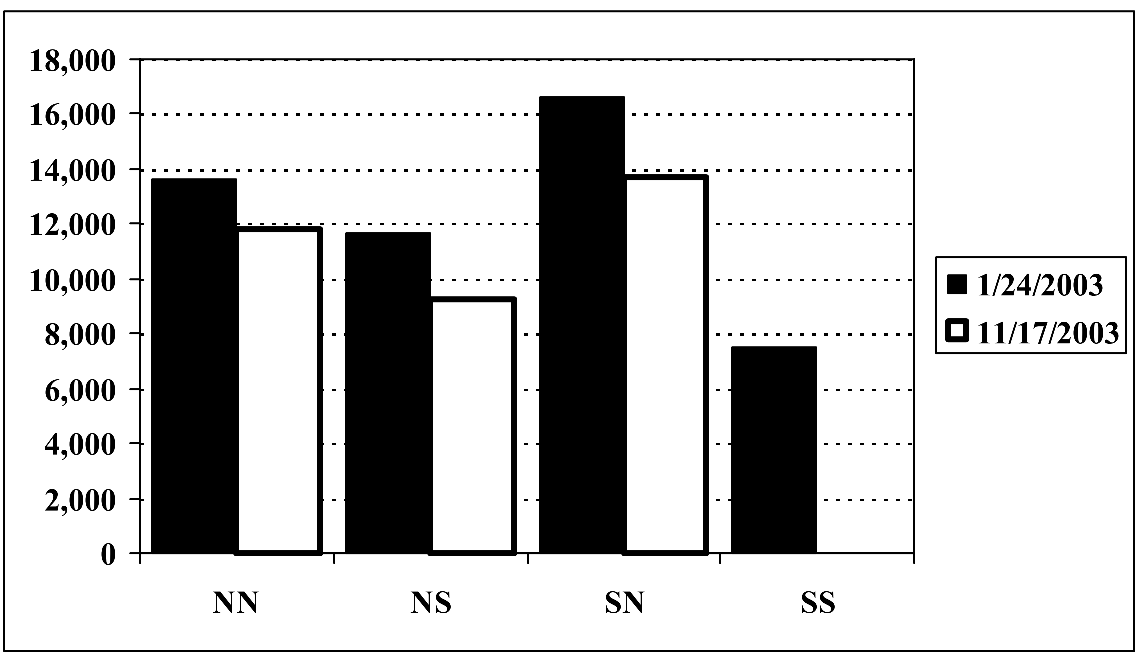


Figure 4.16: Average Backcalculated Subgrade Soil Modulus

4.7 Results of the Weight Drop Tests

Weight drop tests were performed on the same day profile measurements were made at two stations for each test section: one at the West side and one at the East side of the section. The weight drop test consisted of dropping a weight of 60 lbs on a set of rubber plates that transmitted the load to a circular steel plate, nine inches in diameter. The plate was placed at the top of the pavement. The dynamic impact load was measured with a load cell under the rubber plates. The pavement surface deflections were measured by nine LVDTs fixed on a reference plastic beam. The maximum deflections and impact load are provided in Appendix F.

The MODULUS 4.0 program [16] was used to backcalculate the layer moduli from the deflections and load recorded with the weight drop device, using the layer thicknesses measured with the rod-and-level method (Table 3.5), in the points where the weight drop test were performed. However, the backcalculated moduli were too high or too low. In many cases, non-decreasing deflections were recorded, and the backcalculation could not be performed.

A useful comparative indicator of the stiffness of the pavement structures is the ration, K0, between the maximum impact load and the maximum central deflection. The higher is the ration, K0, the stiffer is the pavement structure. The value of the ratio K0 for each weight drop test is given in Appendix F. The average value of this ratio for each test section is plotted versus the number of applied passes of the ATL machine in Figure 4.17.

The figure shows that, for the NN, NS and SN sections, the pavement stiffness was high at the beginning of the accelerated pavement testing, and then dropped after about 200,000 passes of the APT machine and then remained stable. Figure 4.17 also indicates that the pavement structure in the NN test section, which has the embankment soil stabilized with cement, is stiffer than the pavement structure in the NS test section, which has the embankment soil stabilized with fly-ash. Since the two sections have the same nominal asphalt concrete surface layer and the same subgrade soil layer, the difference in the stiffness of the two pavement structures can only be attributed to the difference in the stiffness of the stabilized embankment layers. Thus, the cement stabilization lead to a higher stiffening of the embankment soil layer than the fly-ash stabilization.

No comparison can be made between the stiffnesses of the lanes SN and SS, and those of NN and NS sections, because the weight drop tests were not performed at the same date on all pavement sections, and the temperature at the mid-depth of the asphalt layer was likely different.

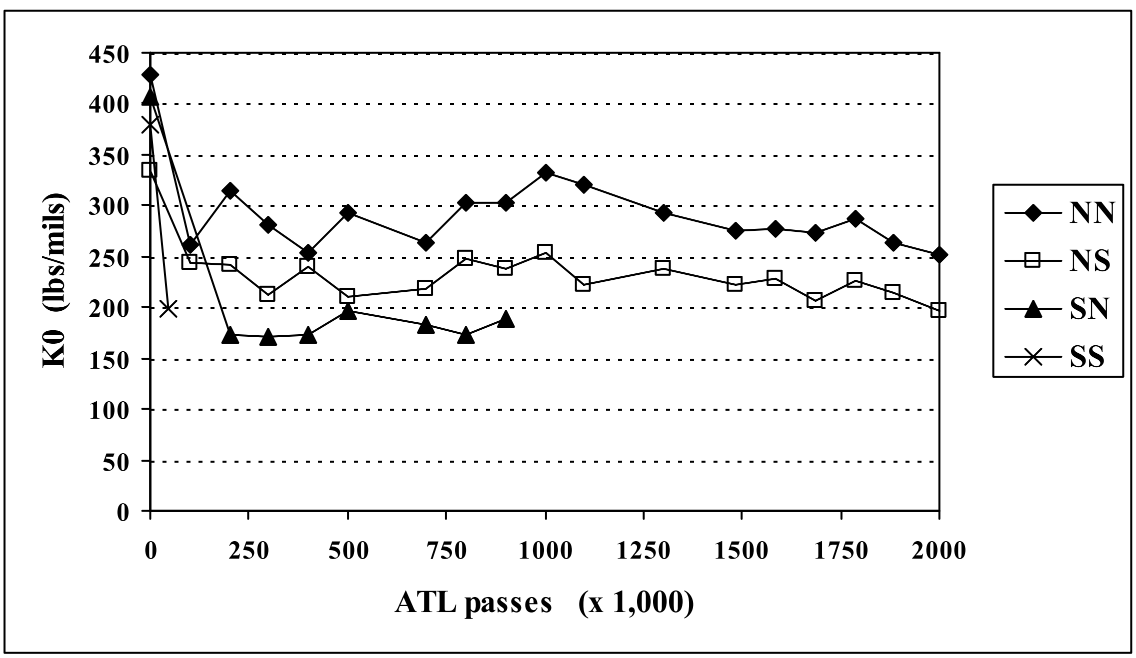


Figure 4.17: K0 from the Weight Drop Central Deflections

4.8 Post-Mortem Evaluation

After 2,000,000 ATL load repetitions applied to the NN and NS sections, the permanent deformation and fatigue cracking reached severe levels. In-service pavements are normally rehabilitated before they reach this poor condition. Due to time and financial constraints, it was decided not to continue loading of the pavement section with lime stabilized embankment soil beyond the 800,000 passes already applied. A destructive post-mortem evaluation was then conducted to further investigate the failure modes of the three pavement sections and to measure the thickness of the HMA layers thru destructive methods.

4.8.1 Trenching and Coring

Three transverse trenches were cut in each of the two pairs of test sections. The trenches were one foot wide. After the cuts were performed with a wet saw, the asphalt concrete was removed without disturbing the base layer (Figure 4.18). Six-inch and four-inch diameter asphalt cores were extracted by a specialized crew from Kansas DOT, from the wheel path and outside the wheel path areas on each lane. After the trenches were cut and three inches of base material were removed, the asphalt concrete layer thickness was measured with a caliper at points spaced at 0.5 inches.

Square slabs (18 in. x 18 in.) were also sawn from the asphalt layer from the outside the wheel path areas. The sawn slabs were numbered and transported to the Advanced Asphalt Laboratory in Fiedler Hall on the KSU campus. The slabs were cut into smaller 10 in. by 13 in. slabs and then set into metal forms. Ready-mix concrete was used to level the uneven bottom of the slabs so they could be tested in the Hamburg Wheel Tester.

Figures 4.19 and 4.20 show the AC layer thicknesses obtained at the post-mortem trenches. Figure 4.21 and 4.22 shows the thicknesses of the asphalt concrete layer determined on cores. Figures 4.19 to 4.22 indicate that the asphalt concrete layer thickness varied greatly in the transverse direction, between three and five inches. It is also evident that the thickness of the HMA layer in section SN, the pavement section with lime stabilized embankment soil, was about one inch smaller than the thickness of the HMA layer in sections NN and NS, the pavement sections with cement and fly-ash stabilized embankment soil, respectively.

Because of the disturbances that are created during the digging of the trench, no post-mortem transverse profile can be measured at the surface of the subgrade soil layer. Therefore, it was impossible to estimate the contribution of the each layer to the permanent deformation at the pavement surface.

Figure 4.18: Trench Cut on the Tested Pavements

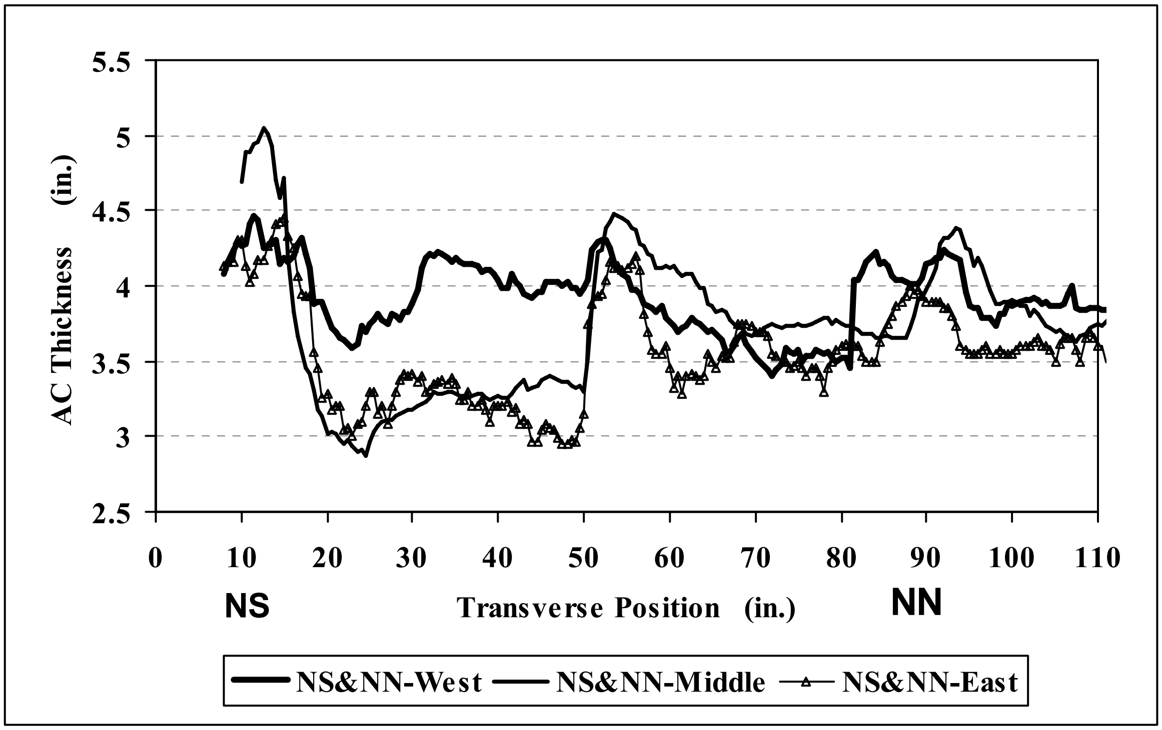


Figure 4.19: Post-Mortem HMA Layer Thickness in the SN Section

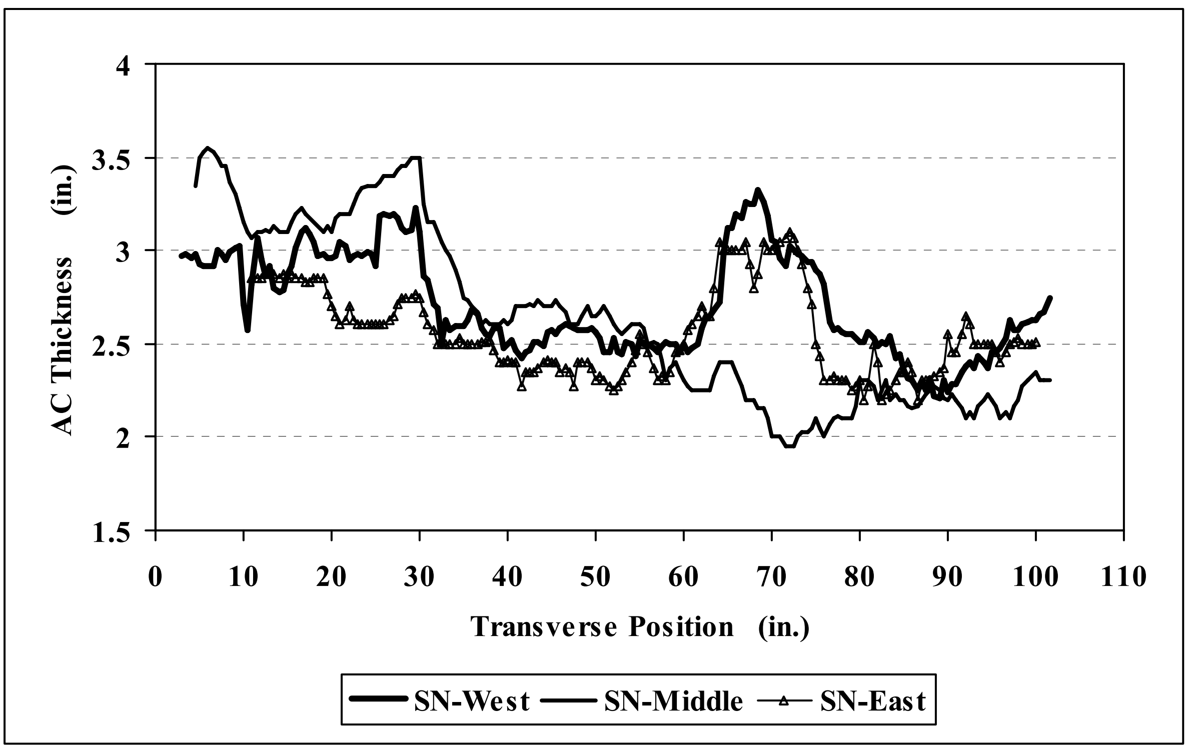


Figure 4.20: Post-Mortem HMA Layer Thickness in the NN and NS Sections

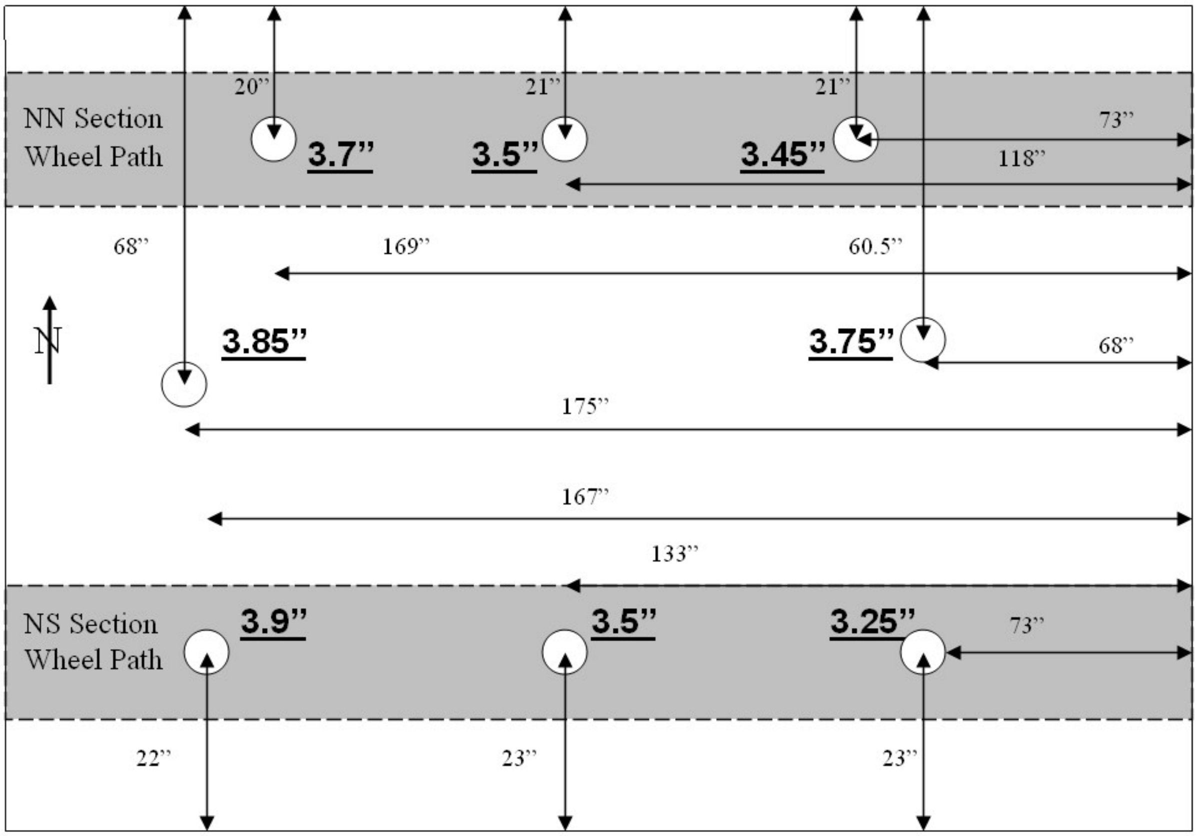


Figure 4.21: HMA Layer Thickness from cores – NN and NS sections

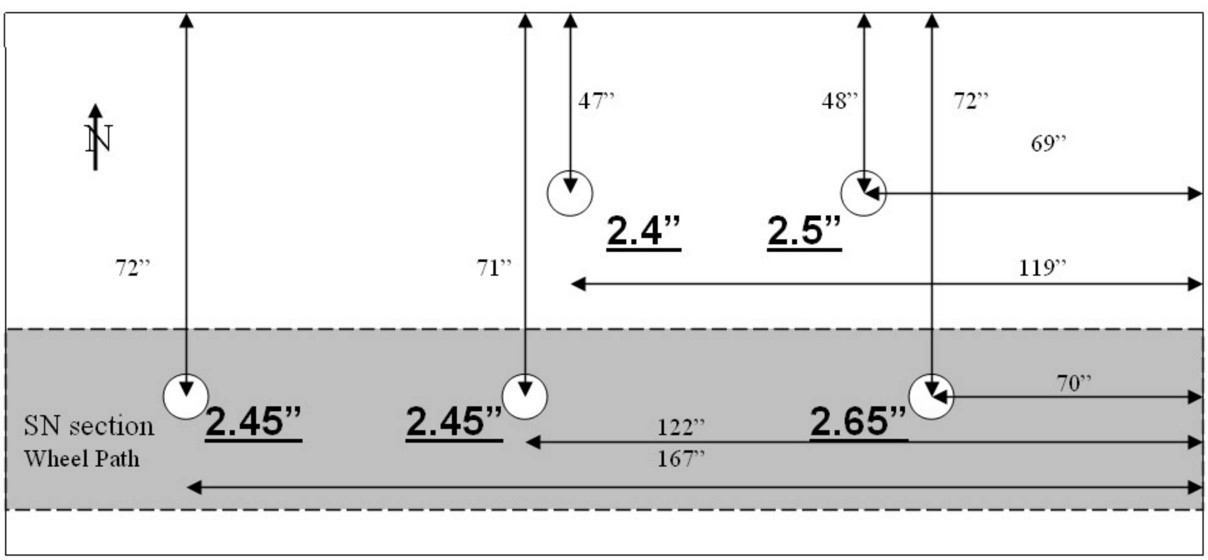


Figure 4.22: HMA Layer Thickness from cores – SN section

4.8.2 Rutting Characteristics of Asphalt Concrete

As mentioned earlier, the slabs sawn from the ATL pavements were trimmed to make specimens for the Hamburg Wheel Tester. The Hamburg wheel-tracking device used in this study has been manufactured by PMW, Inc. based out of Salina, Kansas and is capable of testing a pair of samples simultaneously. Figure 4.23 shows the Hamburg wheel tester at the Advanced Asphalt Test Laboratory of Kansas State University. The sample tested was usually 10.25 in. wide, 12.6 in. long and 1.6 in. deep. The samples were submerged under water at 122°F. The steel wheel of the tester is 4.7cm (1.85in) wide and applies a load of 158lbs and made 52 passes per minute. Each sample was tested for 20,000 passes or until 0.79 in. deformation occurs. Rut depth or deformation is measured at 11 different points along the length of each sample with a Linear Variable Differential Transformer (LVDT).

The results obtained from the Hamburg Wheel Tester are: creep slope, stripping slope and the stripping inflection point as depicted in Figure 4.24 [17]. The creep slope relates to rutting from plastic flow and is the inverse of the rate of deformation in the linear region of the deformation curve, after post compaction effects have been ended and before the onset of stripping. The stripping slope is the inverse of the rate of deformation in the linear region of the deformation curve, after stripping begins and until the end of the test. It is the number of passes required to create one mm impression from stripping, and is related to the severity of moisture damage. The stripping inflection point is the number of passes at the intersection of the creep slope and the stripping slope and is related to the resistance of the HMA to moisture damage. An acceptable mix is specified by the City of Hamburg to have less than 0.16 in. mm rut depth after 20,000 passes at a 122°F test temperature. However, this criterion was found to be very harsh in subsequent studies of the Colorado Department of Transportation [17].

Only one pair of slab sample could be successfully tested in this study using the asphalt concrete slabs sawn from the ATL test pavements. Figure 4.25 shows the vertical deformation of the slabs under the Hamburg Wheel Tester and Table 4.9 tabulates and compares the results of this mixture with a number of Superpave mixtures tested under similar conditions at Kansas State University. Hamburg Wheel tests were also conducted on two sets of cores from the ATL test pavements. The results show that the CISL#12 test pavement asphalt concrete mixture (SM 12.5) outperformed only one Superpave mixtures of similar size in-service tested earlier in terms of rutting.

Figure 4.23: Tested HMA Specimens in the Hamburg Wheel Tester

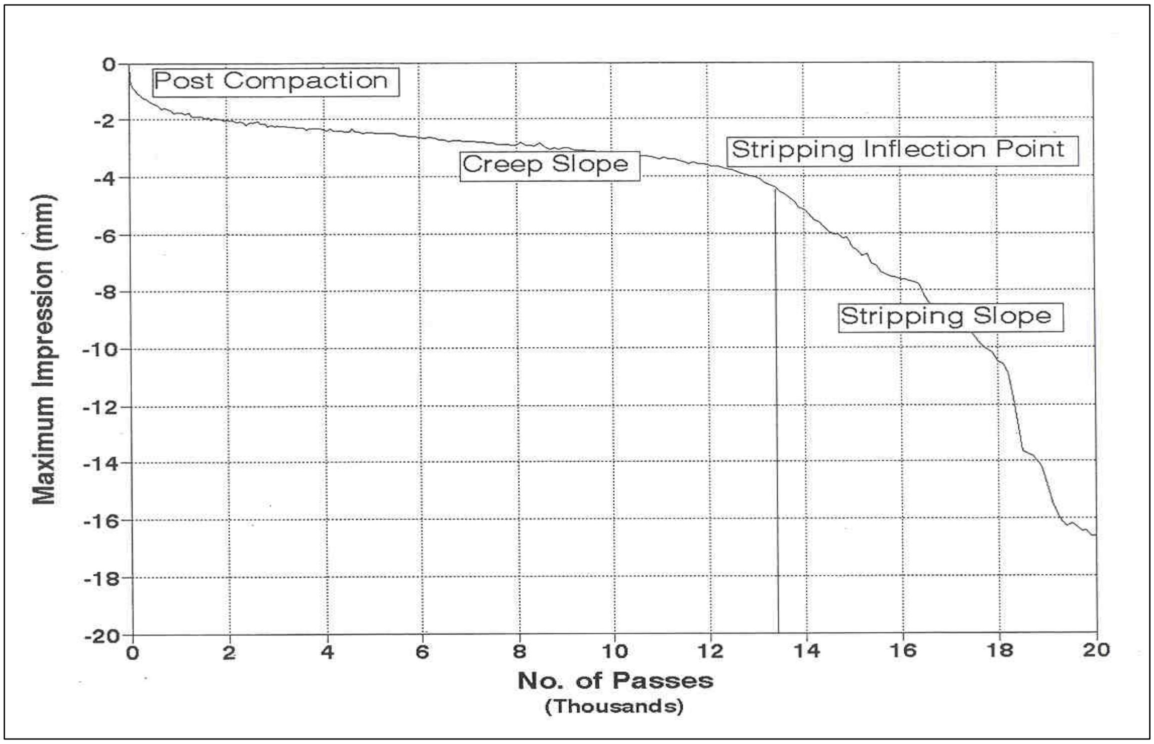


Figure 4.24: Interpretation of Results from the Hamburg Wheel Tester [17]

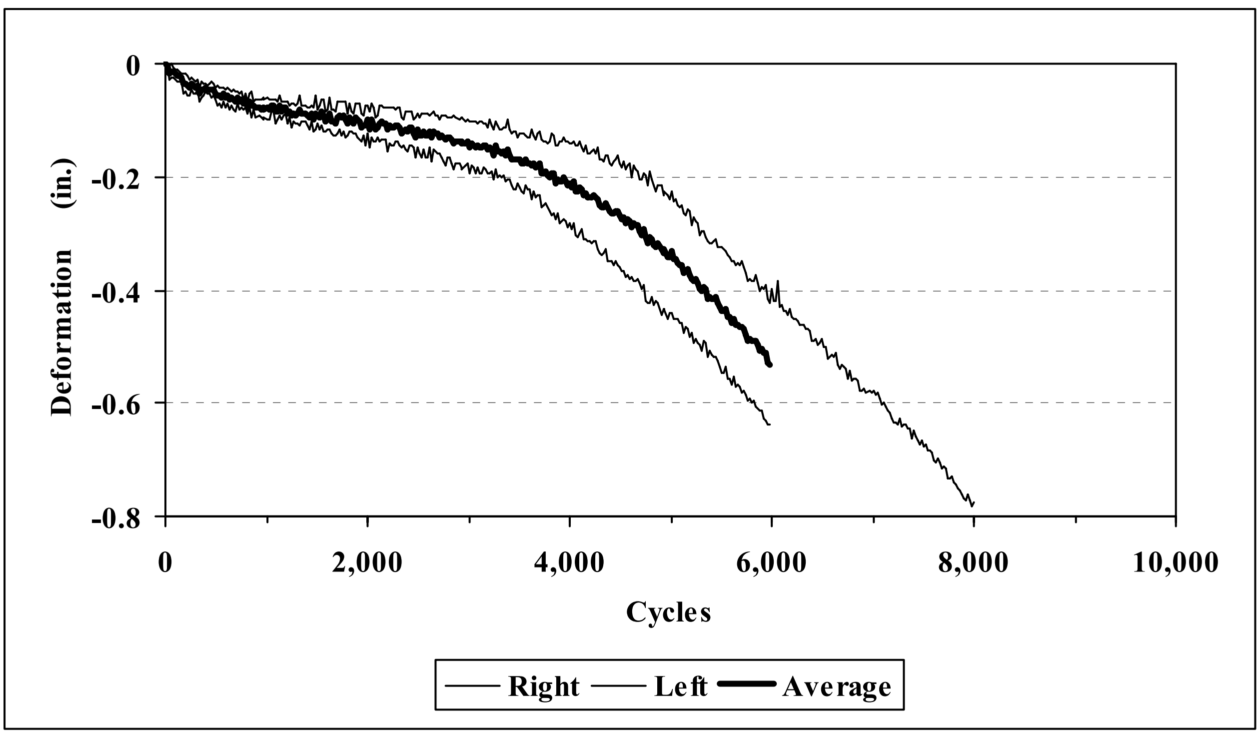


Figure 4.25: Measured Deformation in the Hamburg Wheel Rut Tester

Table 4.9: Summary of Hamburg Wheel Test Results (Ranked by Average Number of Passes)

Mix Type	Description	Number of Passes		Average Number of Passes to 20 mm (0.8 in.) Rut Depth	Average Creep Slope	Average Stripping Inflection Point	Average Stripping Slope
		Specimen 1 (Left)	Specimen 2 (Right)				
SM 19B	Ritchie K-42	1,440	1,320	1,380	117	755	66
SM 12.5A	Shilling K-4	5,421	5,890	5,656	544	3,696	430
CISL#12	**CISL Exp.#12**	**7,981**	**6,500**	**7,240**	**1,117**	**3,800**	**228**
SM 12.5A	Venture K-140	8,861	15,701	12,281	1,333	8,923	420
SM 12.5B	APAC Shears US 50	13,640	11,560	12,600	1,270	10,240	551
SM 12.5B	KAPA Junction City Intersection	13,120	12,321	12,721	954	10,311	788
SM 19A	Venture K-140_4A	12,941	13,721	13,331	1,214	8,347	501
SM 12.5 B	CISL Exp.#11 (Slab)	20,000	15,330	17,675	2,250	10,112	667
SM 12.5 B	CISL Exp.#11 (Cores 1 & 2)	20,000	14,600	17,300	4,208	11,723	705
SM 12.5 B	CISL Exp.#11 (Cores 3 & 4)	19,241	15,640	17,440	2,719	9,104	668
SM 19A	KDOT Research Special	20,000	16,161	18,081	2,667	14,521	1,333
SM 19 B	Shilling US 75 6C	19,981	20,000	19,991	12,413	14,614	6,667

CHAPTER 5 - COMPARISON OF MEASURED PAVEMENT RESPONSE AND THE RESPONSE ESTIMATED WITH A LINEAR-ELASTIC PAVEMENT STRCTURAL MODEL

5.1 The EVERSTRESS Pavement Response Calculation Program

The theoretical pavement response values were computed using Everstress 5.0 software, a layered elastic analysis program developed by the Washington Department of Transportation (WSDOT) [18]. The program is used to compute stresses, strain, and deflections in a layered elastic system under circular surface loads. Figures 5.1 and 5.2 show typical data input screens.

The software performs calculations for up to five layers, 20 circular loads and 50 evaluation points. For each location in the horizontal plane the calculations are done for up to five points in the vertical (Z) direction, as can be seen in Figure 5.2. The software can operate in metric or US customary units.

The program can take into consideration any stress dependent stiffness characteristics [18]. The stresses are calculated at X=0, Y=0 and at the bottom of 1st layer, at the top of the last layer, and at the middle of the intermediate layers. The tensile stresses and strains are considered positive and the compressive stresses and strains are considered negative. The vertical displacements are considered positive when the points are moving downward. A typical output is given in Figure 5.3.

5.2 The Modeling of CISL Pavement Structures and Loading

The modeling of the CISL pavement structures and loading was performed for the days when the FWD tests were performed, because for these dates the backcalculated layer moduli were available. The backcalculated moduli were used as input values in the response calculation process (Table 3.1). No temperature correction was used since the backcalculated moduli of the asphalt layer were the values obtained for the temperature in the asphalt layer at the time of the FWD tests. The same HMA thickness values used when the moduli backcalculation (Table 5.1) was performed was used for the strain and stress calculation with Everstress.

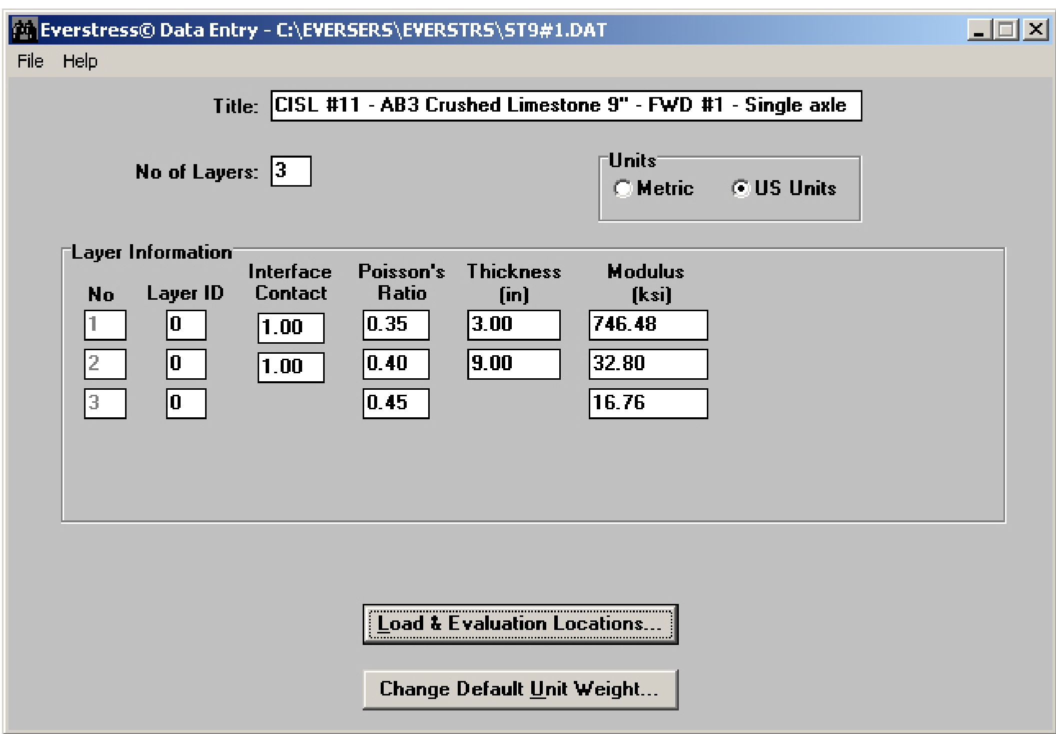


Figure 5.1: Layers Characteristics Input Data

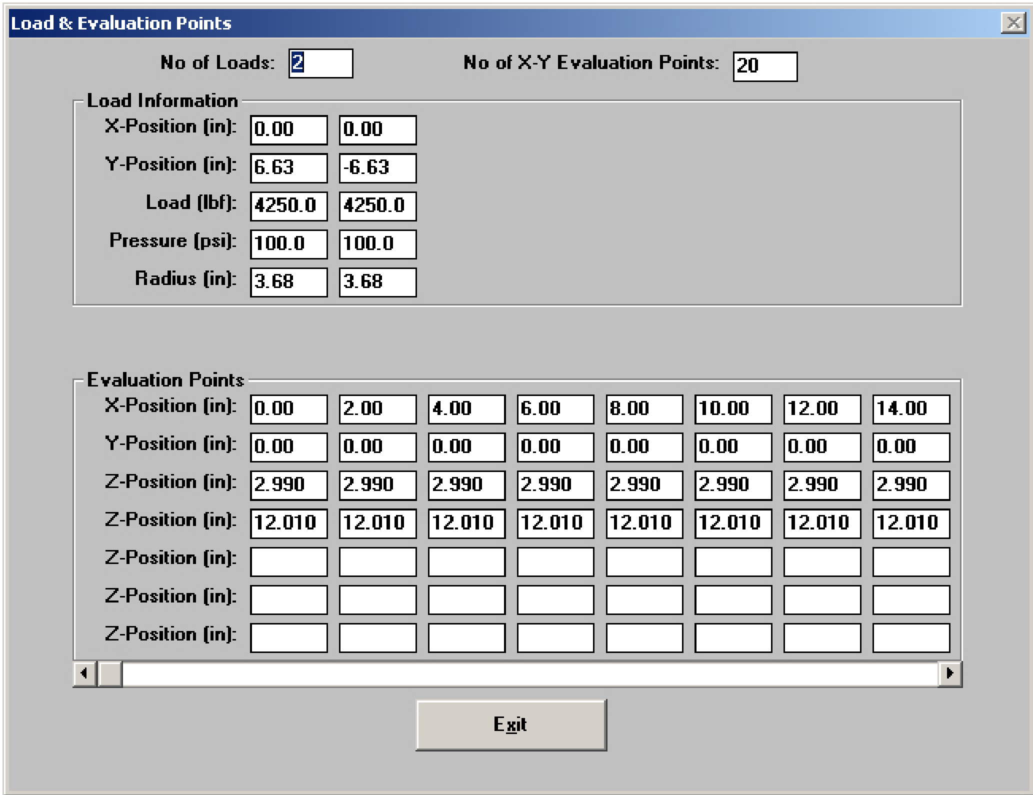
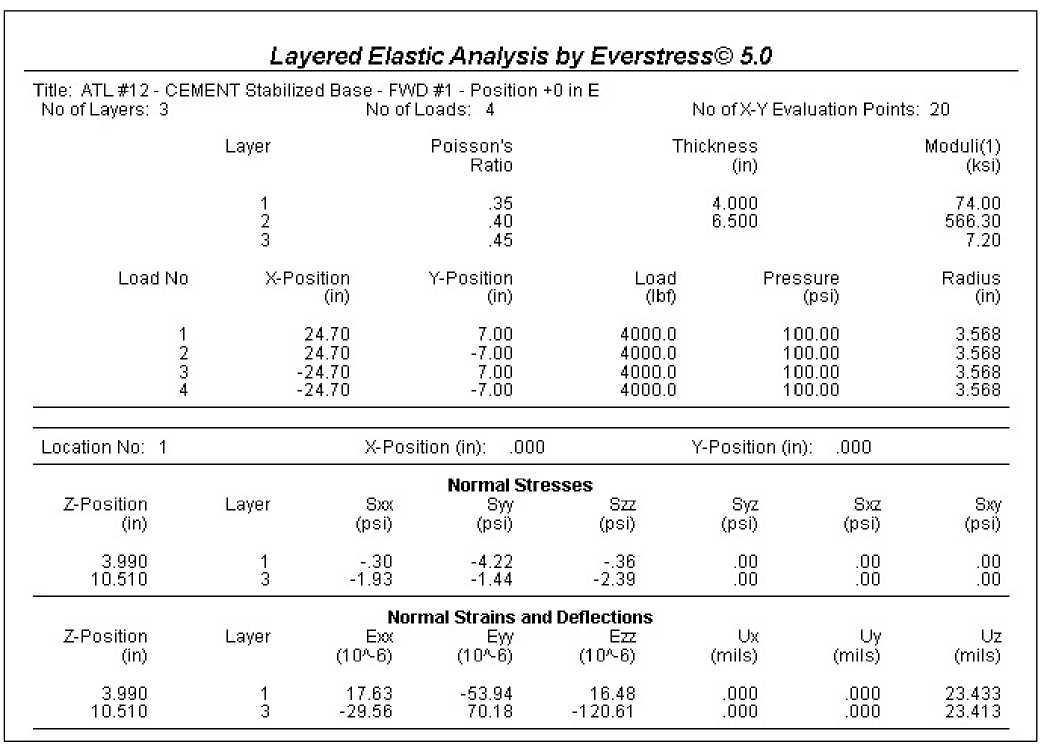


Figure 5.2: Load Characteristics Input Data



Layered Elastic Analysis by Everstress© 5.0

Title: ATL #12 - CEMENT Stabilized Base - FWD #1 - Position +0 in E
No of Layers: 3 No of Loads: 4 No of X-Y Evaluation Points: 20

Layer	Poisson's Ratio	Thickness (in)	Moduli(1) (ksi)
1	.35	4.000	74.00
2	.40	6.500	566.30
3	.45		7.20

Load No	X-Position (in)	Y-Position (in)	Load (lbf)	Pressure (psi)	Radius (in)
1	24.70	7.00	4000.0	100.00	3.568
2	24.70	-7.00	4000.0	100.00	3.568
3	-24.70	7.00	4000.0	100.00	3.568
4	-24.70	-7.00	4000.0	100.00	3.568

Location No: 1 X-Position (in): .000 Y-Position (in): .000

Normal Stresses

Z-Position (in)	Layer	Sxx (psi)	Syy (psi)	Szz (psi)	Syz (psi)	Sxz (psi)	Sxy (psi)
3.990	1	-.30	-4.22	-.36	.00	.00	.00
10.510	3	-1.93	-1.44	-2.39	.00	.00	.00

Normal Strains and Deflections

Z-Position (in)	Layer	Exx (10^-6)	Eyy (10^-6)	Ezz (10^-6)	Ux (mils)	Uy (mils)	Uz (mils)
3.990	1	17.63	-53.94	16.48	.000	.000	23.433
10.510	3	-29.56	70.18	-120.61	.000	.000	23.413

Figure 5.3: Typical Output of EverStress Software

TABLE 5.1: Pavement Structure Information used as Input in the Everstress Software

Pavement	Session	Station	Thickness		Moduli (psi)		
			HMA	Base	HMA	Base	Subgrade
Cement	1	w	3.80	6.40	491,015	38,282	13,212
		e	4.00	6.50	291,814	31,061	14,280
	2	w	3.80	6.40	498,990	28,601	11,728
		e	4.00	6.50	315,545	41,955	12,777
Fly-ash	1	w	3.40	6.80	462,319	39,048	12,377
		e	3.70	6.60	287,775	24,602	11,190
	2	w	3.30	6.80	335,921	24,665	9,993
		e	3.70	6.60	219,249	35,632	6,564
Lime	1	w	2.60	6.10	199,310	37,276	17,030
		e	2.10	6.80	179,830	60,217	18,005
	2	w	2.60	6.10	328,930	45,205	12,341
		e	2.10	6.80	402,210	40,142	15,463
EMC[2]	1	w	2.90	5.40	215,232	43,762	7,561

* *Poisson Ratio Values: H1: v = 0.35, H2: v = 0.40, H3: v = 0.45*
Session 1 – 03/25/2003 – before loading; Session 2 – 11/20/2003 at 1.1 M load repetitions for Cement and Fly-Ash sections, 800,000 load repetitions for the Lime section

A tandem axle bogie with dual tires was modeled, since this loading configuration was used on the dates when the FWD tests were performed. Figure 5.4 shows the geometric characteristics of the tandem axle bogie with dual tires. Because the axle passes above two pavements built in the same pits, only one half of the axle is used for the modeling of a single pavement.

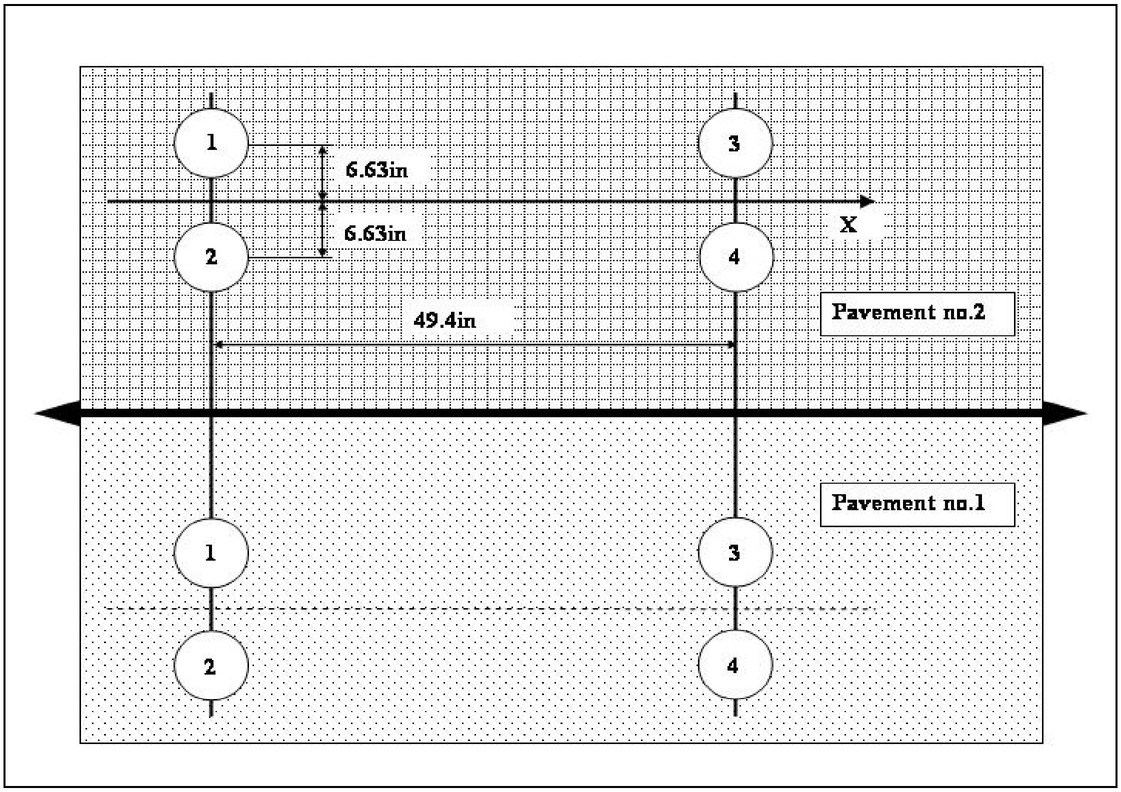


Figure 5.4: Tandem Axle Dual Tire Bogie on Two Pavements

All calculations were performed using a dual-tire set up, as presented in Figures 5.5 and 5.6.

- The calculations were performed for the same geometrical characteristics of the bogie and the constructed pavements.
- The origin of the coordinate system was selected at the center of symmetry of the dual wheel - dual tire half axle.
- The pavement response was computed on 40 points distributed 20 points each on two parallel axes as follows: 20 points on the longitudinal axis of symmetry of the wheel (Y = 0 inches) and 20 points on an axis parallel to the axis of symmetry but at six inches offset (Y = 6 inches).

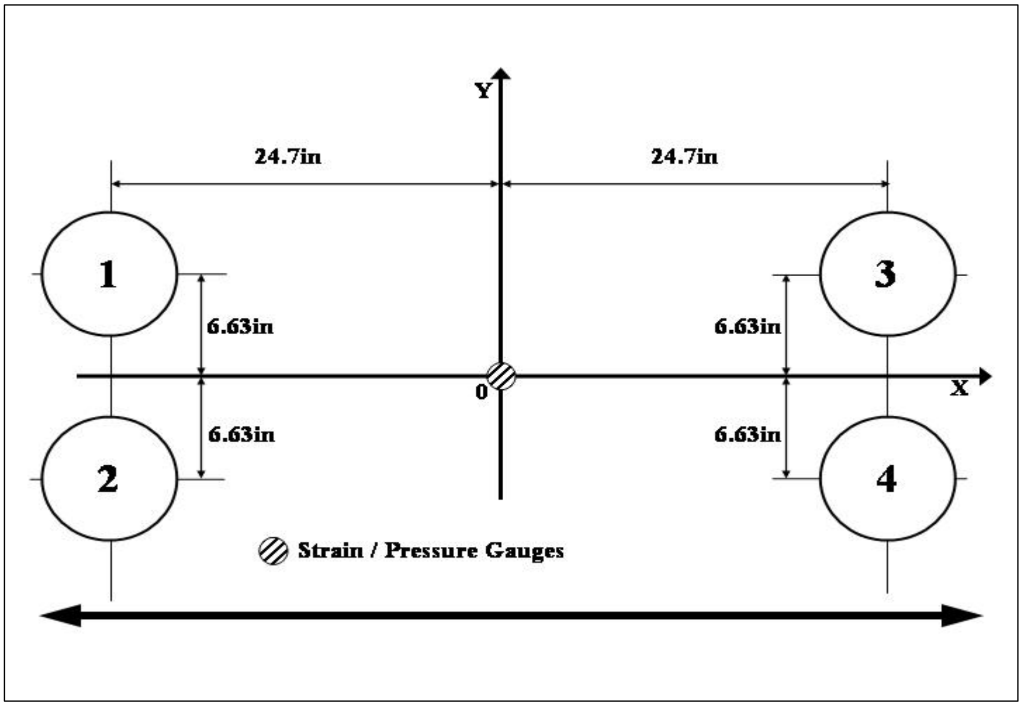


Figure 5.5: Loading Model for Tandem Axle Dual Tire Loading at Position 0"

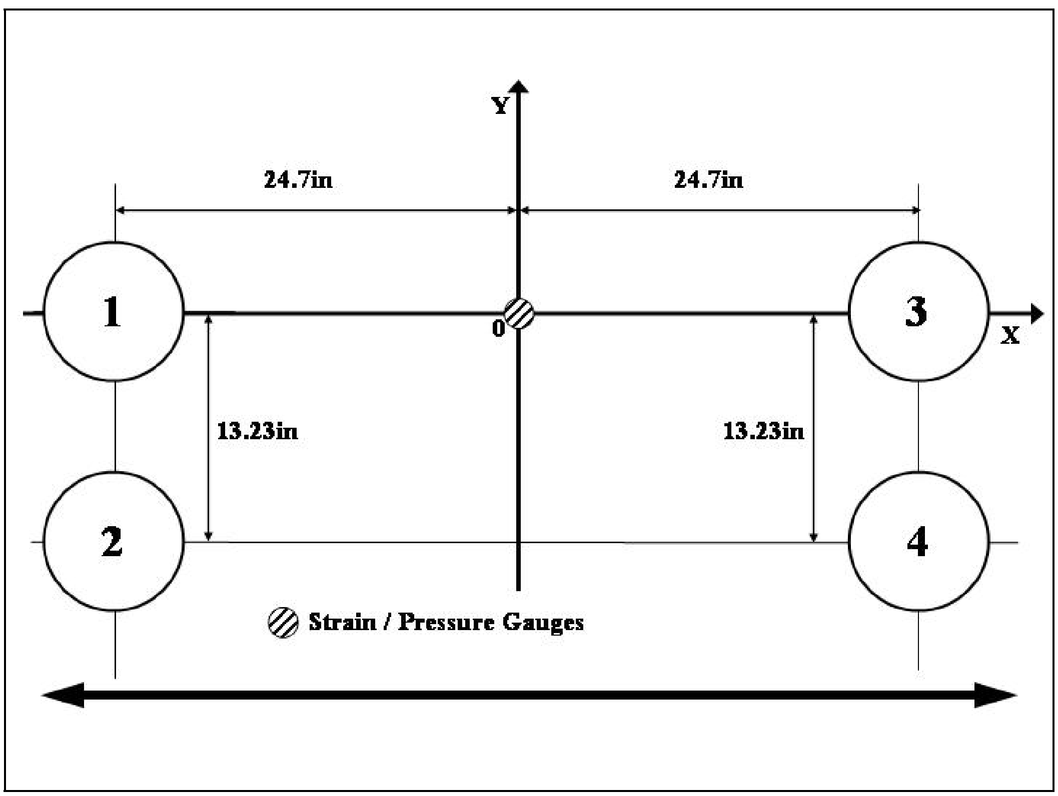


Figure 5.6: Loading Model for Tandem Axle Dual Tire Loading at Position +6"

5.3 Analysis of CISL Experiment #12 Response Data

The peak values of the computed and measured longitudinal and transverse strains at the bottom of the asphalt concrete surface layer and vertical compressive stresses at the top of the subgrade soil layer are presented in Tables 5.2 to 5.4. The following is a discussion of the differences between the measured and computed stresses and strains and the most probable causes for the differences observed.

Table 5.2: Computed and Measured Longitudinal Strain at the Bottom of the HMA Layer

Pavement Section	FWD Session	Position (in)	Computed Strain (C) (microstrain)		Measured Strain (M) (microstrain)		Ratio (M/C)	
			East	West	East	West	East	West
CEMENT	1	0	-60.5	-52.5				
	2	0	-58.4	-21.4				
	2	6	-60.6	-38.8				
FLY-ASH	1	0	-27.2	-49.2		242		-4.92
	2	0	-43.4	-65.0				
	2	6	-79.4	-95.8				
LIME	1	0	-74.7	-91.0		39		-0.43
	1	6	-73.8	-87.6		83		-0.95
	2	0	-111.5	-75.7		25		-0.33
	2	6	-113.1	-99.0		66		-0.67
EMC^2	1	0	-	-123.4		174		-1.41
	1	6	-	-110.3		149		-1.35

Session 1 – 03/25/2003 – before loading; Session 2 – 11/20/2003 at 1.1 M load repetitions for Cement and Fly-Ash sections, 800,000 load repetitions for the Lime section

Table 5.3: Computed and Measured Transverse Strain at the Bottom of the HMA Layer

Pavement Section	FWD Session	Position (in)	Computed Strain (C) (microstrain)		Measured Strain (M) (microstrain)		Ratio (M/C)	
			East	West	East	West	East	West
CEMENT	1	0	-93.4	-78.8	-68		0.73	
	2	0	-87.5	-72.5	-89		1.02	
	2	6	-59.1	-20.2	82		-	
FLY-ASH	1	0	-94.2	-99	-32		0.34	
	2	0	-142.6	-143.2	-24		0.17	
	2	6	-40.2	-61.4	129		-	
LIME	1	0	-84.6	-1.1	-77		0.91	
	1	6	-62.6	-70.6	47		-	
	2	0	-120.4	-121.7	-21		0.17	
	2	6	-85.4	-61.4	86		-	
EMC[2]	1	0		-127.6				
	1	6		-93.8				

Session 1 – 03/25/2003 – before loading; Session 2 – 11/20/2003 at 1.1 M load repetitions for Cement and Fly-Ash sections, 800,000 load repetitions for the Lime section

Table 5.4: Computed and Measured Vertical Stress at the top of the Embankment Soil

Pavement Section	FWD Session	Position (in)	Computed Stress (C) (psi)		Measured Stress (M) (psi)		Ratio (M/C)	
			East	West	East	West	East	West
CEMENT	1	0	-3.84	-3.36	-6.06	-3.80	1.58	1.13
	2	0	-3.31	-3.44	-2.15	-7.12	0.65	2.07
	2	6	-3.28	-3.48	-2.38	-5.22	0.73	1.50
FLY-ASH	1	0	-3.93	-3.53	-12.95	-1.73	3.30	0.49
	2	0	-4.15	-4.19	-11.96	-4.99	2.88	1.19
	2	6	-4.23	-4.26	-9.93	-4.12	2.35	0.97
LIME	1	0	-4.12	-4.72	-5.12	-1.87	1.24	0.40
	1	6	-4.09	-4.69	-5.89	-1.10	1.44	0.23
	2	0	-4.89	-5.22	-3.14	-2.52	0.64	0.48
	2	6	-4.87	-5.29	-2.98	-2.41	0.61	0.46
EMC[2]	1	0		-4.94	-9.72	-13.39		2.71
	1	6		-5.01	-7.08	-10.00		2.00

Session 1 – 03/25/2003 – before loading; Session 2 – 11/20/2003 at 1.1 M load repetitions for Cement and Fly-Ash sections, 800,000 load repetitions for the Lime section

5.3.1 The Pavement Section with Cement Stabilized Soil

Figures 5.7 to 5.12 present the measured and calculated longitudinal and transverse strains at the bottom of the asphalt concrete layer and vertical stresses at the top of the subgrade for the pavement with the cement-treated soil subgrade. The computations were performed only for the two FWD test dates, since the FWD data were used to backcalculate the pavement layer moduli. A tandem axle load was applied to the pavement structure on the dates the FWD tests were performed.

Figures 5.7 to 5.9 indicate that the shapes of the computed and measured transverse strain are similar in shape and they are negative when the wheel passes in Position 0”. However, when the wheel is in Position +6”, with the tires straddling the strain gages, the measured strain was positive while the computed strain was negative.

Figure 5.10 to 5.11 reveal that the shapes of the measured and calculated vertical stresses at the top of the subgrade soil layer are similar. The highest recorded vertical stress is always higher than the computed theoretical stress.

5.3.2 The Pavement Section with Fly Ash Stabilized Soil

Figures 5.13 to 5.16 give the shapes of the measured and calculated longitudinal and transverse strains at the bottom of the asphalt concrete layer and vertical stresses at the top of the subgrade for the pavement with fly-ash-treated soil subgrade. Figures 5.13 and 5.15 indicate that the shapes of the computed and measured transverse strain are not similar and that the maximum computed transverse strain is negative while the maximum measured transverse strain is positive, with the maximum absolute value of the computed strain higher than that of the measured strain.

When the wheel passes in Position +6", the measured and computed transverse strains have the same sign and similar shapes, with the maximum measured strain being almost double the maximum computed strain. Figure 5.14 shows that the maximum measured longitudinal strain value is almost five times greater than the maximum computed longitudinal strain; the signals have similar shapes.

Figure 5.17 to 5.19 indicate that the shapes of the measured and calculated vertical stresses at the top of the subgrade soil layer are similar. The highest recorded vertical stress is always higher than the computed theoretical stress.

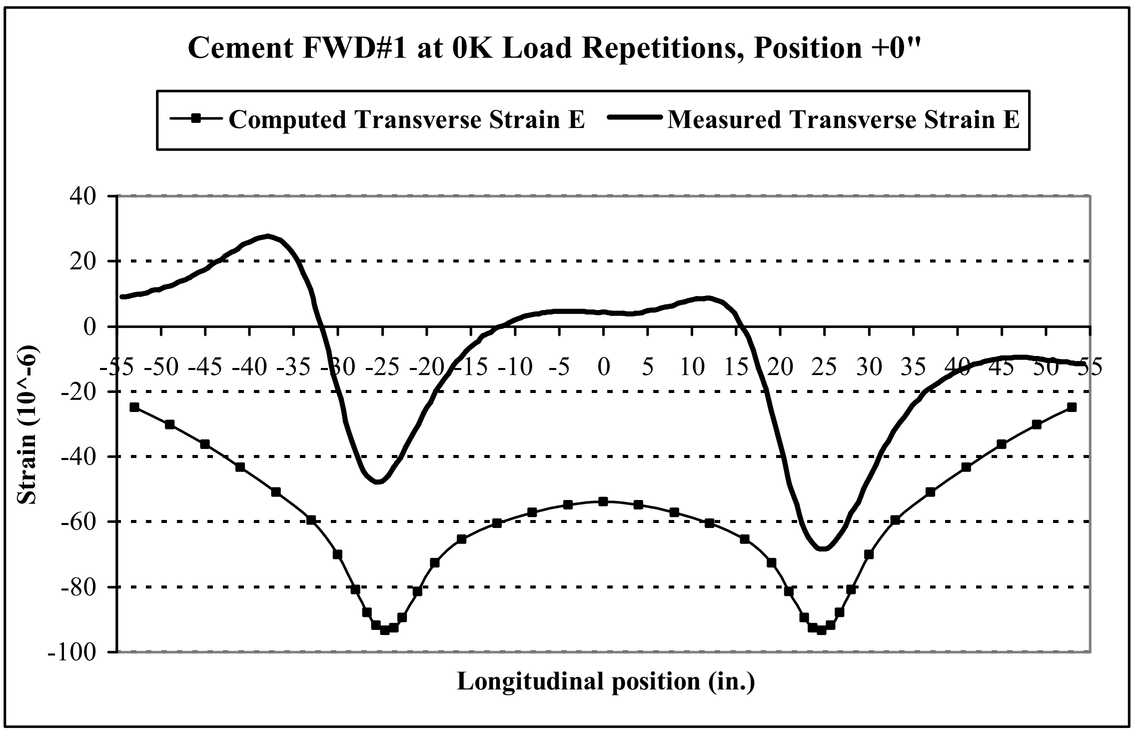


Figure 5.7: Transverse Strain – Section NN – Position +0"(03/25/2003)

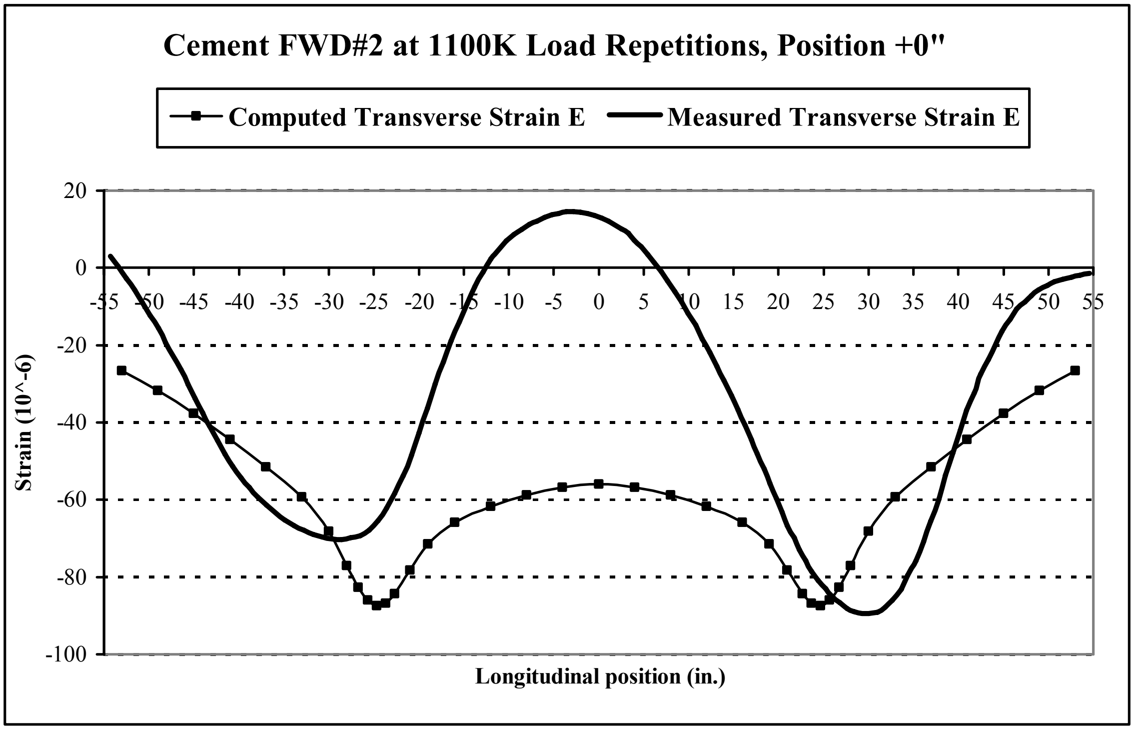


Figure 5.8: Transverse Strain – Section NN – Position +0” (11/20/2003)

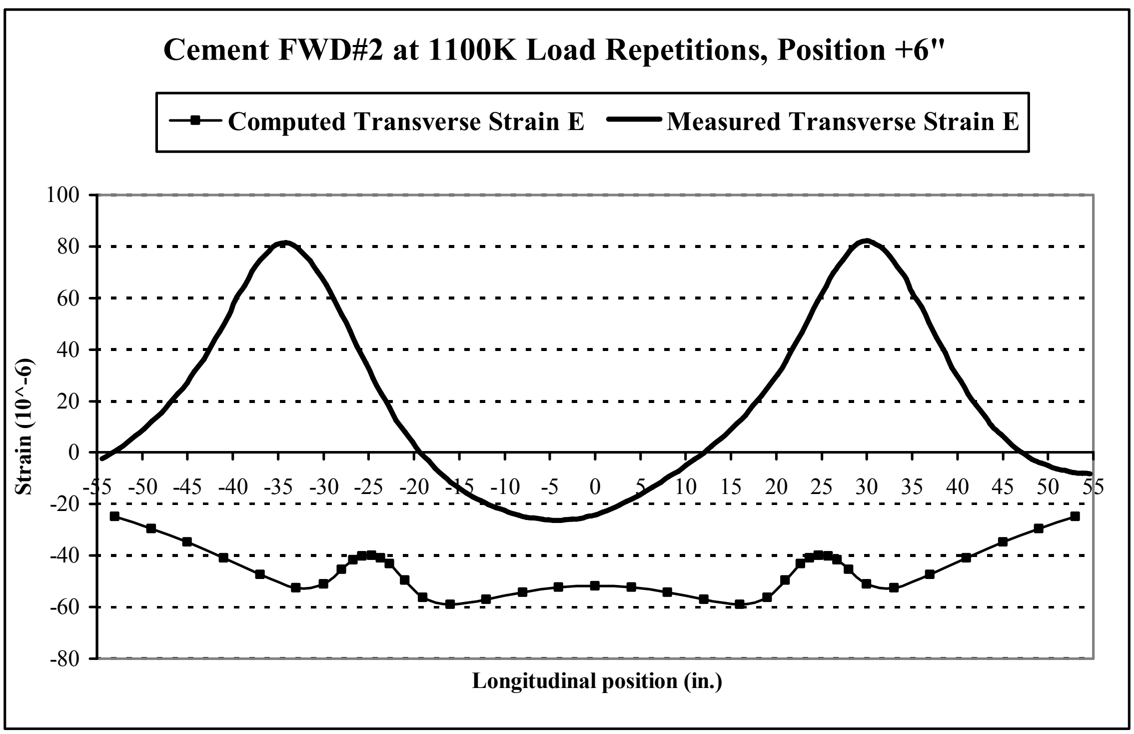


Figure 5.9: Transverse Strain – Section NN – Position +6”((11/20/2003)

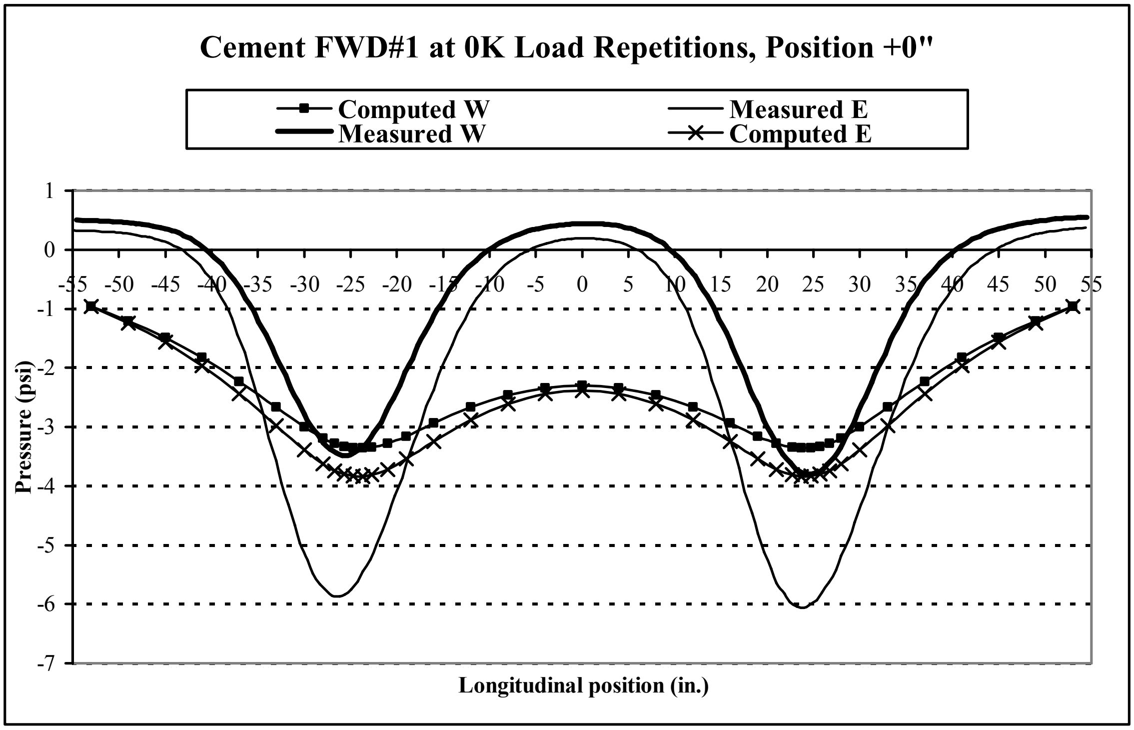


Figure 5.10: Vertical Stress – Section NN – Position +0" (03/25/2003)

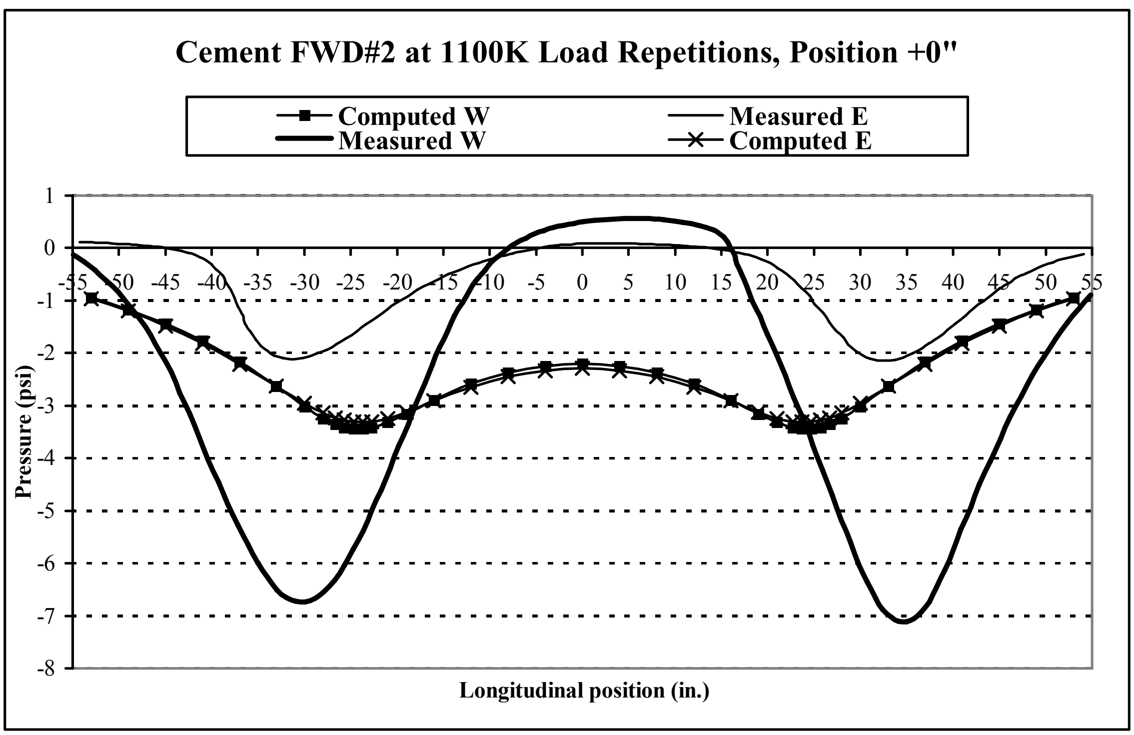


Figure 5.11: Vertical Stress – Section NN – Position +0" (11/20/2003)

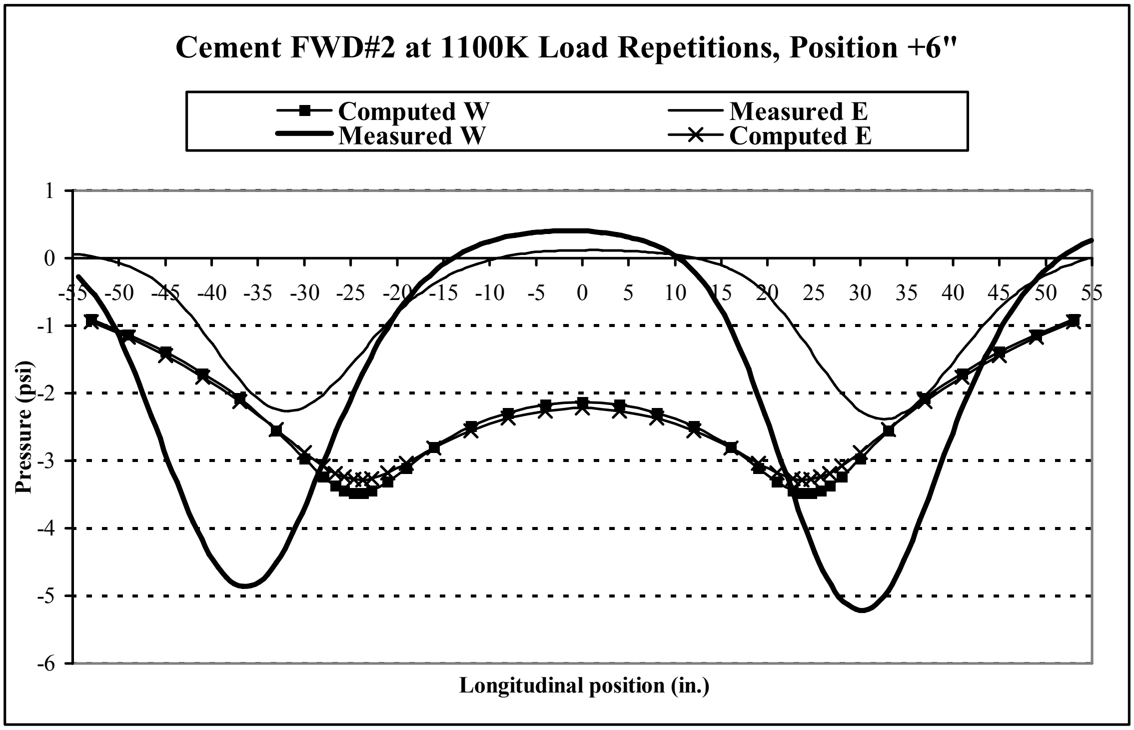


Figure 5.12: Vertical Stress – Section NN – Position +6” (11/20/2003)

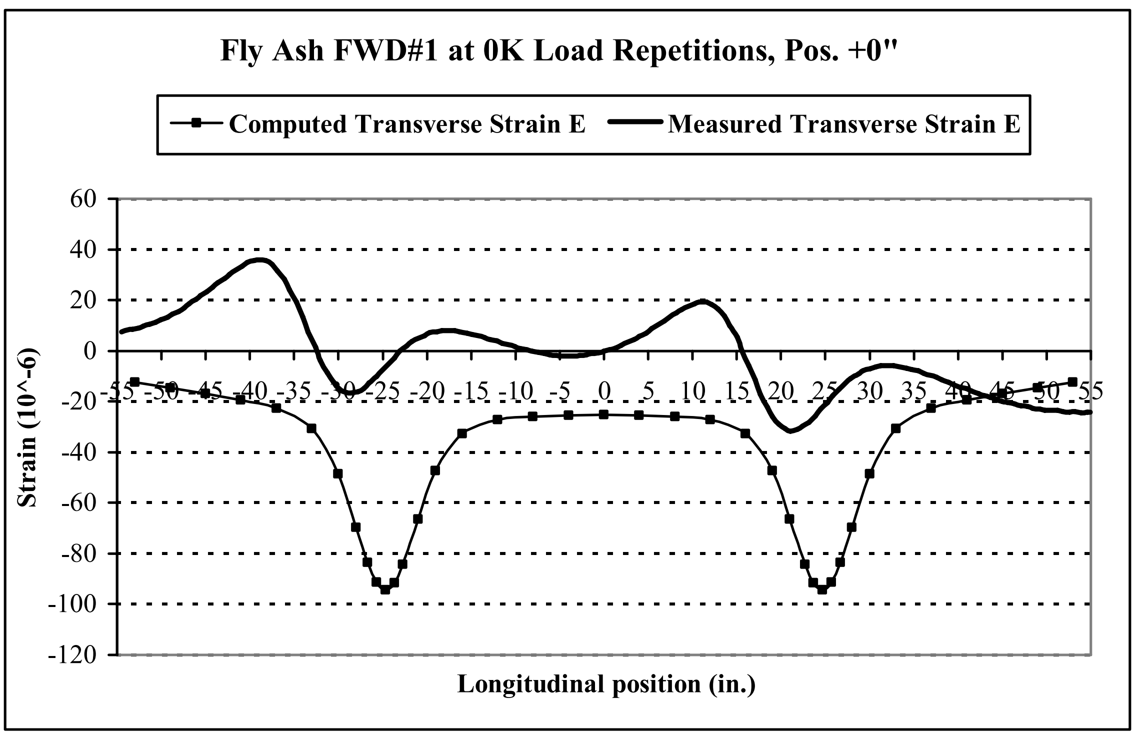


Figure 5.13: Transverse Strain – Section NS – Position +0” (03/25/2003)

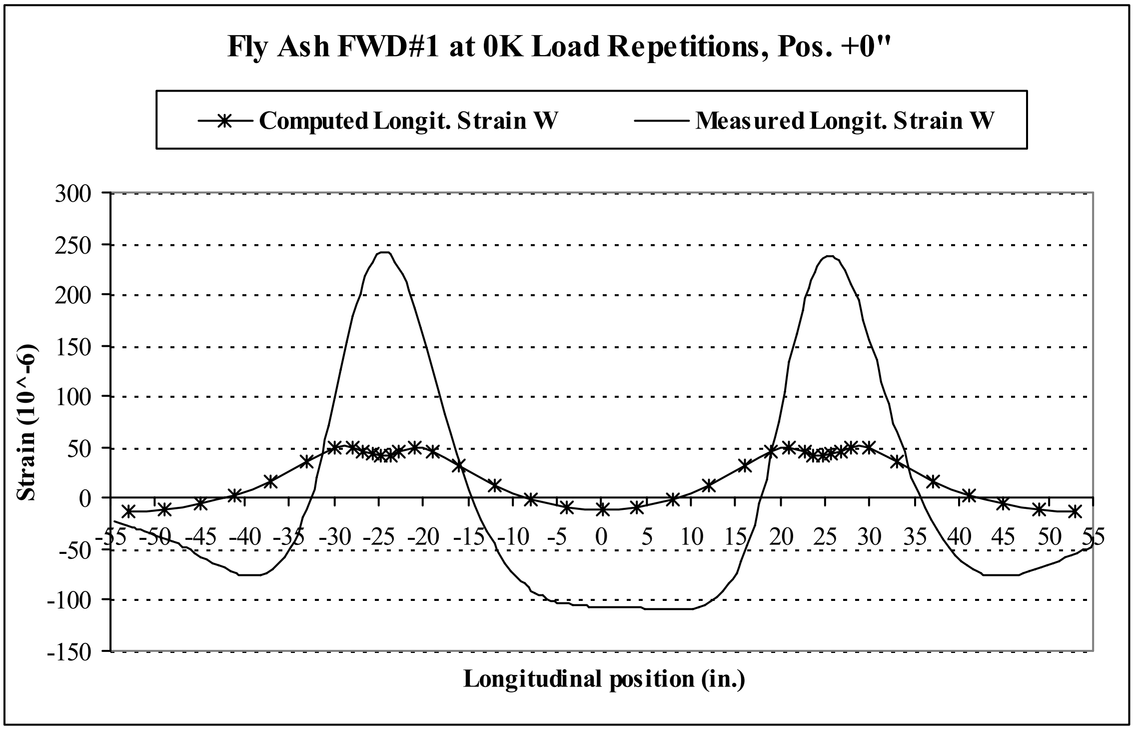


Figure 5.14: Longitudinal Strain – Section NS – Position +0" (03/25/2003)

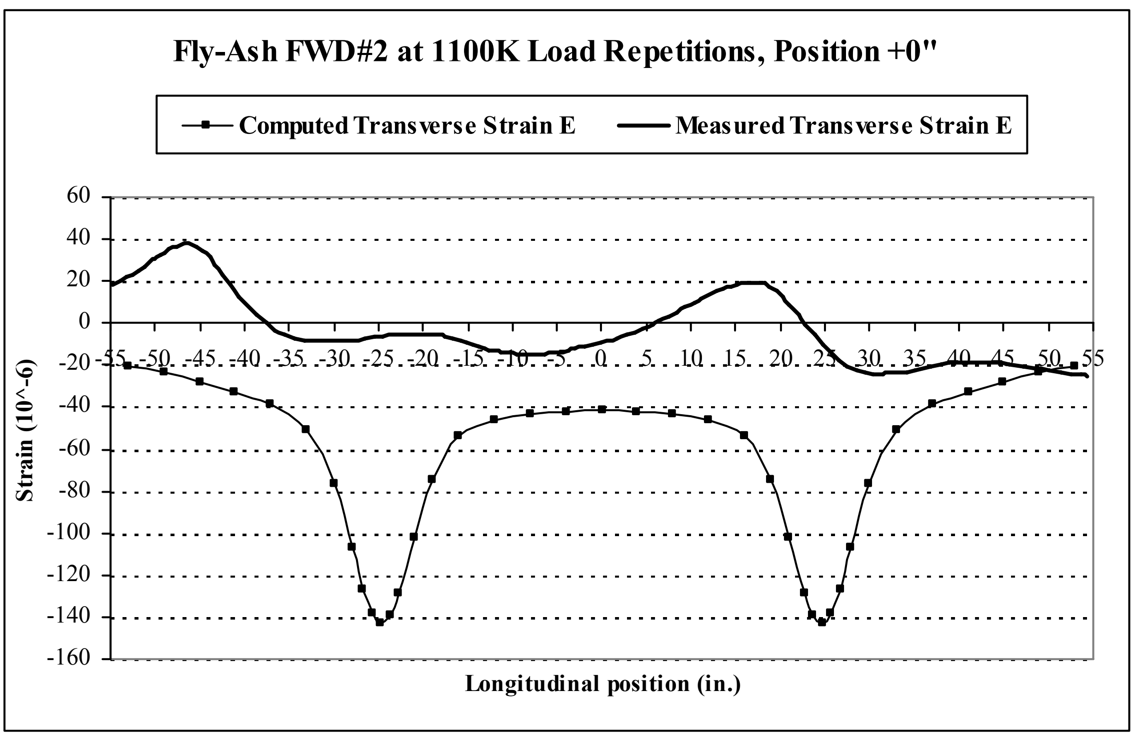


Figure 5.15: Transverse Strain – Section NS – Position +0" (11/20/2003)

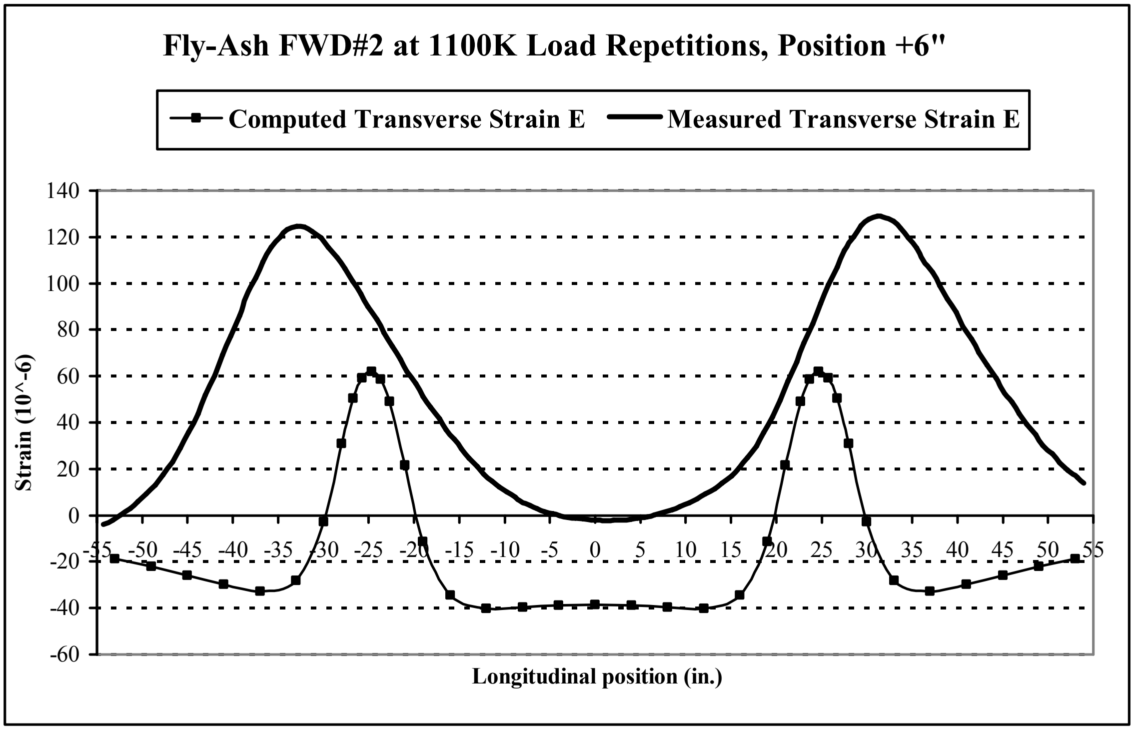


Figure 5.16: Transverse Strain– Section NS – Position +6” (11/20/2003)

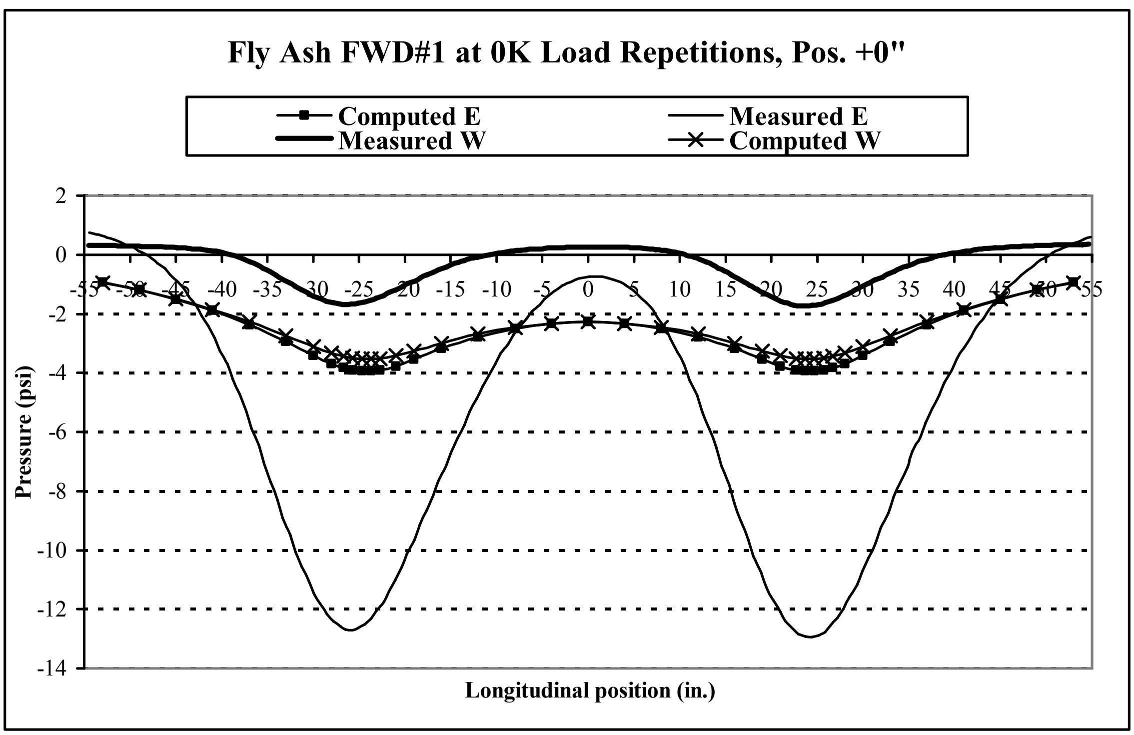


Figure 5.17: Vertical Stress – Section NS – Position +0” (03/25/2003)

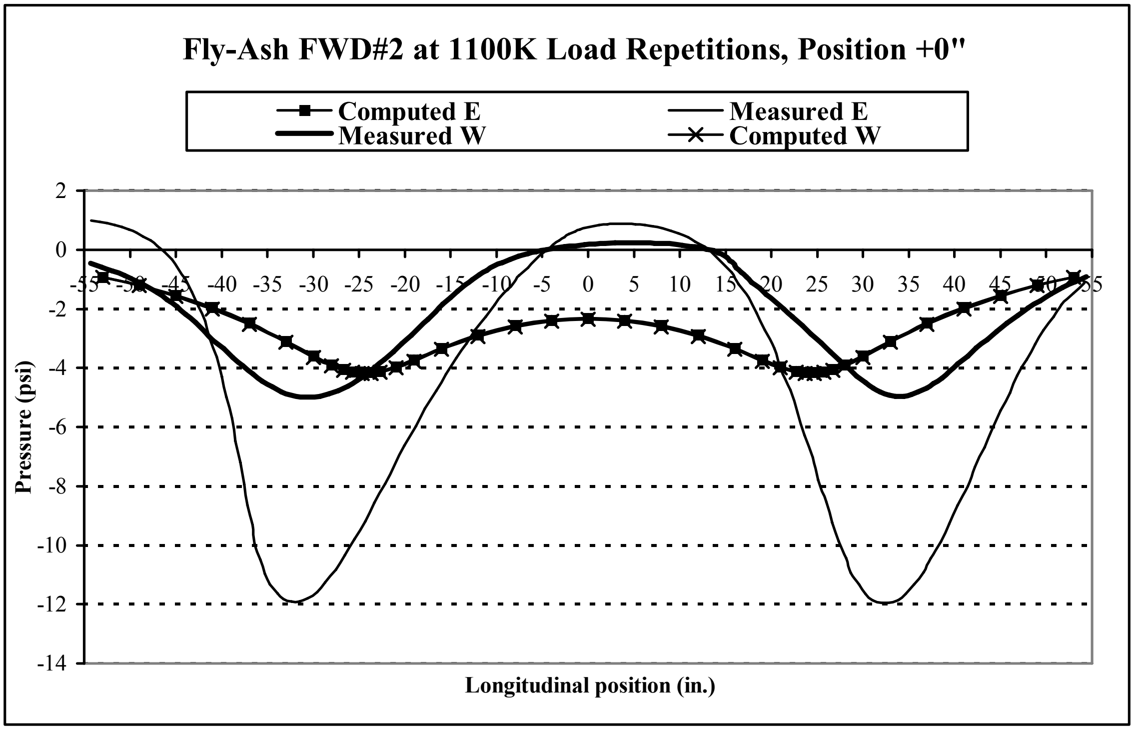


Figure 5.18: Vertical Stress – Section NS – Position +0" (11/20/2003)

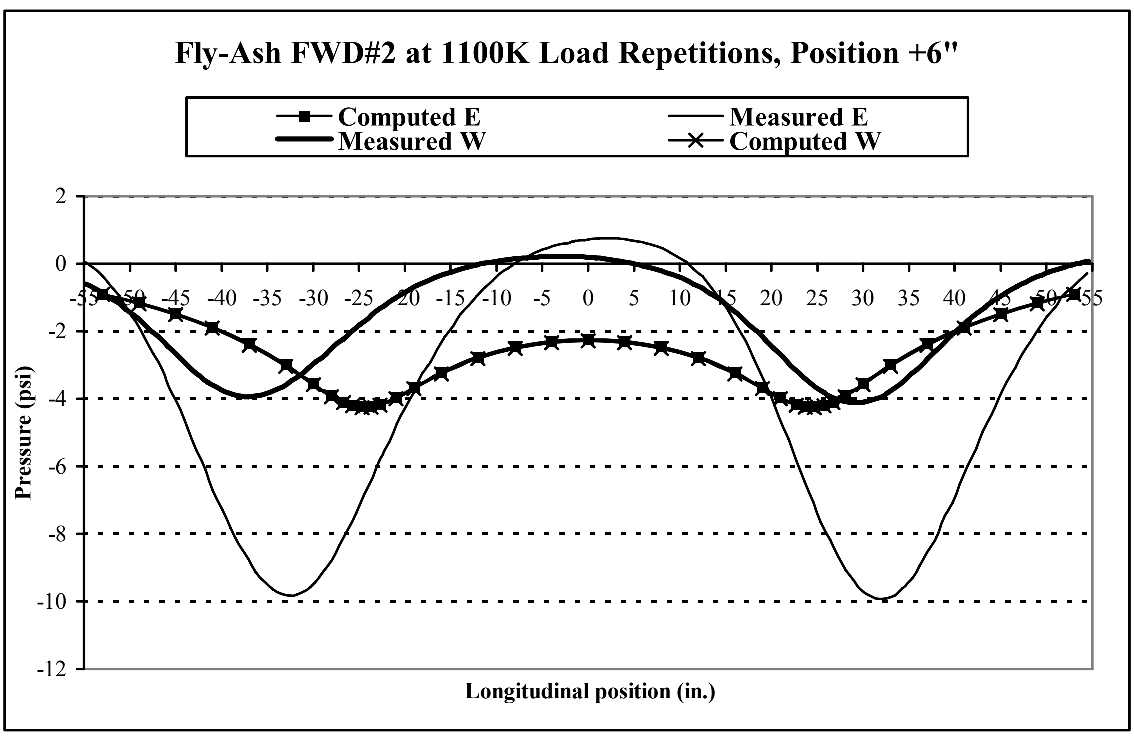


Figure 5.19: Vertical Stress – Section NS – Position +6" (11/20/2003)

5.3.3 The Pavement Section with Lime Stabilized Soil

Figures 5.20 to 5.27 present the measured and calculated longitudinal and transverse strains at the bottom of the asphalt concrete layer and vertical stresses at the top of the subgrade for the pavement section with a lime treated soil subgrade. A tandem axle load was applied to the pavement structure at the dates the FWD tests were performed.

Figures 5.20, 5.21, 5.24 and 5.25 indicate that the shapes of the corresponding computed and measured longitudinal and transverse strain are similar when the wheel passes in Position 0”, but the maximum measured longitudinal strain is about five times smaller than the maximum computed value (Figures 5.20 and 5.21). The measurements were performed on the newly constructed pavement. After the accelerated testing started, the shapes of the strain signals recorded for the wheel in Position 0” are not similar anymore. Also, the maximum measured strains are smaller that the corresponding maximum computed strains (Figures 5.24 and 5.25).

Figures 5.22, 5.23, 5.26 and 5.27 indicate that the shapes of the corresponding computed and measured longitudinal and transverse strain are not similar anymore when the wheel passes in Position +6”. Also, the maximum measured strains are smaller that the corresponding maximum computed strains.

Figures 5.28 to 5.31 reveal similar shapes for the measured and calculated vertical stresses at the top of the subgrade soil layer. The difference between the measurements recorded by the East and West sensors indicate that the East sensor measured the highest stress value and the closest value to that of the computed stress. However, at 800,000 load repetitions, the magnitude vertical stress recorded by the two pressure cells are very close, but they are between 45 and 65 percents of the corresponding computed stresses.

5.3.4 The Pavement Section with EMC^2 Stabilized Soil

Figures 5.32 to 5.35 present the measured and calculated longitudinal strains at the bottom of the asphalt concrete layer and vertical stresses at the top of the subgrade for the pavement with the subgrade layer stabilized with EMC^2. The measurements were performed on the newly constructed pavement. A tandem axle load was applied to measure pavement response.

Figures 5.32 and 5.33 indicate that the shapes of the computed and measured longitudinal strains are similar and the maximum values are relatively close; the maximum measured longitudinal strain being about 40 percent higher than the computed longitudinal strain.

Figures 5.34 and 5.35 indicate that the shapes of the computed and measured vertical stress at the top of the subgrade soil are similar. However, the maximum measured vertical stress is between two and three times higher than the computed vertical stress.

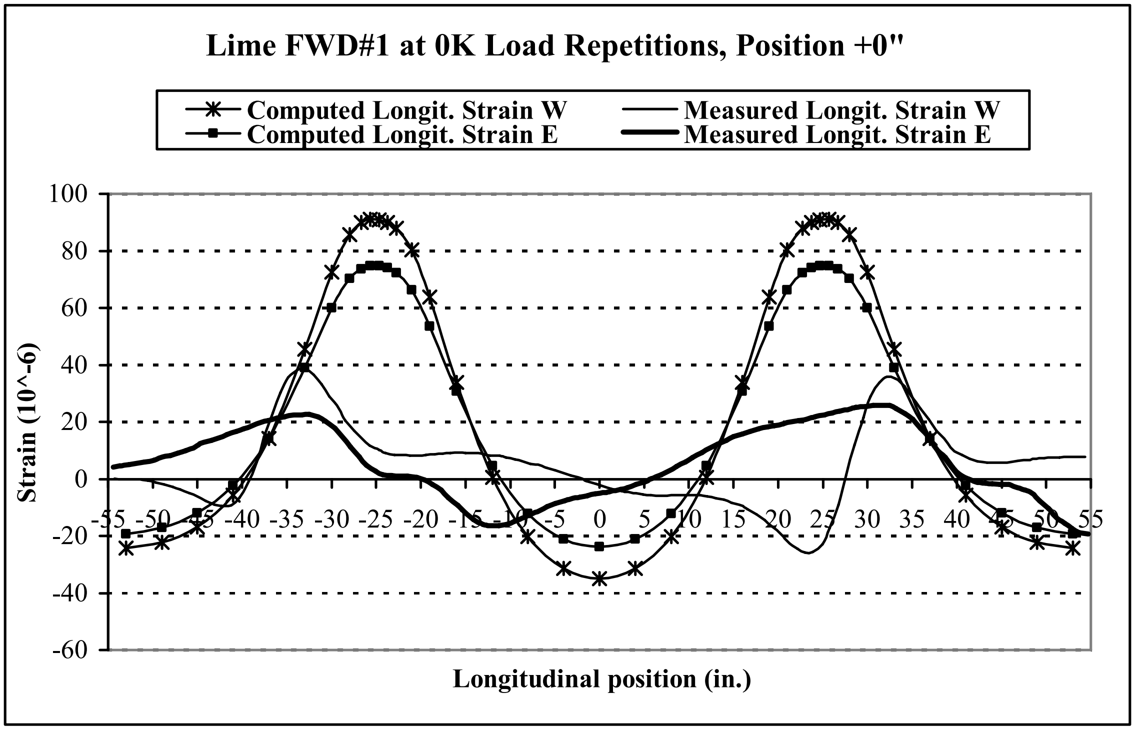


Figure 5.20: Longitudinal Strain– Section SN – Position +0” (05/02/2003)

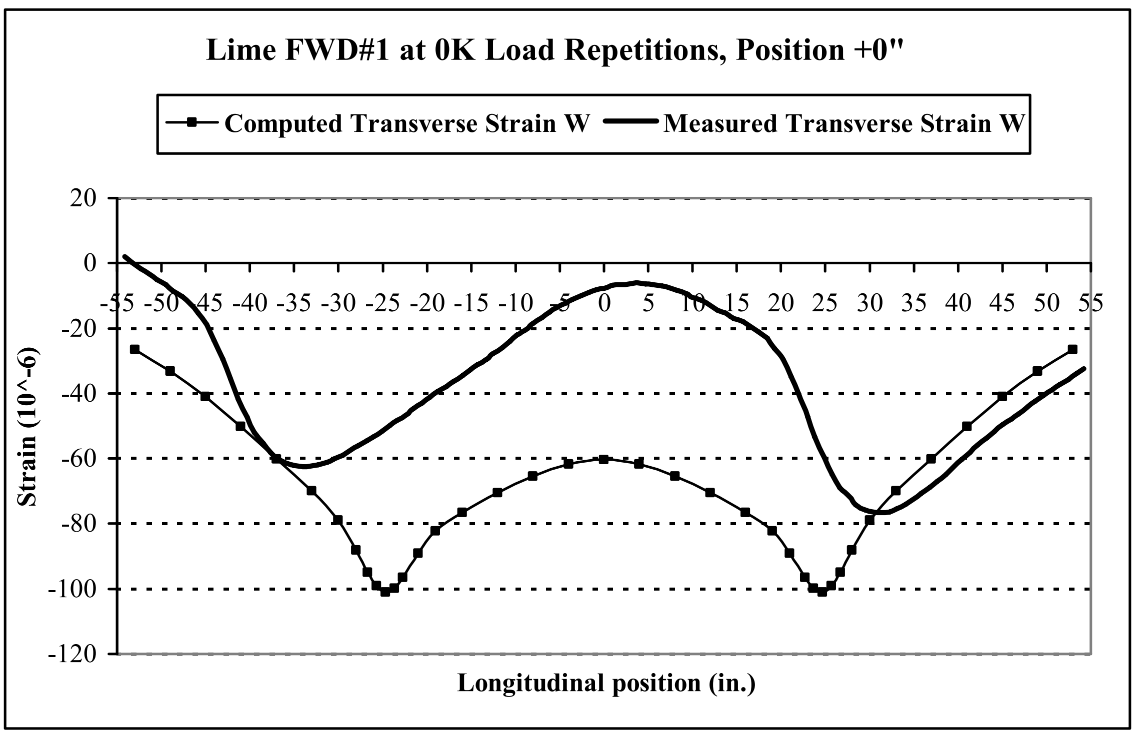


Figure 5.21: Transverse Strain– Section SN – Position +0” (05/02/2003)

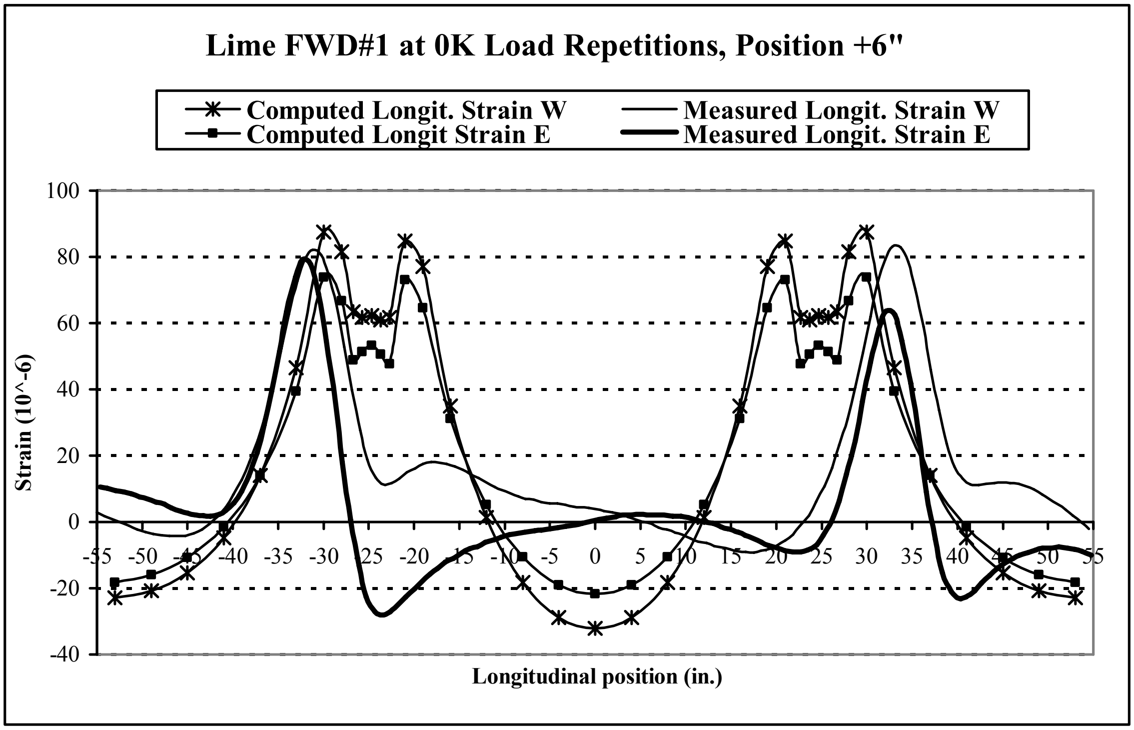


Figure 5.22: Longitudinal Strain – Section SN – Position +6" (05/02/2003)

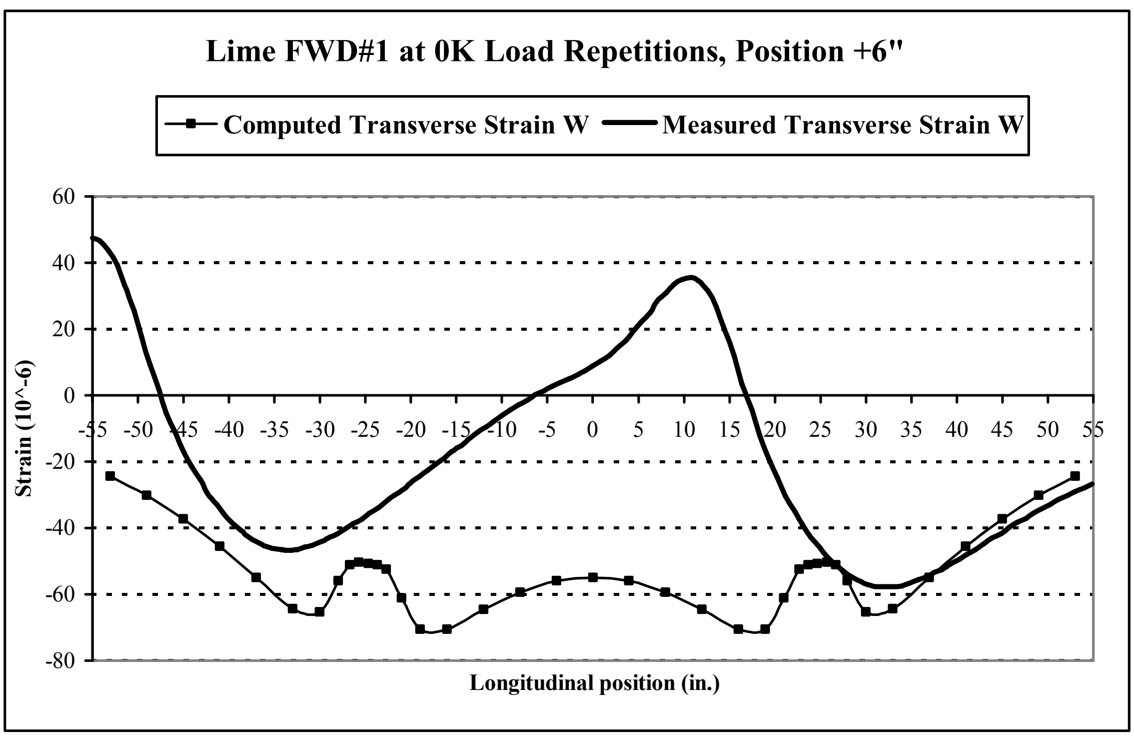


Figure 5.23: Transverse Strain – Section SN – Position +6" (05/02/2003)

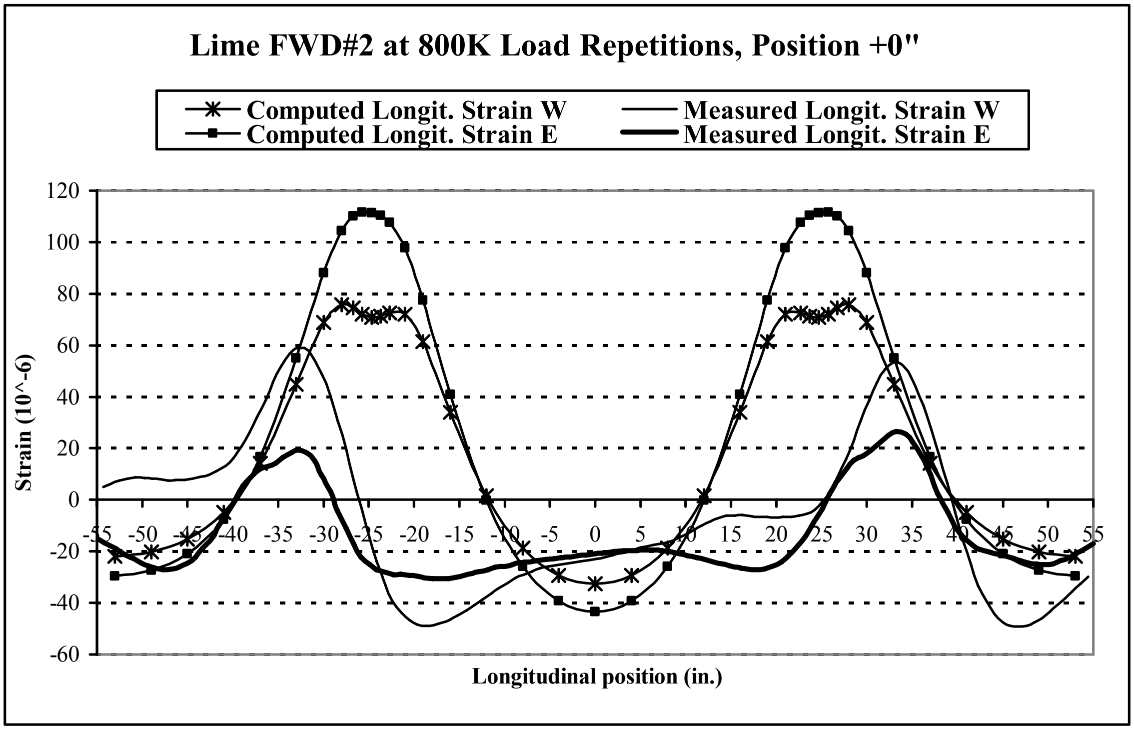


Figure 5.24: Longitudinal Strain – Section SN – Position +0" (10/06/2003)

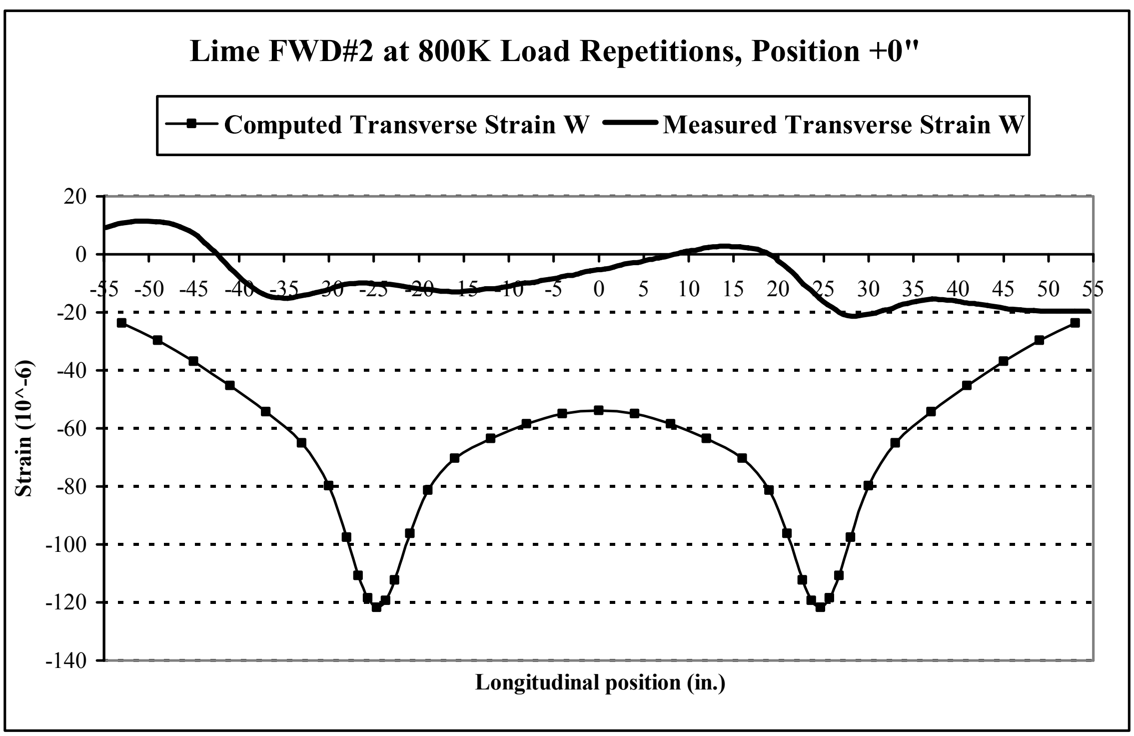


Figure 5.25: Transverse Strain – Section SN – Position +0" (10/06/2003)

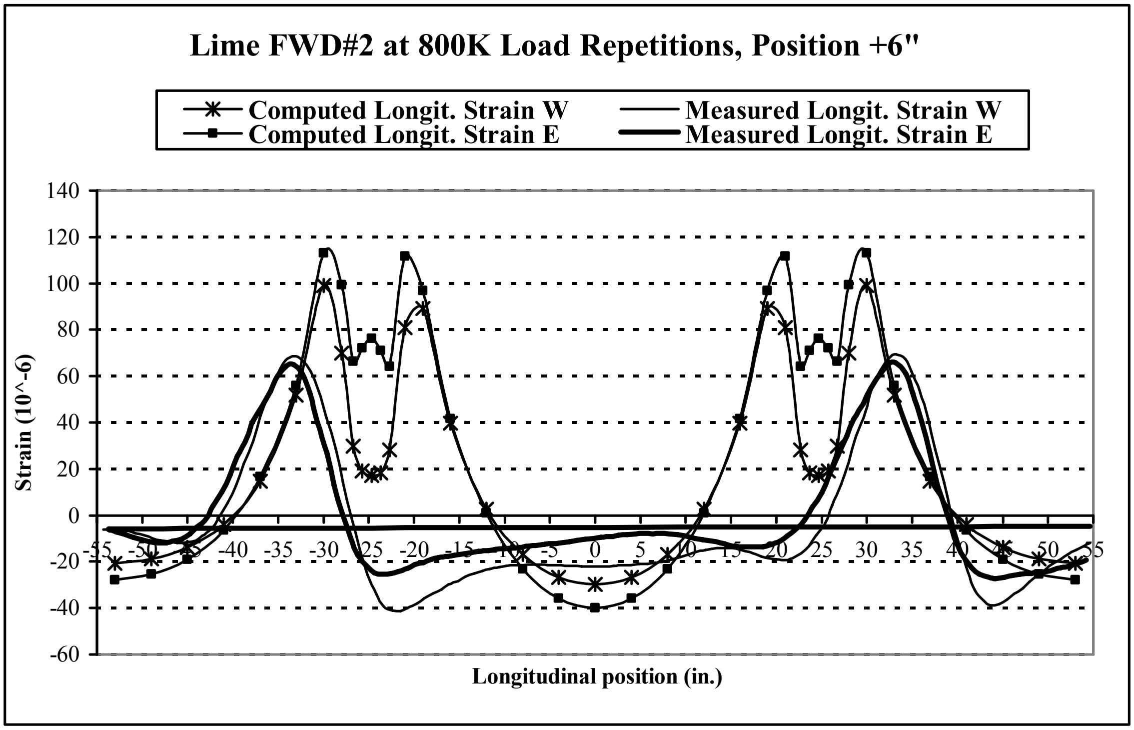


Figure 5.26: Longitudinal Strain – Section SN – Position +6” (10/06/2003)

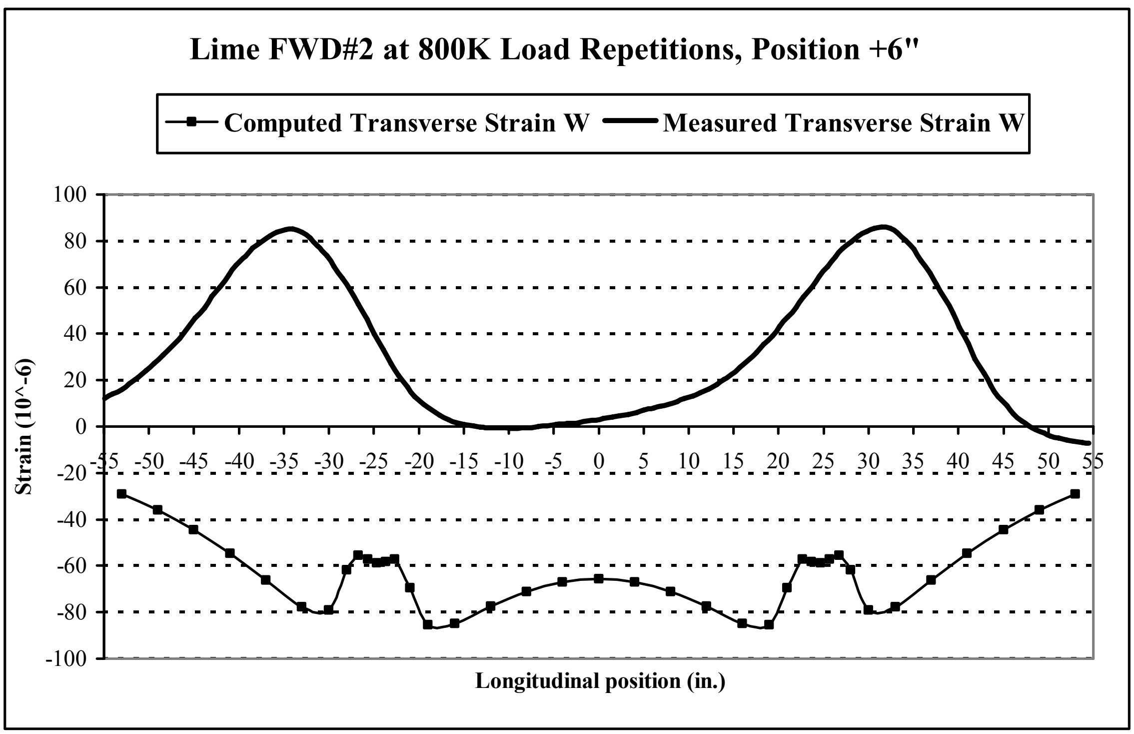


Figure 5.27: Transverse Strain – Section SN – Position +6” (10/06/2003)

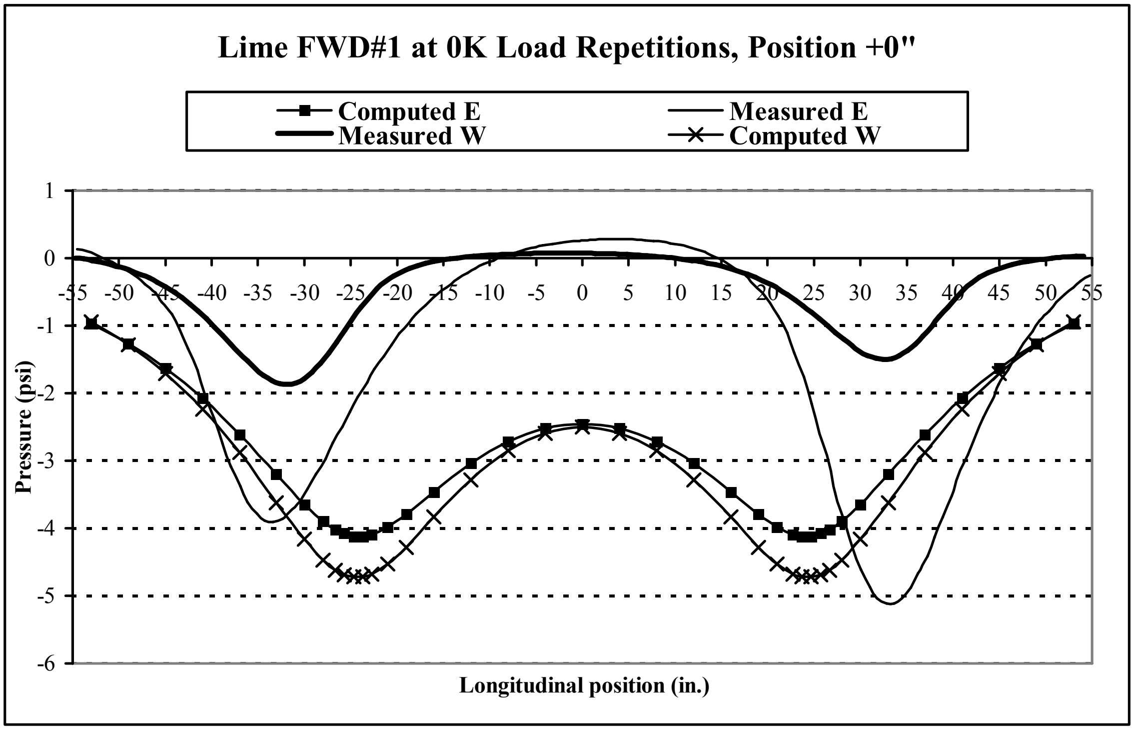


Figure 5.28: Vertical Stress – Section SN – Position +0" (05/02/2003)

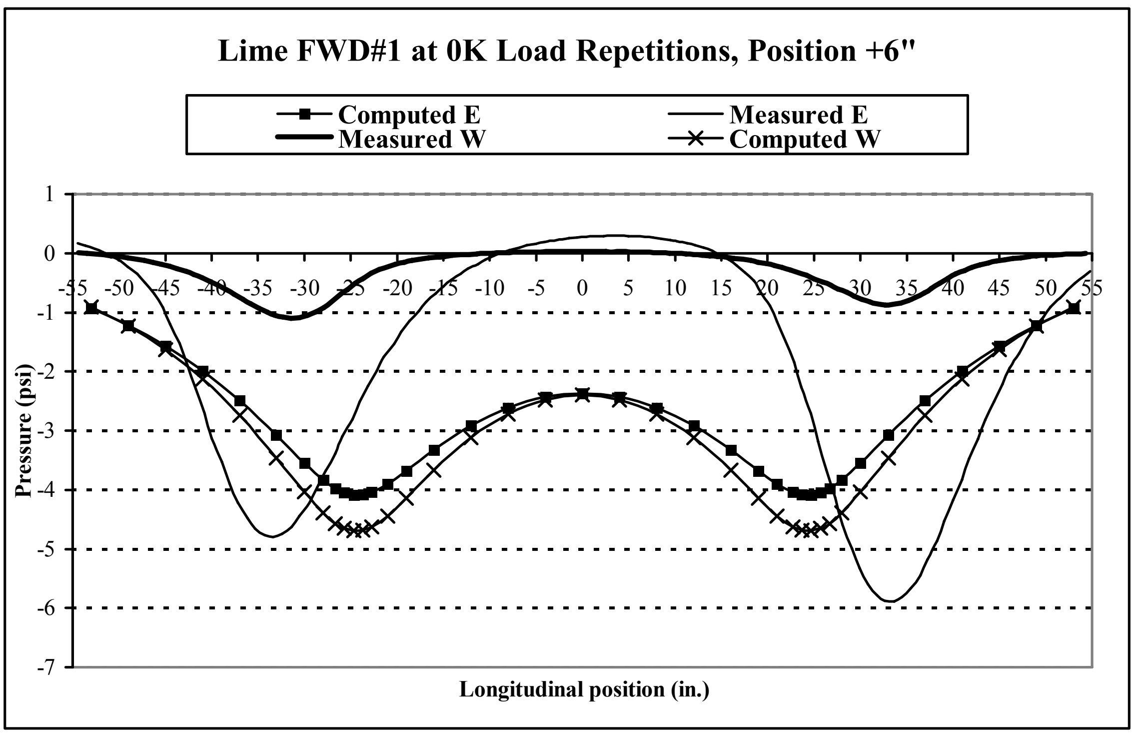


Figure 5.29: Vertical Stress – Section SN – Position +6" (05/02/2003)

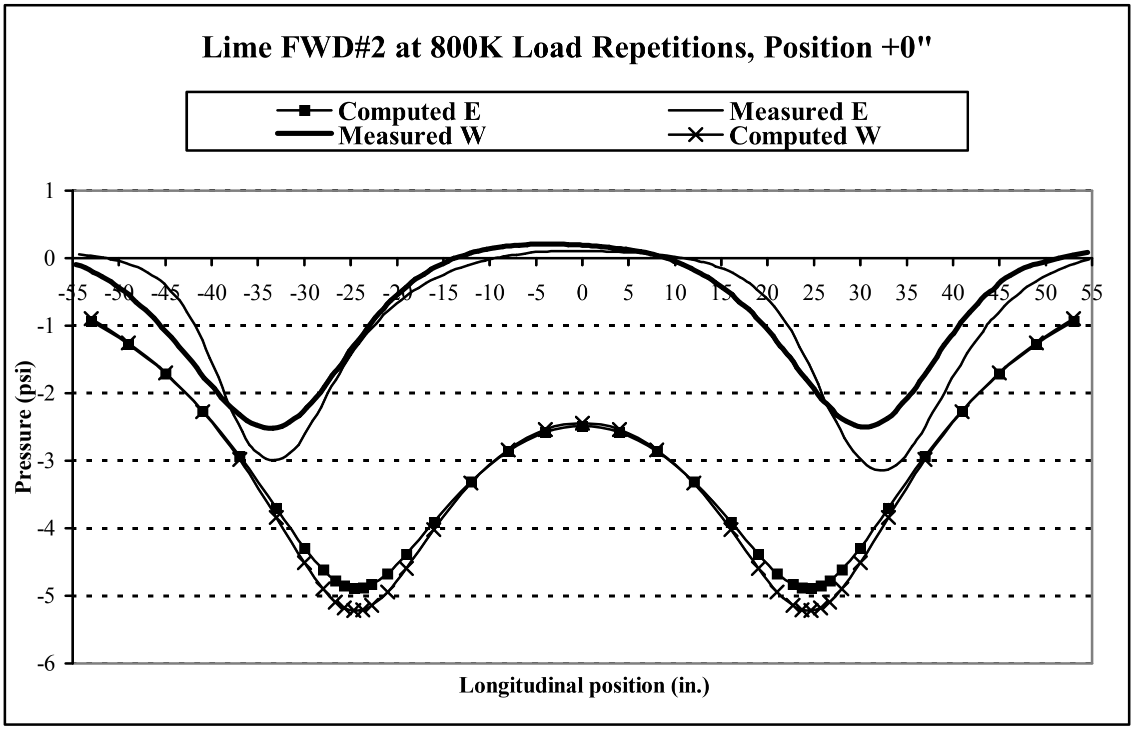


Figure 5.30: Vertical Stress – Section SN – Position +0" (10/06/2003)

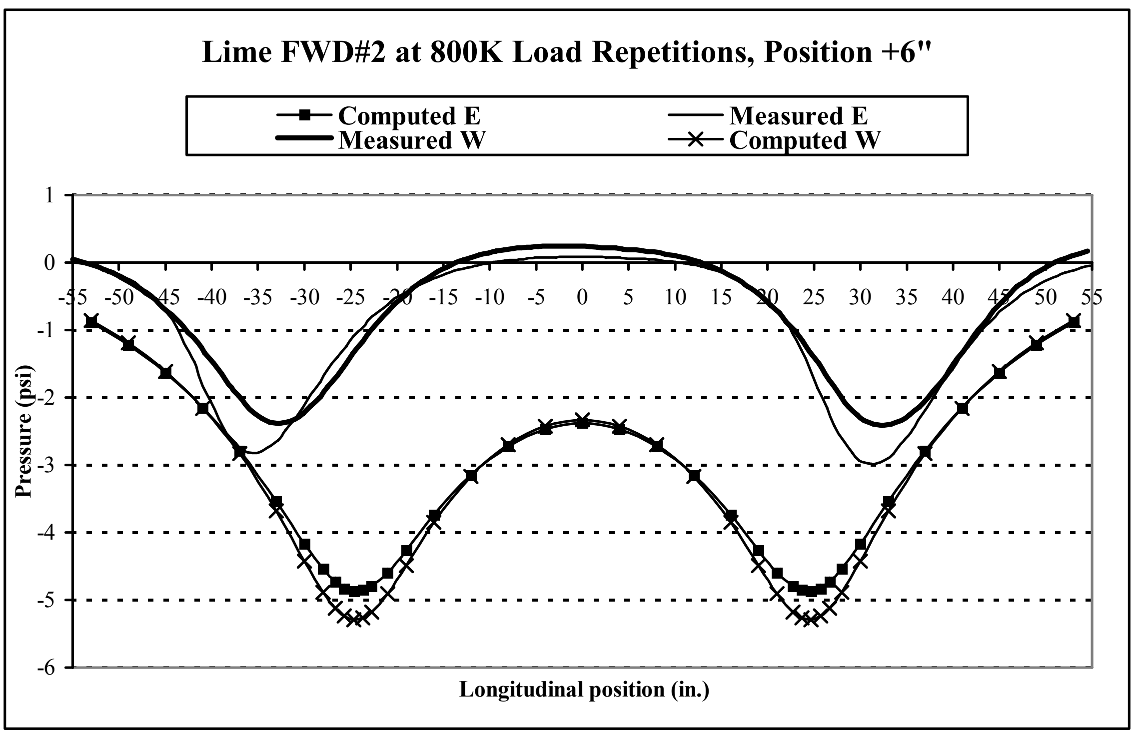


Figure 5.31: Vertical Stress – Section SN – Position +6" (10/06/2003)

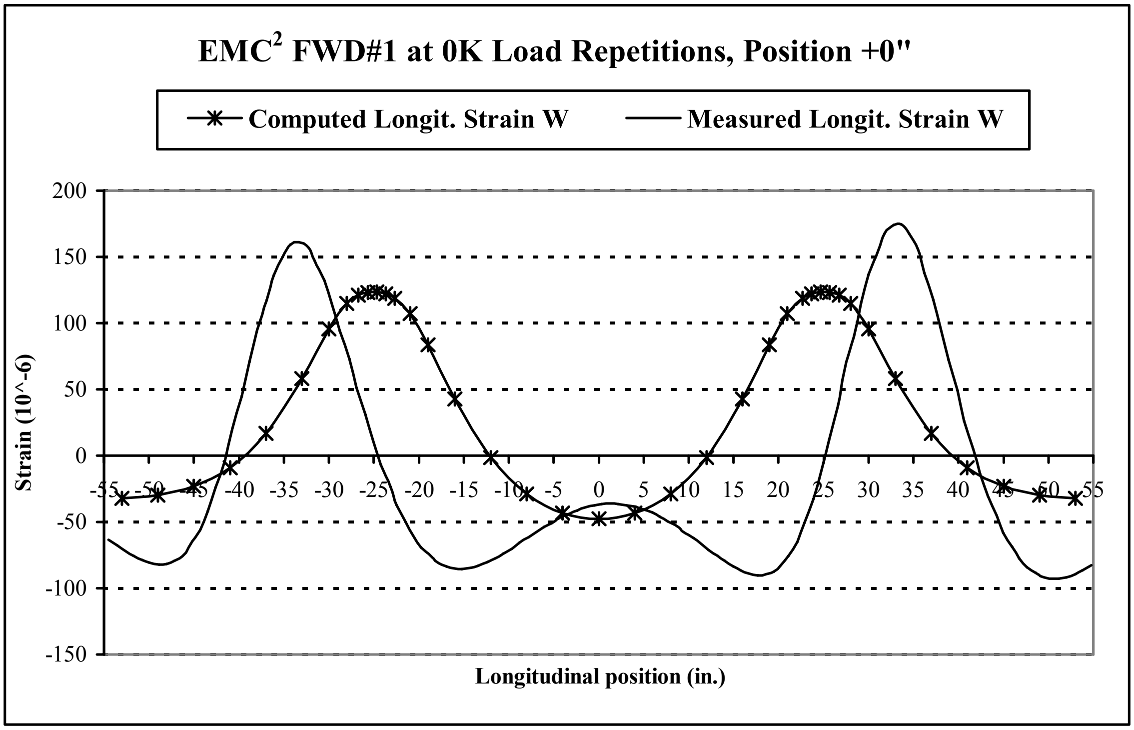


Figure 5.32: Longitudinal Strain – Section SS – Position +0” (05/02/2003)

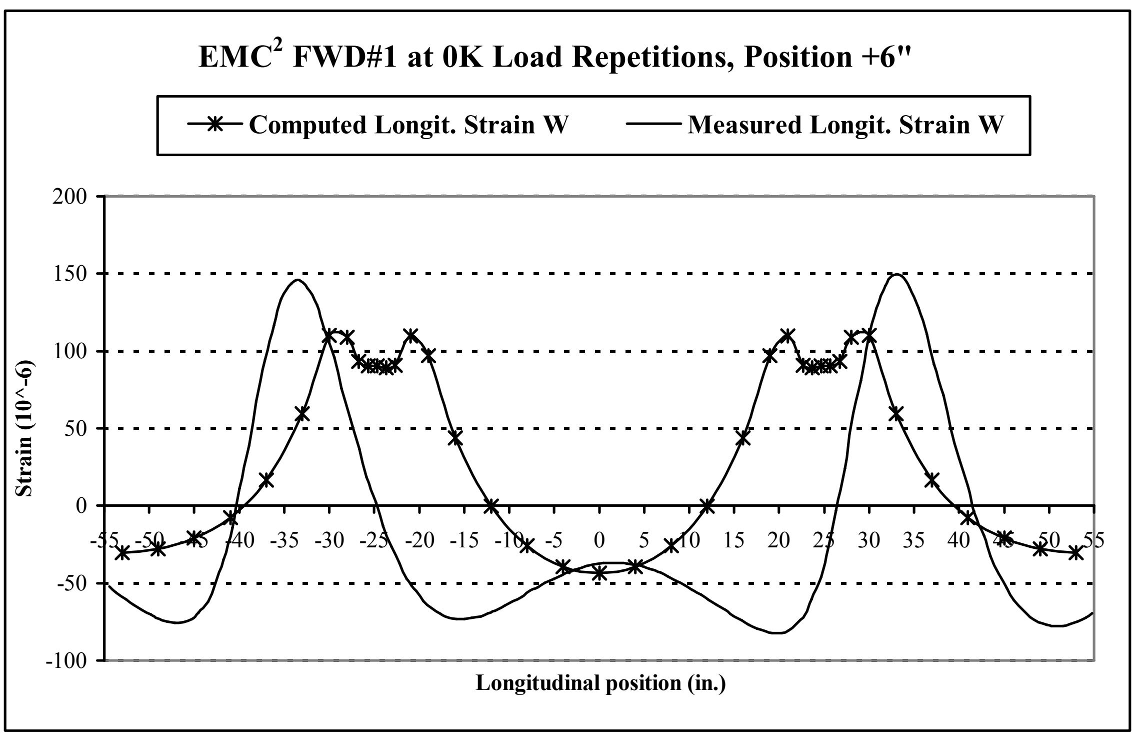


Figure 5.33: Longitudinal Strain – Section SS – Position +6” (05/02/2003)

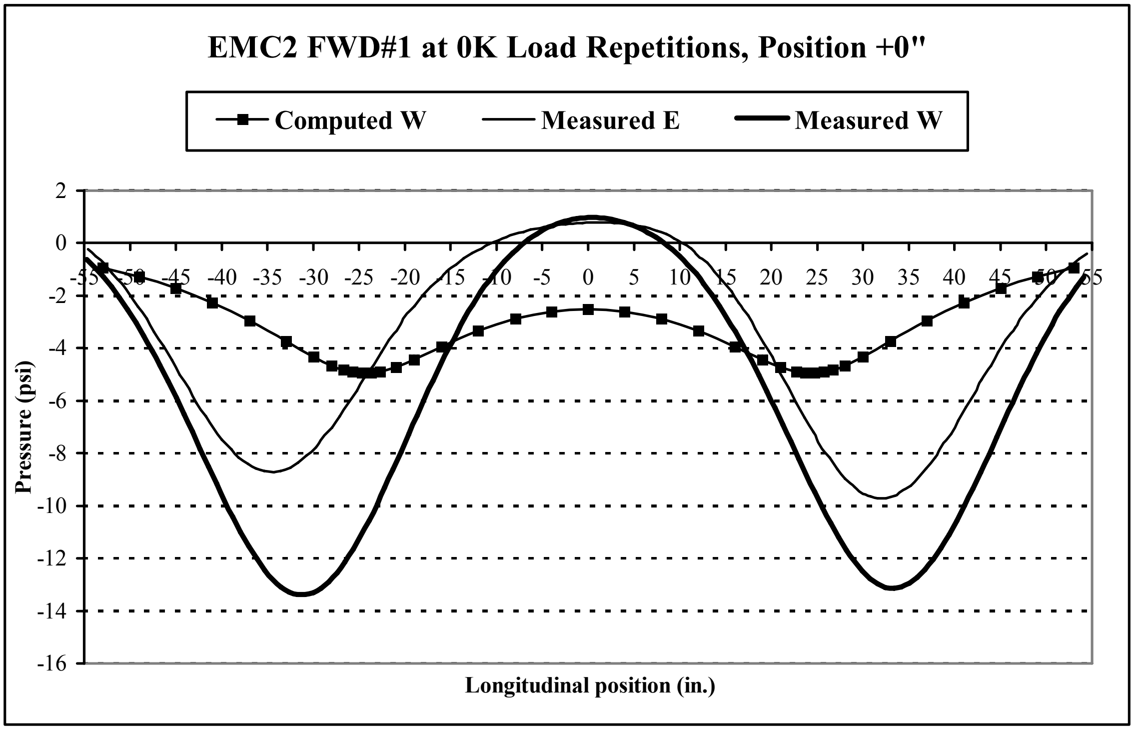


Figure 5.34: Vertical Stress – Section SS – Position +0" (05/02/2003)

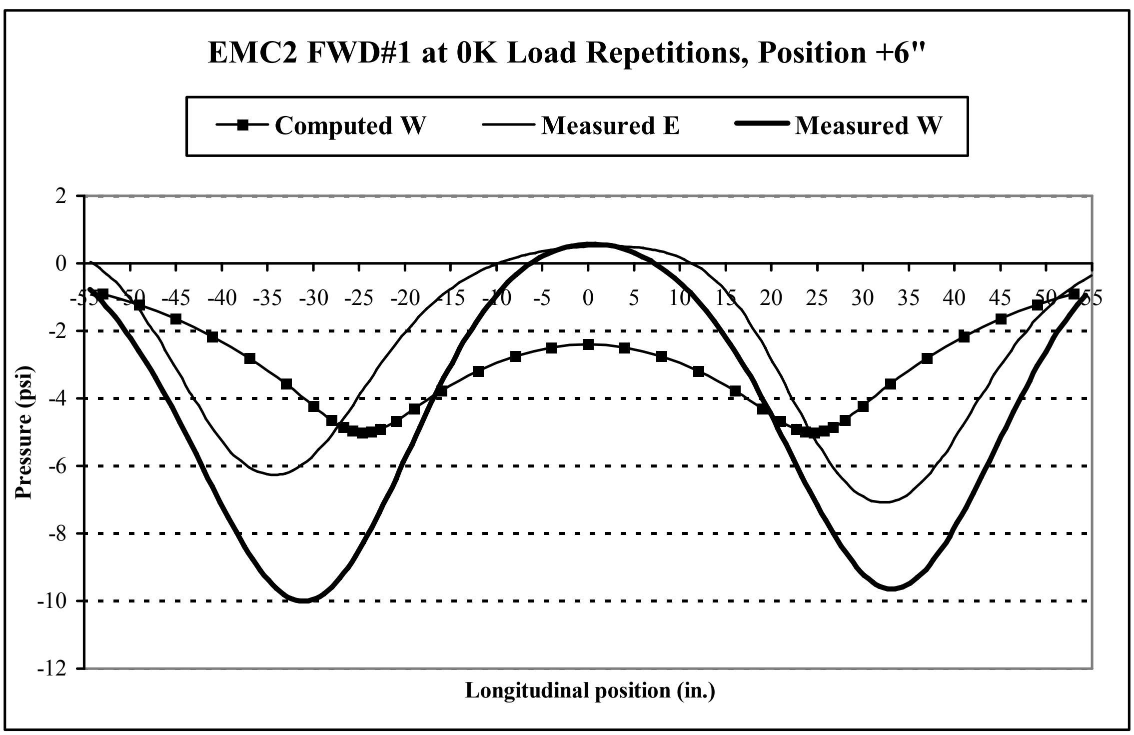


Figure 5.35: Vertical Stress – Section SS – Position +6" (05/02/2003)

CHAPTER 6 - CONCLUSIONS AND RECOMMENDATIONS

The Midwest States Accelerated Pavement Testing Pooled Fund, financed by the highway departments of four Midwestern states, sponsored one Accelerated Pavement Testing (APT) project aimed to investigate the practices related to the design of flexible pavements when the top of the subgrade is improved by chemical stabilization. The experiment was conducted at the Civil Infrastructure Systems Laboratory (CISL) of Kansas State University.

The test program consisted of constructing four flexible pavement structures and subjecting them to full-scale accelerated pavement tests at ambient temperature and moderate moisture condition. The four pavement structures had the same subgrade soil and the same asphalt concrete surface layer. However, the top six inch layer of embankment soil was stabilized with four chemicals: cement, fly-ash, hydrated lime and a commercial product, EMC-Square. The subgrade soil used in the study was a plastic, non-sulfate clay, a soil typical used in the construction of embankment layers under flexible pavements in the four Midwestern states. The APT test was complemented by an extensive laboratory investigation aiming to determine the optimum content of each chemical and the swelling potential reduction due to the chemical stabilization.

The major findings of this research project are:

- Lime was the most effective stabilizers for the non-sulfate bearing clayey soil studied in this research. The lime-soil mixture resulted in the lowest vertical compressive stresses at the top of the unbound clayey subgrade, which indicates that the lime stabilization provided the best protection to the underlying stabilized soil.

- Cement was found to be the second most effective stabilizer. For the first 800,000 of passes of the APT machine, the evolution of rut depth in the at the surface of the pavement section with cement stabilized embankment soil was very close to that measured at the surface of the pavement section with the embankment soil stabilized with lime. However, the HMA layer thicknesses measured with the rod-and-level method as well as those measured during the post-mortem investigation proved that the HMA layer was thinner for the pavement section with the embankment soil stabilized with lime than that of the section with soil-cement embankment.
- The fly ash-treated subgrade soil generated higher vertical compressive stresses at the top of the subgrade and higher rut depth at the pavement surface than the Portland cement and lime- treated subgrade soils. Therefore it is expected that fly-ash is less effective than lime or cement in stabilizing the clay soil.
- The commercial stabilizer proved not to be effective in stabilizing non-sulfate bearing clayey soil. The pavement failed after only 45,000 load repetitions, exhibiting severe fatigue cracking and rutting. The laboratory measured compressive strength of the soil stabilized with the commercial product did not increase with curing time and was very similar to that of the untreated soil.
- For the soil studied and the chemical contents used, hydrated lime, cement and fly-ash reduced significantly the swelling potential of the soil and increased significantly the unconfined compressive strength of the soil. This clearly indicates these chemical stabilizers are effective for improving the engineering properties of the soil.

- The highest compressive strength was recorded for soil-cement followed by that of the soil-lime mixture and of the fly-ash-soil mixture. The soil-lime mixture exhibited an increase in strength even after 90 days of curing; the strength of the soil-cement and soil-fly ash mixture remained almost constant after 90 days of curing.
- The measured vertical compressive stresses at the top of the untreated subgrade soil and the longitudinal and transverse strains at the bottom of the asphalt concrete surface layer were different from the corresponding values computed with EverStress, a linear-elastic pavement structural model. The use of elastic layer moduli backcalculated from the FWD deflections in the response computation might explain the difference between the measured and computed responses.

The major recommendation of this study is to use hydrated lime as the chemical stabilizer for clayey non-sulfate soils with similar properties (plasticity, swelling potential) as those of the soil tested in this research. Hydrated lime improves significantly the engineering properties of the embankment soil; it increases the compressive strength and reduces the swelling potential of the soil. Stabilization with lime leads to better pavement performance than stabilization with cement, even though the soil-cement has higher compressive strength than that of the lime stabilized soil.

The use of EMC-Squared as a chemical stabilizer is not recommended for non-sulfate clay soils. This chemical has no significant impact on the compressive strength of the soil. The strength achieved is much lower than that achieved by lime or cement stabilization.

CHAPTER 7 - REFERENCES

1. Melhem, H.G., *Development of an Accelerated Testing Laboratory for Highway Research in Kansas*, Report No. FHWA-KS-97/5, Kansas Department of Transportation, Topeka, KS, November 1997.

2. Melhem, H.G., *Accelerated Testing for Studying Pavement Design and Performance*; FY97-98, Report No. FHWA-KS-99-2, Kansas Department of Transportation, Topeka, KS, May 1999.

3. Melhem, H.G., Sheffield, F., *Accelerated Testing for Studying Pavement Design and Performance*; FY99, Report No. FHWA-KS-99-7, Kansas Department of Transportation, Topeka, KS, July 2000.

4. Swart, R., Melhem, H.G., *Accelerated Testing for Studying Pavement Design and Performance*; FY2000, Report No. FHWA-KS-XX-X, Kansas Department of Transportation, Topeka, KS, March 2001.

5. Melhem G., Swart R. and S. Walker. *Accelerated Testing For Studying PavementDesign And Performance* (FY 2001): *Evaluation of the Performance of Permeable and Semi-Permeable UnboundGrandular Bases under Portland Cement Concrete Pavement (PCCP) Slabs andAlternate Load Transfer Devices for Joint Repair* FINAL REPORT Report No. FHWA-KS-02-7, Topeka, KS, November 2003.

6. Romanoschi S. A., Hossain M., Lewis P. and O. Dumitru. *Performance Of Foamed Asphalt Stabilized Base In Full-Depth Reclaimed Asphalt Pavement* (ATL EXPERIMENT NO. 11), Final Report, Midwest States Accelerated Pavement Testing Pooled Fund Program, April 2003.

7. Banda, S. *Laboratory Evaluation of Chemical Stabilization of Kansas Embankment Soils*. Master Thesis, Kansas State University, Manhattan, 2004.

8. Soil Stabilization Product Company, Inc., http://sspco.com/emcsquared/emcsquared_OV.html Visited July 2004.

9. AASHTO. *Standard Specifications for Transportation Materials and Methods for Sampling and Testing.*. American Association of State Highway and Transportation Officials, Washington, DC, (2002)

10. ASTM. *Annual Book of ASTM Standards*, ASTM International, West Conshohocken, PA, 2000.

11. Bell, F.G. *Engineering treatment of soils*, First edition, E & FN Spon, London, New York, 1993.

12. PCA. *Soil Cement Construction Handbook*, Engineering Bulletin, Portland Cement Association, Illinois, USA, 1995.

13. Little, D.N. *Stabilization of pavement subgrades and base courses with lime*, Kendall/Hunt Publishing Company, Dubuque, Iowa, 1995.

14. KDOT *Standard Specifications for State Road and Bridge Construction (1990)*, Metric Version, Kansas Department of Transportation, 1990.

15. Parsons R.L., C. P. Johnson, S. A. Cross. *Evaluation of soil modification mixing procedures*, Final Report, Report No. K-TRAN: KU-00-6, Kansas Dept. of Transportation, Topeka, 2001.

16. Scullion T. and C.H. Michalak MODULUS 4.0 User's Manual. Texas Transportation Institute, 1991

17. *Aschenbrener T.* Influence of Refining Processes and Crude Oil Sources Used in Colorado on Results from the Hamburg Wheel-Tracking Device*. Final Report No. CDOT-DTD-R-94-7, Colorado Department of Transportation, April 1994.*

18. EverStress 5.0. http://www.wsdot.wa.gov/TA/Software/). Visited July 2004.

APPENDIX A - RESULTS OF UNCONFINED COMPRESSIVE STRENGTH TESTS

Table A1: UCS of the chemically stabilized soil at 95% of Standard Proctor Dry Density

Stabilizer	Sample No.	Curing Period - days					
		2	7	14	28	90	150
5% Portland Cement	1	143.1	159.4	168.3	195.2	234.6	239.1
	2	128.2	135.3	180.1	202.8	211.9	245.5
	Avg	**135.6**	147.3	174.2	**199.0**	**223.2**	**242.3**
7% Portland Cement	1	177.7	215.4	268.6	309.7	373.4	303.1
	2	171.7	254.5	325.1	345.0	397.2	338.3
	Avg	**174.7**	**234.9**	**296.9**	**327.3**	**385.3**	**320.7**
9% Portland Cement	1	230.2	285.4	335.2	407.3	422.1	438.7
	2	257.8	306.3	301.2	415.7	434.7	462.3
	Avg	**244.0**	**295.9**	**318.2**	**411.5**	**428.4**	**450.5**
4% Lime	1	64.6	94.9	103.7	143.2	242.7	262.5
	2	76.7	103.0	113.1	138.0	221.1	289.8
	Avg	**70.7**	**99.0**	**108.4**	**140.6**	**231.9**	**276.2**
6% Lime	1	99.0	109.7	113.8	156.2	279.0	211.9
	2	93.6	108.4	115.1	137.7	261.5	211.9
	Avg	**96.3**	**109.1**	**114.4**	**146.9**	**270.3**	**211.9**
8% Lime	1	117.1	152.8	164.9	217.6	297.7	417.4
	2	105.0	142.7	153.1	183.4	307.8	417.4
	Avg	**111.1**	**147.8**	**159.0**	**200.5**	**302.8**	**417.4**
12% Fly Ash	1	73.2	80.1	99.0	93.1	93.4	-
	2	66.1	72.4	91.0	97.6	97.9	-
	Avg	**69.7**	**76.2**	**95.0**	**95.3**	**95.7**	**-**
15% Fly Ash	1	74.4	97.4	101.0	107.0	109.7	98.6
	2	82.0	84.8	92.9	98.1	113.3	112.8
	Avg	**78.2**	**91.1**	**96.9**	**102.6**	**111.5**	**105.7**
18% Fly Ash	1	83.6	100.5	106.0	107.2	129.9	-
	2	75.9	98.5	110.4	116.5	107.0	-
	Avg	**79.8**	**99.5**	**108.2**	**111.8**	**118.5**	**-**

Table A2: UCS of the chemically stabilized soil at 100% of Standard Proctor Dry Density

Stabilizer Added	Sample No.	Curing Period - days					
		2	7	14	28	90	150
5% Portland Cement	1	171.7	196.9	235.6	254.1	245.0	302.4
	2	171.7	225.5	227.2	251.8	297.5	302.4
	Avg	**171.7**	**211.2**	**231.4**	**252.9**	**271.3**	**302.4**
7% Portland Cement	1	287.8	356.8	371.9	396.3	450.2	431.5
	2	264.2	365.2	363.5	383.2	450.2	466.5
	Avg	**276.0**	**361.0**	**367.7**	**389.8**	**450.2**	**449.0**
9% Portland Cement	1	390.4	432.2	499.8	565.3	664.6	572.0
	2	378.7	443.5	498.2	516.7	666.3	557.6
	Avg	**384.6**	**437.8**	**499.0**	**541.0**	**665.4**	**564.8**
4% Lime	1	121.2	136.3	158.9	175.9	277.9	374.3
	2	109.4	129.6	145.2	191.9	292.2	334.2
	Avg	**115.3**	**133.0**	**152.1**	**183.9**	**285.0**	**354.3**
6% Lime	1	149.8	143.1	185.3	196.9	316.6	447.7
	2	129.6	156.5	190.2	191.2	315.0	420.7
	Avg	**139.7**	**149.8**	**187.7**	**194.0**	**315.8**	**434.2**
8% Lime	1	186.8	183.4	199.8	203.6	444.0	-
	2	176.7	153.1	178.1	206.8	364.4	-
	Avg	**181.8**	**168.3**	**188.9**	**205.2**	**404.2**	**-**
12% Fly Ash	1	87.7	107.7	107.9	113.8	141.0	-
	2	99.3	105.7	123.0	127.4	152.5	-
	Avg	**93.5**	**106.7**	**115.5**	**120.6**	**146.8**	**-**
15% Fly Ash	1	140.0	**133.6**	151.5	134.8	198.6	199.4
	2	119.2	132.6	144.7	157.7	182.1	218.3
	Avg	**129.6**	**133.1**	**148.1**	**146.2**	**190.3**	**208.9**
18% Fly Ash	1	116.8	145.2	159.0	178.7	211.4	-
	2	104.7	140.5	144.4	185.5	204.1	-
	Avg	**110.7**	**142.9**	**151.7**	**182.1**	**207.8**	**-**

Table A3: UCS of the soil stabilized with EMC SQUARED®– Moist Curing

Moist Curing - Compacted at MDD (Standard Proctor) of Untreated Soil							
95% MDD							
		Curing Period-days					
Moisture Content	Sample No.	2	7	14	28	90	150
18%	1	12.1	12.8	15.0	17.2	-	-
	2	15.5	13.5	13.6	14.6	-	-
	Avg	**13.8**	**13.1**	**14.3**	**15.9**	**-**	**-**
20%	1	24.9	29.6	33.0	32.1	27.1	-
	2	30.3	33.7	31.6	29.1	29.1	-
	Avg	**27.6**	**31.6**	**32.3**	**30.6**	**28.1**	**-**
23%	1	18.8	26.3	22.0	23.6	16.5	25.2
	2	18.8	23.6	23.7	24.9	24.2	26.3
	Avg	**18.8**	**24.9**	**22.9**	**24.2**	**20.4**	**25.7**
100% MDD							
18%	1	23.6	28.6	20.5	27.4	26.1	29.8
	2	30.3	25.2	23.9	25.7	29.3	25.7
	Avg	**26.9**	**26.9**	**22.2**	**26.6**	**27.7**	**27.8**
20%	1	38.7	26.9	31.6	27.3	35.0	25.6
	2	30.3	28.6	36.4	29.3	35.7	21.5
	Avg	**34.5**	**27.8**	**34.0**	**28.3**	**35.3**	**23.6**
23%	1	40.4	38.7	32.6	33.2	44.9	41.9
	2	37.0	32.0	35.8	34.8	43.8	38.4
	Avg	**38.7**	**35.3**	**34.2**	**34.0**	**44.3**	**40.1**
Moist Curing - Compacted at MDD (Modified Proctor) of Untreated Soil							
95% MDD							
		Curing Period-days					
Moisture Content	Sample No.	2	7	14	28	90	150
11%	1	117.8	138.0	112.4	132.1	118.1	110.1
	2	129.6	127.2	125.9	134.3	118.6	121.3
	Avg	**123.7**	**132.6**	**119.2**	**133.2**	**118.4**	**115.7**
14%	1	107.7	106.0	92.6	80.3	99.8	95.8
	2	105.0	102.7	96.3	100.3	114.3	98.3
	Avg	**106.4**	**104.3**	**94.4**	**90.3**	**107.0**	**97.0**
17%	1	74.7	75.7	76.2	67.1	78.4	79.9
	2	75.4	72.4	77.8	66.8	80.3	75.4
	Avg	**75.1**	74.0	**77.0**	**67.0**	**79.4**	**77.7**

Table A4: UCS of the soil stabilized with EMC SQUARED® – Dry Curing for the First Day

MDD	Sample No.	Standard Proctor Curing Period		Modified Proctor Curing Period	
		7 days	14 days	7 days	14 days
	1	101.3	107.0	204.3	170.8
95%	2	116.6	117.3	199.3	174.0
	Avg	**109.0**	**112.2**	**201.8**	**172.4**
	1	187.6	165.4	312.7	217.6
100%	2	177.9	168.8	300.6	202.6
	Avg	**182.8**	**167.1**	**306.6**	**210.1**

Table A5: UCS of the Untreated Soil at seven days

MDD	Sample No.	**Moist Curing**		**Dry Curing for the First Day**	
		Standard Proctor	Modified Proctor	Standard Proctor	Modified Proctor
	1	23.2	46.6	68.0	166.1
95%	2	24.7	42.4	67.0	165.6
	Avg	**24.0**	**44.5**	**67.5**	**165.9**
	1	30.3	85.0	104.8	247.6
100%	2	34.3	81.6	108.0	242.5
	Avg	**32.3**	**83.3**	**106.4**	**245.0**

APPENDIX B - LONGITUDINAL PROFILE ELEVATION DATA

Table B1: Elevation Data for the Longitudinal Profile Lane NN - Cement

	Date / Passes (x 1,000)										
	11/18/02	12/12/02	1/24/03	5/2/03	5/28/03	6/6/03	7/21/03	8/1/03	8/21/03	10/15/03	10/31/03
Station	**Top Soil**	**Top Embank.**	**0**	**100**	**200**	**300**	**400**	**500**	**700**	**800**	**900**
1	-10.344	-4.56	-0.72	-0.72	-0.876	-0.72	-0.912	-0.924	-0.948	-0.96	-0.96
2	-10.884	-4.632	-0.756	-0.78	-0.924	-0.792	-0.984	-0.984	-1.008	-1.032	-1.032
3	-10.584	-4.476	-0.672	-0.72	-0.864	-0.72	-0.912	-0.936	-0.948	-0.96	-0.96
4	-11.016	-4.488	-0.6	-0.624	-0.78	-0.648	-0.816	-0.852	-0.864	-0.888	-0.888
5	-11.064	-4.596	-0.6	-0.588	-0.744	-0.624	-0.792	-0.84	-0.864	-0.876	-0.864
6	-10.68	-4.488	-0.72	-0.708	-0.864	-0.72	-0.912	-0.936	-0.936	-0.972	-0.972
7	-11.136	-4.476	-0.744	-0.696	-0.84	-0.696	-0.9	-0.9	-0.924	-0.948	-0.936
8	-10.884	-4.32	-0.696	-0.708	-0.852	-0.708	-0.9	-0.9	-0.936	-0.948	-0.96
9	-10.764	-3.984	-0.756	-0.72	-0.888	-0.756	-0.936	-0.936	-0.972	-0.984	-0.972
10	-10.404	-3.732	-0.756	-0.792	-0.924	-0.792	-0.996	-0.996	-1.008	-1.032	-1.032
11	-10.488	-3.696	-0.72	-0.732	-0.864	-0.732	-0.924	-0.936	-0.936	-0.972	-0.96
12	-10.308	-3.42	-0.756	-0.732	-0.876	-0.732	-0.912	-0.924	-0.924	-0.96	-0.948
13	-10.272	-3.504	-0.696	-0.672	-0.816	-0.684	-0.864	-0.864	-0.864	-0.888	-0.876
14	-10.368	-3.984	-0.552	-0.54	-0.684	-0.54	-0.732	-0.744	-0.756	-0.756	-0.756
15	-10.416	-4.044	-0.276	-0.432	-0.588	-0.432	-0.624	-0.636	-0.636	-0.648	-0.624
16	-10.188	-3.888	-0.432	-0.408	-0.576	-0.42	-0.6	-0.612	-0.624	-0.624	-0.612
17	-10.224	-4.068	-0.456	-0.456	-0.624	-0.468	-0.648	-0.648	-0.66	-0.66	-0.648
18	-10.452	-4.404	-0.516	-0.516	-0.684	-0.54	-0.708	-0.708	-0.72	-0.732	-0.72
19	-10.284	-4.824	-0.636	-0.624	-0.804	-0.648	-0.804	-0.828	-0.84	-0.828	-0.84
SV			*0.735*	*0.374*	*0.352*	*0.355*	*0.373*	*0.332*	*0.311*	*0.337*	*0.393*

	Date / Passes (x 1,000)										
	11/11/03	11/20/03	12/1/03	12/9/03	1/8/04	1/28/04	2/5/04	2/13/04	2/24/04	3/5/04	3/19/04
Station	**1,000**	**1,100**	**1,200**	**1,300**	**1,385**	**1,485**	**1,585**	**1,685**	**1,785**	**1,885**	**2,000**
1	-0.972	-0.984	-0.984	-0.972	-0.996	-0.972	-0.972	-0.984	-0.972	-0.972	-1.008
2	-1.02	-1.02	-1.02	-0.996	-1.032	-1.044	-1.02	-1.02	-1.02	-1.02	-1.044
3	-0.96	-0.948	-0.96	-0.948	-0.972	-0.972	-0.948	-0.96	-0.96	-0.96	-0.984
4	-0.876	-0.888	-0.888	-0.876	-0.888	-0.9	-0.876	-0.876	-0.876	-0.9	-0.9
5	-0.888	-0.876	-0.864	-0.876	-0.912	-0.888	-0.852	-0.876	-0.864	-0.888	-0.9
6	-0.948	-0.972	-0.972	-0.984	-0.984	-0.996	-0.96	-0.984	-0.972	-0.984	-0.996
7	-0.936	-0.948	-0.96	-0.936	-0.972	-0.984	-0.936	-0.948	-0.96	-0.96	-0.972
8	-0.924	-0.96	-0.96	-0.96	-0.972	-0.972	-0.948	-0.96	-0.96	-0.972	-0.996
9	-0.972	-0.996	-0.984	-0.984	-0.996	-1.008	-0.996	-0.984	-0.984	-0.984	-1.008
10	-1.02	-1.032	-1.032	-1.02	-1.044	-1.056	-1.032	-1.044	-1.056	-1.032	-1.044
11	-0.948	-0.96	-0.948	-0.948	-0.972	-0.984	-0.948	-0.948	-0.96	-0.96	-0.984
12	-0.936	-0.948	-0.948	-0.936	-0.984	-0.984	-0.936	-0.936	-0.948	-0.96	-0.972
13	-0.864	-0.9	-0.876	-0.876	-0.888	-0.888	-0.876	-0.888	-0.864	-0.888	-0.9
14	-0.744	-0.768	-0.756	-0.768	-0.768	-0.78	-0.768	-0.756	-0.756	-0.756	-0.78
15	-0.636	-0.648	-0.636	-0.636	-0.66	-0.66	-0.648	-0.648	-0.624	-0.648	-0.672
16	-0.624	-0.636	-0.624	-0.624	-0.636	-0.648	-0.636	-0.624	-0.624	-0.636	-0.648
17	-0.636	-0.672	-0.66	-0.648	-0.672	-0.672	-0.66	-0.672	-0.66	-0.66	-0.684
18	-0.696	-0.744	-0.708	-0.708	-0.732	-0.732	-0.72	-0.732	-0.72	-0.732	-0.768
19	-0.828	-0.876	-0.84	-0.828	-0.852	-0.84	-0.864	-0.84	-0.852	-0.864	-0.9
SV	*0.32*	*0.345*	*0.353*	*0.327*	*0.327*	*0.354*	*0.369*	*0.355*	*0.399*	*0.344*	*0.345*

Table B2: Elevation Data for the Longitudinal Profile Lane NS – Fly-Ash

	Date / Passes (x 1,000)										
	11/18/02	12/12/02	1/24/03	5/2/03	5/28/03	6/6/03	7/21/03	8/1/03	8/21/03	10/15/03	10/31/03
Station	Top Soil	Top Embank.	0k	100	200	300	400	500	700	800	900
1	-10.668	-5.232	-0.6	-0.576	-0.732	-0.6	-0.78	-0.816	-0.816	-0.852	-0.852
2	-10.584	-5.1	-0.48	-0.48	-0.648	-0.504	-0.708	-0.72	-0.744	-0.78	-0.78
3	-10.452	-4.764	-0.456	-0.408	-0.588	-0.444	-0.66	-0.66	-0.672	-0.72	-0.696
4	-10.26	-4.512	-0.516	-0.492	-0.66	-0.516	-0.72	-0.732	-0.744	-0.792	-0.78
5	-10.836	-4.26	-0.6	-0.54	-0.696	-0.552	-0.756	-0.756	-0.768	-0.804	-0.804
6	-10.44	-4.308	-0.48	-0.468	-0.624	-0.48	-0.684	-0.696	-0.708	-0.744	-0.756
7	-10.056	-4.368	-0.48	-0.42	-0.552	-0.444	-0.636	-0.66	-0.672	-0.696	-0.696
8	-10.356	-4.212	-0.432	-0.396	-0.564	-0.42	-0.612	-0.636	-0.648	-0.684	-0.684
9	-10.764	-4.476	-0.456	-0.396	-0.564	-0.42	-0.612	-0.636	-0.66	-0.696	-0.684
10	-10.656	-4.404	-0.516	-0.48	-0.636	-0.504	-0.708	-0.732	-0.768	-0.804	-0.792
11	-10.404	-4.296	-0.552	-0.492	-0.648	-0.528	-0.732	-0.744	-0.768	-0.804	-0.816
12	-10.2	-4.356	-0.6	-0.552	-0.708	-0.576	-0.768	-0.804	-0.84	-0.852	-0.852
13	-10.62	-3.792	-0.516	-0.492	-0.636	-0.516	-0.708	-0.708	-0.732	-0.78	-0.768
14	-10.716	-3.9	-0.384	-0.312	-0.456	-0.324	-0.516	-0.564	-0.576	-0.588	-0.6
15	-10.464	-3.66	-0.312	-0.264	-0.408	-0.276	-0.468	-0.504	-0.516	-0.552	-0.552
16	-10.272	-3.672	-0.396	-0.372	-0.54	-0.396	-0.576	-0.612	-0.636	-0.66	-0.66
17	-10.476	-3.828	-0.516	-0.444	-0.612	-0.468	-0.66	-0.684	-0.684	-0.72	-0.708
18	-10.536	-4.068	-0.756	-0.696	-0.84	-0.696	-0.876	-0.9	-0.888	-0.936	-0.9
19	-10.644	-4.596	-0.912	-0.84	-0.984	-0.84	-1.02	-1.044	-0.924	-1.056	-1.068
SV			*0.726*	*0.728*	*0.687*	*0.687*	*0.642*	*0.621*	*0.560*	*0.619*	*0.627*

	Date / Passes (x 1,000)										
	11/11/03	11/20/03	12/1/03	12/9/03	1/8/04	1/28/04	2/5/04	2/13/04	2/24/04	3/5/04	3/19/04
Station	1,000	1,100	1,200	1,300	1,385	1,485	1,585	1,685	1,785	1,885	2,000
1	-0.876	-1.116	-0.864	-0.852	-0.888	-0.888	-0.852	-0.864	-0.864	-0.864	-0.888
2	-0.78	-0.756	-0.792	-0.768	-0.792	-0.792	-0.78	-0.792	-0.78	-0.78	-0.816
3	-0.696	-0.72	-0.732	-0.72	-0.732	-0.732	-0.72	-0.744	-0.732	-0.732	-0.78
4	-0.744	-0.78	-0.792	-0.78	-0.816	-0.816	-0.792	-0.804	-0.792	-0.792	-0.816
5	-0.792	-0.804	-0.816	-0.816	-0.84	-0.852	-0.816	-0.828	-0.816	-0.816	-0.852
6	-0.744	-0.756	-0.768	-0.744	-0.78	-0.768	-0.744	-0.78	-0.756	-0.768	-0.78
7	-0.708	-0.708	-0.72	-0.696	-0.72	-0.732	-0.696	-0.708	-0.72	-0.708	-0.732
8	-0.672	-0.684	-0.708	-0.684	-0.696	-0.708	-0.672	-0.696	-0.708	-0.708	-0.708
9	-0.672	-0.696	-0.708	-0.684	-0.72	-0.72	-0.708	-0.72	-0.72	-0.708	-0.744
10	-0.816	-0.828	-0.732	-0.816	-0.852	-0.96	-0.828	-0.828	-0.84	-0.828	-0.864
11	-0.816	-0.828	-0.852	-0.852	-0.9	-0.876	-0.84	-0.852	-0.828	-0.84	-0.876
12	-0.852	-0.876	-0.864	-0.888	-0.948	-0.936	-0.876	-0.9	-0.876	-0.912	-0.924
13	-0.78	-0.804	-0.768	-0.792	-0.804	-0.816	-0.792	-0.804	-0.792	-0.804	-0.828
14	-0.612	-0.624	-0.6	-0.6	-0.612	-0.624	-0.6	-0.636	-0.612	-0.624	-0.648
15	-0.54	-0.564	-0.576	-0.54	-0.564	-0.576	-0.564	-0.576	-0.564	-0.564	-0.588
16	-0.624	-0.66	-0.66	-0.648	-0.672	-0.684	-0.66	-0.672	-0.672	-0.672	-0.708
17	-0.684	-0.696	-0.732	-0.72	-0.72	-0.744	-0.708	-0.744	-0.732	-0.732	-0.756
18	-0.888	-0.9	-0.936	-0.912	-0.948	-0.936	-0.924	-0.924	-0.936	-0.936	-0.948
19	-1.02	-1.056	-1.068	-1.08	-1.08	-1.092	-1.08	-1.08	-1.08	-1.092	-1.092
SV	*0.633*	*1.134*	*0.571*	*0.697*	*0.785*	*0.924*	*0.672*	*0.588*	*0.622*	*0.675*	*0.623*

Table B3: Elevation Data for the Longitudinal Profile Lane SN - Lime

	Date / Passes (x 1,000)										
	11/25/02	12/11/02	1/24/03	5/7/03	6/20/03	7/11/03	8/29/03	9/9/03	9/18/03	9/26/03	10/6/03
Station	**Top Soil**	**Top Embank.**	**0**	**45**	**200**	**300**	**400**	**500**	**600**	**700**	**800**
1	-10.176	-3.552	-0.828	-0.996	-1.044	-1.056	-0.9	-1.14	-1.14	-1.176	-1.152
2	-10.104	-3.648	-1.008	-1.092	-1.188	-1.188	-1.032	-1.272	-1.26	-1.284	-1.296
3	-10.644	-3.612	-1.128	-1.32	-1.356	-1.392	-1.224	-1.452	-1.44	-1.476	-1.452
4	-9.96	-3.528	-1.332	-1.428	-1.488	-1.488	-1.332	-1.572	-1.548	-1.596	-1.572
5	-10.224	-3.408	-1.284	-1.392	-1.452	-1.464	-1.308	-1.536	-1.536	-1.56	-1.536
6	-10.272	-3.672	-1.284	-1.392	-1.464	-1.476	-1.32	-1.56	-1.536	-1.56	-1.548
7	-10.092	-3.792	-1.332	-1.428	-1.476	-1.488	-1.32	-1.56	-1.536	-1.584	-1.56
8	-9.996	-3.792	-1.368	-1.464	-1.524	-1.536	-1.368	-1.608	-1.596	-1.608	-1.608
9	-10.02	-4.152	-1.428	-1.56	-1.608	-1.632	-1.464	-1.704	-1.692	-1.728	-1.716
10	-9.996	-4.248	-1.608	-1.716	-1.74	-1.752	-1.56	-1.812	-1.812	-1.836	-1.812
11	-10.164	-4.332	-1.692	-1.788	-1.824	-1.86	-1.68	-1.896	-1.908	-1.92	-1.932
12	-10.2	-4.272	-1.764	-1.848	-1.884	-1.896	-1.716	-1.944	-1.944	-1.968	-1.968
13	-10.164	-4.176	-1.728	-1.8	-1.848	-1.836	-1.668	-1.92	-1.92	-1.92	-1.92
14	-10.092	-4.092	-1.608	-1.656	-1.704	-1.716	-1.548	-1.788	-1.776	-1.8	-1.8
15	-10.284	-4.212	-1.572	-1.644	-1.68	-1.68	-1.512	-1.764	-1.752	-1.776	-1.776
16	-10.128	-4.416	-1.572	-1.632	-1.668	-1.692	-1.512	-1.752	-1.752	-1.788	-1.764
17	-9.672	-4.416	-1.608	-1.692	-1.716	-1.752	-1.56	-1.812	-1.8	-1.824	-1.824
18	-10.116	-4.632	-1.428	-1.476	-1.548	-1.536	-1.38	-1.644	-1.62	-1.656	-1.656
19	-10.56	-5.016	-1.248	-1.284	-1.356	-1.32	-1.2	-1.428	-1.452	-1.428	-1.428
SV			*0.882*	*0.919*	*0.777*	*0.901*	*0.748*	*0.781*	*0.718*	*0.801*	*0.804*

Table B4: Elevation Data for the Longitudinal Profile Lane SS

	Date / Passes			
	11/25/02	12/11/02	1/24/03	5/7/03
Station	**Top Soil**	**Top Embank.**	**0**	**45,000**
1	-11.016	-4.152	-0.948	-1.452
2	-10.008	-4.092	-0.948	-1.788
3	-9.66	-3.972	-0.924	-2.016
4	-9.672	-3.972	-0.948	-1.956
5	-10.368	-4.416	-1.044	-1.632
6	-10.284	-4.728	-1.128	-1.56
7	-9.96	-4.728	-1.248	-1.62
8	-9.936	-5.112	-1.368	-1.668
9	-10.116	-4.92	-1.488	-1.8
10	-9.6	-4.932	-1.644	-1.896
11	-9.72	-5.16	-1.764	-2.016
12	-10.38	-5.232	-1.848	-1.908
13	-10.044	-5.376	-1.908	-2.112
14	-9.624	-5.172	-1.932	-2.148
15	-10.224	-4.812	-1.884	-2.088
16	-10.308	-4.968	-1.764	-1.992
17	-10.5	-5.088	-1.548	-1.752
18	-10.488	-5.052	-1.212	-1.44
19	-10.56	-4.68	-1.092	-1.332
SV			*1.226*	*2.283*

APPENDIX C - HORIZONTAL STRAINS AT THE BOTTOM OF THE ASPHALT CONCRETE LAYER

Table C1: Longitudinal Strains at the Bottom of the Asphalt Layer

Section	Location	Date	Passes (x 1,000)	Signal Type	Values					Strain (10^{-6})
					A	B	C	D	E	
Position 0”										
NS	W	25-Mar-03	0	2	-246.4	-244.1	2.2	-239.0	-238.3	-244.2
NS	W	30-Apr-03	100	2	-1363.6	-1369.2	55.9	-1337.0	-1284.0	-1394.4
SS	E	2-May-03	0	2	-161.1	-174.4	-4.4	-151.2	-149.8	-154.7
SS	E	5-May-03	45	2	-238.7	-240.7	-28.4	-155.3	-130.8	-163.0
Position +6”										
SN	E	2-May-03	0	2	-21.8	-21.4	7.0	-63.7	-79.2	-53.5
SN	W	2-May-03	0	2	-43.8	-53.8	8.1	83.1	82.1	8.8
SN	W	5-May-03	45	2	-69.8	-142.2	-34.7	-111.9	-218.5	-100.9
SS	E	2-May-03	0	2	-145.9	-148.6	-6.0	-141.0	-136.8	-137.1
SS	E	5-May-03	45	2	-281.3	-266.2	-31.8	-233.8	-208.3	-215.6

Table C2: Transverse Strains at the Bottom of the Asphalt Layer – Position 0”

Section	Location	Date	Passes (x 1,000)	Signal Type	Value					Strain (10^{-6})
					A	B	C	D	E	
NN	E	17-Oct-03	800	2	-59.4	-58.1	9.3	-78.6	-68.8	-75.5
NN	E	31-Oct-03	900	2	-57.6	-57.8	8.1	-81.6	-68.5	-74.5
NN	E	9-Dec-03	1300	2	-62.9	-69.1	7.9	-96.1	-89.5	-87.3
NN	E	28-Jan-04	1485	1	-68.5		-5.4		-90.2	-74.0
NN	E	6-Feb-04	1585	1	-78.6		-5.7		-97.9	-82.6
NN	E	13-Feb-04	1685	1	-87.4		-4.7		-96.5	-87.3
NN	E	24-Feb-04	1785	1	-68.2		-14.4		-92.0	-65.7
NN	E	5-Mar-04	1885	1	-86.1		-4.2		-102.1	-89.9
NS	E	17-Oct-03	800	2	-39.5	-28.7	2.1	-33.4	-21.9	-33.0
NS	E	31-Oct-03	900	2	-40.9	-31.4	1.7	-33.7	-22.7	-33.9
NS	E	11-Nov-03	1000	2	-34.8	-23.7	1.1	-29.4	-17.0	-27.3
NS	E	20-Nov-03	1100	2	-38.0	-19.7	-2.2	-34.9	-15.5	-24.8
NS	E	9-Dec-03	1300	2	-31.4	-25.7	3.5	-36.6	-24.7	-33.1
NS	E	28-Jan-04	1485	1	-36.6		0.9		-36.2	-37.3
NS	E	6-Feb-04	1585	1	-34.3		1.2		-39.8	-38.3
NS	E	13-Feb-04	1685	1	-39.1		2.5		-54.4	-49.3
NS	E	24-Feb-04	1785	1	-32.4		-1.0		-32.8	-31.6
NS	E	5-Mar-04	1885	1	-28.3		0.0		-33.9	-31.1
NS	E	16-Mar-04	2000	1	-33.1		0.0		-32.5	-32.8
SN	W	2-May-03	0	1	61.9	76.4	-14.1	53.2	68.8	79.2
SN	W	5-May-03	45	1	98.5	113.5	-20.6	126.0	163.9	146.1
SN	W	29-Aug-03	400	1	71.8	94.3	-19.3	83.5	111.7	109.6
SN	W	26-Sep-03	700	1	37.4	51.4	-12.8	38.8	59.5	59.6

Table C3: Transverse Strains at the Bottom of the Asphalt Layer– Position +6”

Section	Location	Date	Passes (x 1,000)	Signal Type	Value					Strain (10^{-6})
					A	B	C	D	E	
NN	E	30-Apr-03	100	2	-53.2	-70.0	-2.8	-50.8	-49.9	-53.2
NN	E	28-May-03	200	2	-66.2	-81.8	-2.1	-64.6	-63.7	-67.0
NN	E	5-Jun-03	300	2	-49.1	-62.8	-2.2	-52.4	-56.6	-53.0
NN	E	21-Jul-03	400	2	-116.0	-132.3	13.4	-116.3	-117.9	-134.0
NN	E	1-Aug-03	500	2	-126.7	-140.8	11.4	-125.9	-124.8	-141.0
NN	E	21-Aug-03	700	2	-90.0	-105.9	-0.5	-92.1	-85.7	-92.9
NN	E	17-Oct-03	800	2	-76.6	-84.4	11.4	-84.2	-87.4	-94.6
NN	E	31-Oct-03	900	2	-73.2	-80.8	8.9	-83.7	-82.8	-89.0
NN	E	11-Nov-03	1000	2	-57.8	-60.0	4.3	-71.5	-74.1	-70.2
NN	E	20-Nov-03	1100	2	-81.4	-82.2	5.4	-90.4	-93.1	-92.2
NN	E	9-Dec-03	1300	2	-78.2	-83.3	6.8	-95.8	-97.2	-95.4
NN	E	28-Jan-04	1485	2	-90.9		3.9		-101.2	-100.0
NN	E	6-Feb-04	1585	2	-93.8		-0.8		-100.7	-96.5
NN	E	13-Feb-04	1685	2	-92.1		-0.4		-97.2	-94.3
NN	E	24-Feb-04	1785	2	-115.7		-1.0		-117.1	-115.4
NN	E	5-Mar-04	1885	2	-101.3		-0.1		-106.8	-104.0
NN	E	16-Mar-04	2000	2	-93.5		2.8		-102.3	-100.7
NS	E	30-Apr-03	100	2	-209.5	-204.5	19.5	-213.8	-198.0	-226.0
NS	E	28-May-03	200	2	-173.3	-172.1	10.8	-184.9	-163.2	-184.2
NS	E	5-Jun-03	300	2	-208.9	-204.0	18.1	-208.9	-203.6	-224.5
NS	E	21-Jul-03	400	2	-169.4	-173.3	15.8	-178.6	-171.5	-189.0
NS	E	1-Aug-03	500	2	-154.4	-148.5	13.0	-153.3	-150.8	-164.8
NS	E	21-Aug-03	700	2	-168.2	-163.2	13.8	-172.8	-164.5	-181.0
NS	E	17-Oct-03	800	2	-112.2	-117.3	12.1	-115.0	-116.7	-127.4
NS	E	31-Oct-03	900	2	-105.3	-109.4	11.0	-111.0	-110.4	-120.0
NS	E	11-Nov-03	1000	2	-99.8	-106.0	11.0	-105.5	-103.8	-114.8
NS	E	20-Nov-03	1100	2	-124.2	-128.5	12.6	-129.9	-129.5	-140.6
NS	E	9-Dec-03	1300	2	-112.7	-114.0	11.0	-118.0	-116.5	-126.3
NS	E	28-Jan-04	1485	2	-102.0		6.7		-112.2	-113.8
NS	E	6-Feb-04	1585	2	-117.5		8.2		-125.0	-129.5

Table C3: Transverse Strains at the Bottom of the Asphalt Layer– Position +6” (continued)

Section	Location	Date	Passes (x 1,000)	Signal Type	Value					Strain (10^{-6})
					A	B	C	D	E	
NS	E	5-Mar-04	1885	2	-103.0		4.4		-115.2	-113.5
NS	E	16-Mar-04	2000	2	-95.5		3.4		-106.9	-104.6
NS	W	30-Apr-03	100	2	-175.6	-179.0	22.8	-199.2	-197.7	-210.7
NS	W	28-May-03	200	2	-139.2	-141.8	16.6	-185.5	-179.8	-178.2
SN	W	2-May-03	0	2	-47.4	-35.6	-11.6	-20.2	-3.0	-15.0
SN	W	5-May-03	45	2	-109.8	-97.1	19.6	-129.1	-128.6	-135.8
SN	W	20-Jun-03	200	2	-105.0	-95.3	12.6	-124.9	-114.8	-122.6
SN	W	11-Jul-03	300	2	-113.1	-101.4	1.7	-118.9	-101.9	-110.5
SN	W	29-Aug-03	400	2	-125.6	-118.9	6.3	-126.1	-113.3	-127.3
SN	W	9-Sep-03	500	2	-116.1	-114.2	8.6	-121.0	-118.3	-126.0
SN	W	18-Sep-03	600	2	-109.1	-106.3	9.6	-108.5	-109.0	-117.8
SN	W	26-Sep-03	700	2	-102.6	-99.3	11.1	-104.9	-100.7	-113.0
SN	W	6-Oct-03	800	2	-82.7	-82.7	10.4	-86.0	-84.9	-94.5

APPENDIX D - VERTICAL COMPRESSIVE STRESS AT THE TOP OF THE SOIL SUBGRADE

Table D1: Vertical Stress at the Top of the Soil Subgrade – Position 0”

Section	Sensor	Date	Passes (x 1000)	Signal Type	Values					Stress (psi)
					A	B	C	D	E	
NN	W	25-Mar-03	0	1	3.38	3.73	-0.54	3.43	3.78	4.1
NN	E	25-Mar-03	0	1	5.84	6.02	-0.42	5.04	5.07	5.9
NN	W	30-Apr-03	100	1	4.39	4.77	-0.39	4.60	4.70	5.0
NN	E	30-Apr-03	100	1	4.51	4.81	-0.43	4.12	4.24	4.9
NN	W	28-May-03	200	1	5.96	6.22	-0.58	6.09	6.10	6.7
NN	E	28-May-03	200	1	3.98	4.18	-0.36	3.47	3.75	4.2
NN	W	5-Jun-03	300	1	6.06	6.35	-0.61	6.20	6.23	6.8
NN	E	5-Jun-03	300	1	3.95	4.20	-0.36	3.62	3.78	4.2
NN	W	21-Jul-03	400	1	6.79	7.14	-0.70	7.05	6.90	7.7
NN	E	21-Jul-03	400	1	4.86	5.14	-0.46	4.46	4.76	5.3
NN	W	1-Aug-03	500	1	6.89	7.15	-0.69	6.90	6.95	7.7
NN	E	1-Aug-03	500	1	3.75	4.00	-0.35	3.38	3.72	4.1
NN	W	21-Aug-03	700	1	7.64	7.82	-0.75	7.75	7.88	8.5
NN	E	21-Aug-03	700	1	4.15	4.29	-0.37	3.83	4.19	4.5
NN	W	17-Oct-03	800	1	6.99	7.28	-0.73	7.09	7.06	7.8
NN	E	17-Oct-03	800	1	1.56	1.71	-0.15	1.26	1.42	1.6
NN	W	31-Oct-03	900	1	6.71	7.15	-0.70	7.15	7.07	7.7
NN	E	31-Oct-03	900	1	1.86	1.95	-0.16	1.44	1.65	1.9
NN	W	11-Nov-03	1000	1	5.61	5.98	-0.56	6.09	6.08	6.5
NN	E	11-Nov-03	1000	1	1.54	1.57	-0.13	1.13	1.33	1.5
NN	W	20-Nov-03	1100	1	6.65	7.12	-0.67	7.23	6.98	7.7
NN	E	20-Nov-03	1100	1	2.10	2.15	-0.18	1.67	1.99	2.2
NN	W	9-Dec-03	1300	1	6.33	6.54	-0.62	6.61	6.84	7.2
NN	E	9-Dec-03	1300	1	2.12	2.38	-0.19	1.85	2.04	2.3
NN	W	28-Jan-04	1485	1	9.34		-0.42		9.58	9.9
NN	E	28-Jan-04	1485	1	2.73		-0.13		2.55	2.8
NN	W	6-Feb-04	1585	1	10.12		-0.48		10.63	10.9
NN	E	6-Feb-04	1585	1	3.13		-0.15		3.06	3.2
NN	W	13-Feb-04	1685	1	9.90		-0.68		10.58	10.9
NN	E	13-Feb-04	1685	1	3.47		-0.13		3.48	3.6
NN	W	24-Feb-04	1785	1	10.40		-0.70		11.15	11.5
NN	E	24-Feb-04	1785	1	3.06		-0.09		3.00	3.1
NN	W	5-Mar-04	1885	1	10.91		-0.74		11.77	12.1
NN	E	5-Mar-04	1885	1	3.53		-0.17		3.53	3.7
NN	W	16-Mar-04	2000	1	11.74		-0.80		12.70	13.0
NN	E	16-Mar-04	2000	1	3.74		-0.15		3.86	4.0

Table D1: Vertical Stress at the Top of the Soil Subgrade – Position 0" (continued)

Section	Sensor	Date	Passes (x 1000)	Signal Type	Values A	B	C	D	E	Stress (psi)
NS	W	25-Mar-03	0	1	2.25	2.34	-0.37	1.64	1.72	2.4
NS	E	25-Mar-03	0	1	12.52	12.77	-1.31	11.85	11.91	13.6
NS	W	30-Apr-03	100	1	2.19	2.09	-0.20	2.50	2.57	2.5
NS	E	30-Apr-03	100	1	8.86	9.33	-0.91	8.46	8.35	9.7
NS	W	28-May-03	200	1	3.38	3.26	-0.36	3.77	3.72	3.9
NS	E	28-May-03	200	1	8.74	9.22	-0.84	8.45	8.28	9.5
NS	W	5-Jun-03	300	1	3.87	3.69	-0.42	4.19	4.23	4.4
NS	E	5-Jun-03	300	1	8.83	9.40	-0.83	8.58	8.45	9.6
NS	W	21-Jul-03	400	1	5.17	5.16	-0.61	5.57	5.55	6.0
NS	E	21-Jul-03	400	1	10.17	10.46	-0.98	9.71	9.45	10.9
NS	W	1-Aug-03	500	1	5.29	5.32	-0.62	5.52	5.40	6.0
NS	E	1-Aug-03	500	1	10.36	10.79	-1.05	10.33	10.00	11.4
NS	W	21-Aug-03	700	1	6.11	6.02	-0.68	6.48	6.46	6.9
NS	E	21-Aug-03	700	1	11.83	12.24	-1.20	11.92	11.44	13.1
NS	W	17-Oct-03	800	1	5.20	5.12	-0.67	5.62	5.60	6.1
NS	E	17-Oct-03	800	1	11.54	11.81	-1.24	11.21	11.13	12.7
NS	W	31-Oct-03	900	1	5.20	5.18	-0.66	5.62	5.66	6.1
NS	E	31-Oct-03	900	1	11.87	12.09	-1.24	11.52	11.32	12.9
NS	W	11-Nov-03	1000	1	4.61	4.59	-0.57	5.02	5.13	5.4
NS	E	11-Nov-03	1000	1	12.01	12.16	-1.23	11.54	11.40	13.0
NS	W	20-Nov-03	1100	1	4.96	4.95	-0.57	5.31	5.47	5.7
NS	E	20-Nov-03	1100	1	11.91	11.96	-1.21	11.73	11.51	13.0
NS	W	9-Dec-03	1300	1	5.11	5.13	-0.60	5.55	5.69	6.0
NS	E	9-Dec-03	1300	1	11.21	11.72	-1.17	11.56	11.25	12.6
NS	W	28-Jan-04	1485	1	7.38		-0.38		8.03	8.1
NS	E	28-Jan-04	1485	1	15.12		-0.88		15.61	16.2
NS	W	6-Feb-04	1585	1	7.93		-0.41		8.53	8.6
NS	E	6-Feb-04	1585	1	15.51		-0.90		16.16	16.7
NS	W	13-Feb-04	1685	1	7.68		-0.58		8.44	8.6
NS	E	13-Feb-04	1685	1	15.59		-0.69		16.14	16.6
NS	W	24-Feb-04	1785	1	7.99		-0.58		8.76	9.0
NS	E	24-Feb-04	1785	1	16.21		-0.55		16.67	17.0
NS	W	5-Mar-04	1885	1	8.62		-0.62		9.49	9.7
NS	E	5-Mar-04	1885	1	16.40		-0.91		17.19	17.7
NS	W	16-Mar-04	2000	1	8.93		-0.64		9.83	10.0
NS	E	16-Mar-04	2000	1	17.23		-0.76		18.24	18.5

Table D1: Vertical Stress at the Top of the Soil Subgrade – Position 0” (continued)

Section	Sensor	Date	Passes (x 1000)	Signal Type	Values					Stress (psi)
					A	B	C	D	E	
SN	W	2-May-03	0	1	1.01	0.97	-0.09	1.50	1.86	1.4
SN	E	2-May-03	0	1	3.91	5.11	-0.35	3.11	3.45	4.2
SN	W	5-May-03	45	1	2.38	2.66	-0.19	2.78	3.08	2.9
SN	E	5-May-03	45	1	2.64	3.31	-0.16	1.55	2.06	2.6
SN	W	20-Jun-03	200	1	2.47	2.50	-0.19	2.96	3.20	3.0
SN	E	20-Jun-03	200	1	3.49	3.79	-0.21	1.95	2.78	3.2
SN	W	11-Jul-03	300	1	2.15	2.09	-0.16	2.67	2.85	2.6
SN	E	11-Jul-03	300	1	3.11	3.76	-0.20	1.96	2.55	3.0
SN	W	29-Aug-03	400	1	1.60	1.58	-0.12	2.12	2.12	2.0
SN	E	29-Aug-03	400	1	2.02	2.20	-0.14	1.37	1.57	1.9
SN	W	9-Sep-03	500	1	1.63	1.52	-0.13	2.14	2.27	2.0
SN	E	9-Sep-03	500	1	2.48	2.61	-0.17	1.75	1.89	2.4
SN	W	18-Sep-03	600	1	2.20	2.13	-0.20	2.76	2.93	2.7
SN	E	18-Sep-03	600	1	3.22	3.42	-0.23	2.21	2.67	3.1
SN	W	26-Sep-03	700	1	2.34	2.36	-0.22	2.79	2.83	2.8
SN	E	26-Sep-03	700	1	3.16	3.10	-0.24	2.21	2.61	3.0
SN	W	6-Oct-03	800	1	2.52	2.50	-0.23	2.93	2.95	3.0
SN	E	6-Oct-03	800	1	2.99	3.13	-0.23	2.12	2.56	2.9
SS	W	2-May-03	0	1	11.07	10.61	-1.46	13.10	13.38	13.5
SS	E	2-May-03	0	1	8.66	9.71	-0.95	8.27	8.58	9.8
SS	W	5-May-03	45	1	15.31	15.19	-1.31	16.23	16.10	17.0
SS	E	5-May-03	45	1	12.47	12.75	-1.04	11.39	11.16	13.0

Table D2. Vertical Stress at the Top of the Soil Subgrade – Position +6”

Section	Sensor	Date	Passes (x 1,000)	Signal Type	Value A	Value B	Value C	Value D	Value E	Stress (psi)
NN	**W**	25-Mar-03	0	1	3.38	3.73	-0.54	3.43	3.78	4.1
NN	**E**	25-Mar-03	0	1	5.84	6.02	-0.42	5.04	5.07	5.9
NN	**W**	30-Apr-03	100	1	2.59	2.85	-0.22	2.70	3.07	3.0
NN	**E**	30-Apr-03	100	1	4.22	4.55	-0.39	3.82	4.17	4.6
NN	**W**	28-May-03	200	1	5.34	5.57	-0.53	5.52	5.59	6.0
NN	**E**	28-May-03	200	1	4.46	5.03	-0.41	4.17	4.22	4.9
NN	**W**	5-Jun-03	300	1	4.33	4.60	-0.45	4.57	4.85	5.0
NN	**E**	5-Jun-03	300	1	3.24	3.97	-0.39	3.95	4.12	4.2
NN	**W**	21-Jul-03	400	1	6.03	6.24	-0.65	6.26	6.46	6.9
NN	**E**	21-Jul-03	400	1	5.66	6.19	-0.54	5.39	5.53	6.2
NN	**W**	1-Aug-03	500	1	5.62	5.88	-0.60	5.79	6.06	6.4
NN	**E**	1-Aug-03	500	1	4.07	4.59	-0.37	3.90	4.08	4.5
NN	**W**	21-Aug-03	700	1	6.23	6.38	-0.64	6.51	7.01	7.2
NN	**E**	21-Aug-03	700	1	5.29	5.73	-0.48	5.03	5.30	5.8
NN	**W**	17-Oct-03	800	1	4.89	5.20	-0.53	5.35	5.34	5.7
NN	**E**	17-Oct-03	800	1	1.38	1.45	-0.12	1.12	1.32	1.4
NN	**W**	31-Oct-03	900	1	5.40	5.58	-0.55	5.57	5.81	6.1
NN	**E**	31-Oct-03	900	1	1.77	2.15	-0.16	1.60	1.58	1.9
NN	**W**	11-Nov-03	1000	1	4.10	4.33	-0.41	4.47	4.61	4.8
NN	**E**	11-Nov-03	1000	1	1.41	1.42	-0.12	1.02	1.27	1.4
NN	**W**	20-Nov-03	1100	1	4.86	5.22	-0.49	5.50	5.46	5.8
NN	**E**	20-Nov-03	1100	1	2.27	2.38	-0.19	1.91	2.23	2.4
NN	**W**	9-Dec-03	1300	1	4.76	4.97	-0.47	5.19	5.32	5.5
NN	**E**	9-Dec-03	1300	1	1.82	1.75	-0.14	1.36	1.65	1.8
NN	**W**	28-Jan-04	1485	1	8.10		-0.36		8.40	8.6
NN	**E**	28-Jan-04	1485	1	2.45		-0.11		2.10	2.4
NN	**W**	6-Feb-04	1585	1	9.10		-0.42		9.54	9.7
NN	**E**	6-Feb-04	1585	1	2.91		-0.13		2.51	2.8
NN	**W**	13-Feb-04	1685	1	9.18		-0.64		9.78	10.1
NN	**E**	13-Feb-04	1685	1	3.27		-0.10		2.92	3.2
NN	**W**	24-Feb-04	1785	1	8.62		-0.56		9.22	9.5
NN	**E**	24-Feb-04	1785	1	3.11		-0.08		2.64	3.0
NN	**W**	5-Mar-04	1885	1	10.00		-0.67		10.76	11.1
NN	**E**	5-Mar-04	1885	1	3.24		-0.13		2.83	3.2
NN	**W**	16-Mar-04	2000	1	10.22		-0.68		10.80	11.2
NN	**E**	16-Mar-04	2000	1	3.57		-0.10		3.31	3.5

Table D2. Vertical Stress at the Top of the Soil Subgrade – Position +6" (continued)

Section	Sensor	Date	Passes (x 1,000)	Signal Type	Value					Stress (psi)
					A	B	C	D	E	
NS	**W**	25-Mar-03	0	1	2.25	2.34	-0.37	1.64	1.72	2.4
NS	**E**	25-Mar-03	0	1	12.52	12.77	-1.31	11.85	11.91	13.6
NS	**W**	30-Apr-03	100	1	1.51	1.53	-0.14	1.88	2.07	1.9
NS	**E**	30-Apr-03	100	1	7.10	7.32	-0.71	6.72	7.13	7.8
NS	**W**	28-May-03	200	1	3.54	3.53	-0.38	3.97	3.88	4.1
NS	**E**	28-May-03	200	1	8.23	8.53	-0.78	7.76	7.84	8.9
NS	**W**	5-Jun-03	300	1	3.19	3.32	-0.38	3.85	3.88	3.9
NS	**E**	5-Jun-03	300	1	7.38	7.71	-0.69	7.17	7.23	8.1
NS	**W**	21-Jul-03	400	1	5.24	5.45	-0.65	5.85	5.69	6.2
NS	**E**	21-Jul-03	400	1	9.36	9.70	-0.94	9.14	8.93	10.2
NS	**W**	1-Aug-03	500	1	5.03	5.26	-0.63	5.60	5.47	6.0
NS	**E**	1-Aug-03	500	1	9.45	9.52	-0.96	9.19	9.34	10.3
NS	**W**	21-Aug-03	700	1	5.87	6.00	-0.70	6.48	6.60	6.9
NS	**E**	21-Aug-03	700	1	10.60	10.79	-1.09	10.63	10.56	11.7
NS	**W**	17-Oct-03	800	1	4.06	4.11	-0.54	4.89	4.91	5.0
NS	**E**	17-Oct-03	800	1	9.26	9.40	-1.00	9.04	9.15	10.2
NS	**W**	31-Oct-03	900	1	4.73	4.74	-0.60	5.21	5.28	5.6
NS	**E**	31-Oct-03	900	1	10.20	10.24	-1.08	9.95	10.03	11.2
NS	**W**	11-Nov-03	1000	1	4.16	4.23	-0.53	4.64	4.74	5.0
NS	**E**	11-Nov-03	1000	1	9.89	10.03	-1.05	9.59	9.65	10.8
NS	**W**	20-Nov-03	1100	1	3.94	4.12	-0.48	4.65	4.66	4.8
NS	**E**	20-Nov-03	1100	1	9.79	9.91	-1.02	9.69	9.77	10.8
NS	**W**	9-Dec-03	1300	1	3.99	4.04	-0.46	4.57	4.72	4.8
NS	**E**	9-Dec-03	1300	1	9.75	9.60	-0.98	9.67	9.56	10.6
NS	**W**	28-Jan-04	1485	1	6.19		-0.31		6.64	6.7
NS	**E**	28-Jan-04	1485	1	13.56		-0.77		14.09	14.6
NS	**W**	6-Feb-04	1585	1	6.74		-0.34		7.26	7.3
NS	**E**	6-Feb-04	1585	1	14.44		-0.86		14.90	15.5
NS	**W**	13-Feb-04	1685	1	6.78		-0.50		7.42	7.6
NS	**E**	13-Feb-04	1685	1	15.07		-0.59		15.38	15.8
NS	**W**	24-Feb-04	1785	1	6.58		-0.47		7.22	7.4
NS	**E**	24-Feb-04	1785	1	14.37		-0.49		14.75	15.1
NS	**W**	5-Mar-04	1885	1	7.57		-0.54		8.23	8.4
NS	**E**	5-Mar-04	1885	1	15.46		-0.73		15.88	16.4
NS	**W**	16-Mar-04	2000	1	7.62		-0.54		8.21	8.5
NS	**E**	16-Mar-04	2000	1	15.88		-0.54		16.56	16.8

Table D2. Vertical Stress at the Top of the Soil Subgrade – Position +6" (continued)

Section	Sensor	Date	Passes (x 1,000)	Signal Type	Value					Stress (psi)
					A	B	C	D	E	
SN	**W**	2-May-03	0	1	0.46	0.53	-0.05	0.88	1.10	0.8
SN	**E**	2-May-03	0	1	4.74	5.89	-0.41	4.06	4.57	5.2
SN	**W**	5-May-03	45	1	2.97	3.29	-0.23	3.61	3.83	3.7
SN	**E**	5-May-03	45	1	3.69	4.70	-0.23	2.59	3.16	3.8
SN	**W**	20-Jun-03	200	1	2.23	2.42	-0.18	2.98	3.10	2.9
SN	**E**	20-Jun-03	200	1	3.66	4.78	-0.23	2.62	3.48	3.9
SN	**W**	11-Jul-03	300	1	2.04	2.21	-0.16	2.82	2.89	2.7
SN	**E**	11-Jul-03	300	1	3.63	4.57	-0.24	2.66	3.32	3.8
SN	**W**	29-Aug-03	400	1	1.87	2.03	-0.14	2.60	2.58	2.4
SN	**E**	29-Aug-03	400	1	2.47	2.97	-0.16	1.66	2.07	2.5
SN	**W**	9-Sep-03	500	1	1.66	1.74	-0.13	2.36	2.51	2.2
SN	**E**	9-Sep-03	500	1	2.31	2.50	-0.17	1.64	1.91	2.3
SN	**W**	18-Sep-03	600	1	2.25	2.32	-0.20	3.01	3.13	2.9
SN	**E**	18-Sep-03	600	1	3.46	7.78	-0.24	2.99	3.22	4.6
SN	**W**	26-Sep-03	700	1	2.45	2.51	-0.28	2.91	3.01	3.0
SN	**E**	26-Sep-03	700	1	3.33	3.48	-0.25	2.83	3.03	3.4
SN	**W**	6-Oct-03	800	1	2.37	2.41	-0.26	2.77	2.88	2.9
SN	**E**	6-Oct-03	800	1	2.64	2.81	-0.21	2.26	2.51	2.8
SS	**W**	2-May-03	0	1	7.91	7.58	-1.12	9.56	9.94	9.9
SS	**E**	2-May-03	0	1	6.11	6.90	-0.64	5.69	6.05	6.8
SS	**W**	5-May-03	45	1	14.78	13.40	-1.35	14.83	15.44	16.0
SS	**E**	5-May-03	45	1	12.39	12.52	-1.08	11.25	11.40	13.0

APPENDIX E - FALLING WEIGHT DEFLECTOMETER DATA

Table E1: FWD deflection data and corresponding backcalculated moduli

Lane	Date	Passes (x1000)	Station	Drop Nr.	Load (lbs)	D0 (mils)	D1 (mils)	D2 (mils)	D3 (mils)	E(AC) (psi)	E(base) (psi)	Mr (psi)
NN	1/24/2003	0	1	1	6,130	13.06	10.02	7.96	5.73	111,022	836,670	7,046
NN	1/24/2003	0	1	2	8,990	20.72	15.73	12.49	8.94	94,501	795,108	6,621
NN	1/24/2003	0	1	3	9,009	20.76	15.78	12.54	8.98	95,160	794,609	6,606
NN	1/24/2003	0	2	1	6,101	13.37	10	7.97	5.5	98,648	745,177	7,298
NN	1/24/2003	0	2	2	9,077	20.91	15.71	12.54	8.72	92,240	748,008	6,830
NN	1/24/2003	0	2	3	9,104	20.98	15.8	12.62	8.78	93,965	744,552	6,799
NN	1/24/2003	0	3	1	6,061	14.2	11	8.61	5.9	107,166	599,265	6,796
NN	1/24/2003	0	3	2	8,961	21.68	16.84	13.26	9.14	104,586	602,047	6,475
NN	1/24/2003	0	3	3	8,942	21.58	16.78	13.22	9.1	107,466	595,617	6,487
NN	1/24/2003	0	4	1	6,077	13.59	10.15	7.87	5.44	74,313	717,754	7,450
NN	1/24/2003	0	4	2	8,569	20.37	15.17	11.82	8.13	68,579	683,632	7,011
NN	1/24/2003	0	4	3	8,500	20.27	15.11	11.79	8.12	68,402	688,360	6,957
NN	1/24/2003	0	6	1	6,109	15.04	10.8	8.14	5.52	82,286	453,965	7,403
NN	1/24/2003	0	6	2	8,373	21.07	15.28	11.62	7.95	82,130	468,827	7,027
NN	1/24/2003	0	6	3	8,378	21.02	15.29	11.63	7.96	82,912	472,393	7,011
NN	1/24/2003	0	5	1	6,125	14.76	10.23	7.89	5.48	61,560	679,718	7,521
NN	1/24/2003	0	5	2	8,418	20.64	14.65	11.42	7.91	66,634	657,141	7,084
NN	1/24/2003	0	5	3	8,410	20.61	14.63	11.43	7.92	66,565	665,657	7,059
NS	1/24/2003	0	1	1	6,109	14.97	11.02	8.63	5.93	140,000	379,071	6,946
NS	1/24/2003	0	1	2	8,998	22.89	16.9	13.31	9.16	140,000	367,828	6,610
NS	1/24/2003	0	1	3	9,021	22.99	16.96	13.36	9.2	140,000	366,358	6,603
NS	1/24/2003	0	2	1	6,082	14.98	11.04	8.67	6.02	140,000	390,741	6,827
NS	1/24/2003	0	2	2	8,998	22.96	17	13.39	9.3	140,000	377,160	6,522
NS	1/24/2003	0	2	3	8,993	22.99	17.02	13.41	9.31	140,000	375,970	6,510
NS	1/24/2003	0	3	1	5,962	16.29	12.13	9.33	6.3	350,514	194,065	6,095
NS	1/24/2003	0	3	2	8,866	24.69	18.34	14.24	9.71	206,663	291,489	5,882
NS	1/24/2003	0	3	3	8,823	24.56	18.25	14.16	9.65	204,655	293,738	5,888
NS	1/24/2003	0	4	1	6,201	16.97	13.04	10.06	6.59	1,194,497	84,728	6,012
NS	1/24/2003	0	4	2	9,056	25.82	20.06	15.61	10.3	1,229,774	82,787	5,599
NS	1/24/2003	0	4	3	9,069	25.9	20.15	15.7	10.37	1,118,322	90,384	5,548
NS	1/24/2003	0	6	1	6,037	18.19	13.2	9.75	6.38	195,522	189,243	6,094
NS	1/24/2003	0	6	2	8,902	27.61	20.25	15.09	9.83	204,148	183,091	5,788
NS	1/24/2003	0	6	3	8,910	27.58	20.28	15.12	9.85	200,286	187,077	5,780
NS	1/24/2003	0	5	1	6,066	15.62	11.55	8.67	5.99	311,044	217,529	6,594
NS	1/24/2003	0	5	2	8,993	24.83	18.22	13.78	9.48	242,664	227,939	6,179
NS	1/24/2003	0	5	3	8,966	24.83	18.25	13.79	9.46	288,695	200,762	6,167
SN	1/24/2003	0	1	1	6,148	16.45	9.62	6.79	4.48	39,748	703,035	9,055
SN	1/24/2003	0	1	2	8,945	25.97	16	11.34	7.2	46,196	523,969	8,039
SN	1/24/2003	0	1	3	8,961	26	16.05	11.4	7.22	46,695	521,656	8,022
SN	1/24/2003	0	2	1	6,141	15.74	9.74	6.95	4.64	47,799	723,307	8,636
SN	1/24/2003	0	2	2	8,982	24.36	15.78	11.39	7.34	57,115	591,297	7,937
SN	1/24/2003	0	2	3	8,985	24.39	15.88	11.43	7.33	59,556	568,943	7,969
SN	1/24/2003	0	3	1	6,114	13.77	9.63	7.06	4.73	90,995	917,460	8,755
SN	1/24/2003	0	3	2	9,033	22.93	15.99	11.66	7.51	88,645	692,888	8,063
SN	1/24/2003	0	3	3	8,982	22.85	15.96	11.65	7.52	89,489	692,888	8,007
SN	1/24/2003	0	4	1	6,121	14.91	10.93	7.8	4.9	183,640	455,971	8,239
SN	1/24/2003	0	4	2	9,093	22.98	16.79	12.06	7.67	153,509	496,135	7,853
SN	1/24/2003	0	4	3	9,101	23.07	16.82	12.1	7.72	149,624	503,195	7,825
SN	1/24/2003	0	6	1	6,233	14.76	8.72	6.41	4.47	35,097	824,983	9,747
SN	1/24/2003	0	6	2	9,149	22.66	13.98	10.32	7.06	39,631	718,980	9,037

Table E1: FWD deflection data and corresponding backcalculated moduli - continued

Lane	Date	Passes (x1000)	Station	Drop Nr.	Load (lbs)	D0 (mils)	D1 (mils)	D2 (mils)	D3 (mils)	E(AC) (psi)	E(base) (psi)	Mr (psi)
SN	1/24/2003	0	6	3	9,109	22.55	13.98	10.3	7.05	40,622	709,849	9,012
SN	1/24/2003	0	5	1	6,172	13.3	9.1	6.7	4.57	74,130	690,065	9,333
SN	1/24/2003	0	5	2	9,152	20.93	14.61	10.92	7.35	75,968	655,847	8,474
SN	1/24/2003	0	5	3	9,156	21.02	14.75	10.94	7.34	83,001	607,133	8,507
SS	1/24/2003	0	1	1	6,153	21.06	14.65	10.75	6.56	40,943	577,229	6,202
SS	1/24/2003	0	1	2	9,152	34.15	24	17.73	10.78	39,387	530,603	5,592
SS	1/24/2003	0	1	3	9,088	33.87	23.82	17.63	10.73	39,829	532,536	5,580
SS	1/24/2003	0	2	1	6,125	21.15	14.75	10.73	6.65	40,404	579,788	6,127
SS	1/24/2003	0	2	2	9,085	33.3	23.69	17.4	10.75	42,186	542,241	5,592
SS	1/24/2003	0	2	3	9,080	33.31	23.63	17.39	10.76	41,211	553,342	5,586
SS	1/24/2003	0	3	1	6,014	23.23	15.71	11.23	6.57	36,444	688,897	5,488
SS	1/24/2003	0	3	2	9,056	38.08	25.85	18.44	10.69	34,595	610,468	5,049
SS	1/24/2003	0	3	3	9,029	37.9	25.76	18.38	10.65	34,614	612,139	5,053
SS	1/24/2003	0	4	1	6,085	21.89	15.24	11	6.65	42,319	800,743	5,533
SS	1/24/2003	0	4	2	9,033	35.24	24.48	17.71	10.66	38,374	742,677	5,116
SS	1/24/2003	0	4	3	9,045	35.25	24.52	17.74	10.67	39,116	733,817	5,116
NN	11/17/2003	1,050	1	1	5,791	15.99	12.35	9.48	6.2	169,642	256,205	5,988
NN	11/17/2003	1,050	1	2	9,435	24.28	19.19	15.2	10.32	175,337	369,262	5,844
NN	11/17/2003	1,050	1	3	9,419	24.19	19.15	15.16	10.29	574,315	174,669	5,786
NN	11/17/2003	1,050	3	1	5,799	16.19	12.44	9.55	6.35	107,327	377,043	6,032
NN	11/17/2003	1,050	3	2	9,427	24.07	19.17	15.07	10.45	129,902	510,629	5,897
NN	11/17/2003	1,050	3	3	9,462	24.02	19.15	15.06	10.45	129,902	517,335	5,918
NN	11/17/2003	1,050	4	1	5,843	15.87	11.59	9.15	6.37	49,946	698,380	6,096
NN	11/17/2003	1,050	4	2	9,506	23.69	18.09	14.58	10.5	63,822	873,976	5,870
NN	11/17/2003	1,050	4	3	9,525	23.69	18.11	14.61	10.52	62,783	892,083	5,857
NN	11/17/2003	1,050	6	1	5,899	15.8	11.1	8.66	6.09	55,983	671,960	6,465
NN	11/17/2003	1,050	6	2	9,501	23.53	17.4	13.96	10.05	72,965	788,257	6,148
NN	11/17/2003	1,050	6	3	9,490	23.44	17.35	13.92	10.01	72,945	793,753	6,158
NS	11/17/2003	1,050	1	1	5,780	21.65	14.55	11.14	6.85	70,000	200,000	5,423
NS	11/17/2003	1,050	1	2	9,268	31.44	22.77	17.62	11.06	84,916	229,999	5,432
NS	11/17/2003	1,050	1	3	9,220	31.31	22.65	17.55	11	84,601	229,999	5,432
NS	11/17/2003	1,050	3	1	5,875	22.28	16.34	11.17	6.9	884,615	27,250	5,528
NS	11/17/2003	1,050	3	2	9,064	33.18	24.87	17.7	11.09	1,163,814	25,041	5,356
NS	11/17/2003	1,050	3	3	9,053	33.12	24.84	17.69	11.07	1,171,188	24,758	5,356
NS	11/17/2003	1,050	4	1	5,894	22.01	15.28	10.09	6.44	440,485	51,044	5,961
NS	11/17/2003	1,050	4	2	8,982	32.59	23.6	16.32	10.44	676,220	47,115	5,608
NS	11/17/2003	1,050	4	3	8,958	32.57	23.7	16.41	10.49	694,670	46,463	5,560
NS	11/17/2003	1,050	6	1	5,886	21.36	15.59	11.2	6.33	122,519	128,410	4,723
NS	11/17/2003	1,050	6	2	9,422	31.68	24.52	18.11	10.41	155,900	155,408	4,493
NS	11/17/2003	1,050	6	3	9,414	31.62	24.52	18.1	10.4	159,476	152,899	4,494
SN	11/17/2003	800	1	1	5,978	17.8	11.74	8.22	4.95	36,616	424,251	7,903
SN	11/17/2003	800	1	2	9,287	27.09	18.61	13.49	8.44	193,760	208,097	7,407
SN	11/17/2003	800	1	3	9,271	27.03	18.59	13.49	8.44	196,486	207,587	7,394
SN	11/17/2003	800	3	1	5,862	16.56	11.56	8.5	5.32	43,705	642,540	7,400
SN	11/17/2003	800	3	2	9,390	24.88	17.81	13.38	8.69	47,544	814,622	7,278
SN	11/17/2003	800	3	3	9,366	24.75	17.63	13.26	8.61	44,813	847,671	7,307
SN	11/17/2003	800	4	1	5,950	16.49	11.41	8.12	4.86	51,398	497,287	8,108
SN	11/17/2003	800	4	2	9,482	25.37	18.25	13.37	8.32	57,953	616,820	7,612
SN	11/17/2003	800	4	3	9,466	25.35	18.21	13.33	8.27	59,749	595,634	7,651
SN	11/17/2003	800	6	1	6,037	18.51	11.15	7.48	4.76	18,716	375,566	8,610
SN	11/17/2003	800	6	2	9,506	27.3	17.59	12.4	8.18	22,061	489,371	7,882
SN	11/17/2003	800	6	3	9,533	27.33	17.64	12.46	8.18	22,356	485,369	7,887

APPENDIX F - WEIGHT DROP DATA

Table F1: Weight Drop Device - Deflection data – Lane NN

Passes (x 1,000)	Point	Load (lbs)	D0	D6	D12	D21	D30	D39	D48	D57	D66	K0
0	**W**	2600	7.000	3.525	1.225	0.525	0.325	0.150	0.138	0.100	0.200	371.4
0	**E**	2708	5.600	3.450	1.400	0.538	0.513	0.575	0.575	0.488	0.338	483.6
100	**W**	2425	9.700	6.713	3.450	1.100	0.388	0.238	0.200	0.313	0.275	250.0
100	**E**	2567	9.450	7.475	4.300	1.575	0.425	0.138	0.288	0.400	0.413	271.6
200	**W**	1175	3.750	1.975	0.713	0.063	0.025	0.025	0.288	0.550	0.113	313.3
200	**E**	2650	8.450	5.963	3.200	1.325	1.200	1.088	0.813	0.538	0.350	313.6
300	**W**	2725	9.538	6.600	3.200	1.088	0.425	0.388	0.463	0.450	0.275	285.7
300	**E**	2492	9.038	7.100	3.900	1.388	0.500	0.413	0.463	0.513	0.388	275.7
400	**W**	2783	10.88	7.413	3.525	1.213	0.512	0.400	0.288	0.113	0.300	255.6
400	**E**	2583	10.21	7.563	4.038	1.563	0.538	0.188	0.200	0.225	0.338	253.0
500	**W**	2700	10.52	6.800	3.150	0.713	0.238	0.163	0.363	0.500	0.450	256.5
500	**E**	2742	8.375	5.450	3.350	1.000	0.400	0.200	0.463	0.675	0.625	327.4
700	**W**	2783	10.32	7.713	3.563	1.075	0.800	0.650	0.488	0.525	0.375	269.6
700	**E**	2708	10.47	7.738	4.000	1.538	0.625	0.425	0.400	0.338	0.313	258.6
800	**W**	2758	9.513	6.613	3.375	1.088	0.325	0.138	0.375	0.463	0.338	290.0
800	**E**	2883	9.100	7.513	4.400	1.138	0.513	0.413	0.388	0.375	0.400	316.8
900	**W**	2825	9.588	6.525	3.525	1.363	0.563	0.300	0.363	0.413	0.313	294.7
900	**E**	2683	8.663	7.100	3.175	1.013	0.588	0.600	0.725	0.800	0.612	309.8
1,000	**W**	2775	8.363	5.988	3.388	1.525	0.713	0.600	0.650	0.513	0.400	331.8
1,000	**E**	2733	8.200	6.375	3.213	0.863	0.675	0.863	0.875	0.763	0.650	333.3
1,100	**W**	2750	8.675	6.150	2.825	1.263	0.638	0.600	0.575	0.463	0.250	317.0
1,100	**E**	3050	9.438	6.900	3.600	1.463	0.737	0.488	0.463	0.413	0.250	323.2
1,300	**W**	2767	9.613	6.625	3.538	1.525	0.650	0.438	0.450	0.263	0.325	287.8
1,300	**E**	2775	9.375	7.213	4.225	1.700	1.000	0.775	0.688	0.550	0.363	296.0
1,485	**W**	2783	10.88	7.413	3.525	1.213	0.512	0.400	0.288	0.113	0.300	255.6
1,485	**E**	2783	9.413	7.225	3.475	1.400	0.775	0.713	0.450	0.375	0.413	295.7
1,585	**W**	2842	9.763	6.088	3.050	1.025	0.387	0.250	0.275	0.400	0.400	291.1
1,585	**E**	2633	10.08	7.600	4.450	1.875	1.200	1.150	1.088	0.787	0.125	261.0
1,685	**W**	2892	9.363	6.338	2.975	0.963	0.488	0.550	0.700	0.700	0.563	308.9
1,685	**E**	2950	12.50	8.638	4.838	2.063	1.088	0.875	0.800	0.662	0.575	236.0
1,785	**W**	2783	9.163	6.450	3.125	1.000	0.413	0.413	0.563	0.575	0.488	303.8
1,785	**E**	2650	9.813	7.175	4.238	1.125	0.600	0.613	0.600	0.638	0.425	270.1
1,885	**W**	2775	9.863	6.138	2.650	0.938	0.363	0.288	0.500	0.475	0.400	281.4
1,885	**E**	2758	11.16	7.863	3.788	0.625	0.400	0.425	0.463	0.450	0.350	247.1
2,000	**W**	2767	10.40	7.625	3.150	1.075	0.350	0.263	0.413	0.450	0.275	266.0
2,000	**E**	2700	11.46	8.800	3.788	1.363	0.538	0.662	0.800	0.725	0.425	235.6

<u>Note:</u> D30 – deflection (in mils) at an offset of 30 inches

Table F2: Weight Drop Device - Deflection data – Lane NS

Passes (x 1,000)	Point	Load (lbs)	D0	D6	D12	D21	D30	D39	D48	D57	D66	K0
0	**W**	2375	7.825	5.775	3.200	1.300	0.650	0.375	0.488	0.475	0.413	303.5
0	**E**	2425	6.675	4.763	2.838	1.325	0.800	0.375	0.313	0.100	0.288	363.3
100	**W**	2600	12.56	9.300	4.738	1.475	0.425	0.088	0.013	0.125	0.175	207.0
100	**E**	2592	9.250	5.813	2.313	0.163	0.325	0.375	0.625	0.537	0.088	280.2
200	**W**	2558	11.16	8.975	4.075	1.263	0.513	0.325	0.213	0.188	0.263	229.2
200	**E**	2617	10.28	7.050	3.788	1.363	0.600	0.488	0.625	0.550	0.450	254.4
300	**W**	2575	13.31	9.600	4.413	1.300	0.600	0.575	0.713	0.713	0.525	193.4
300	**E**	2600	11.20	7.138	3.338	1.200	0.575	0.450	0.600	0.600	0.500	232.1
400	**W**	2692	12.31	8.725	4.425	1.288	0.613	0.750	0.725	0.575	0.438	218.6
400	**E**	2767	10.67	6.550	3.125	0.900	0.100	0.163	0.388	0.600	0.313	259.2
500	**W**	2567	11.41	7.950	4.025	1.038	0.725	0.575	0.712	0.525	0.388	224.9
500	**E**	2517	12.76	8.763	4.238	1.575	0.712	0.438	0.625	0.625	0.575	197.2
700	**W**	2717	12.77	9.575	4.850	1.238	0.625	0.438	0.475	0.388	0.375	212.7
700	**E**	2650	11.90	8.088	3.875	0.963	0.225	0.025	0.063	0.363	0.350	222.7
800	**W**	2633	12.06	10.06	5.325	1.363	0.525	0.163	0.438	0.525	0.450	218.3
800	**E**	2792	10.12	6.700	3.450	0.950	0.500	0.388	0.488	0.663	0.575	275.7
900	**W**	2792	12.02	8.763	3.388	0.688	0.500	0.575	0.788	0.763	0.563	232.2
900	**E**	2758	11.32	6.825	3.713	1.275	0.588	0.575	0.625	0.638	0.625	243.6
1,000	**W**	2842	12.01	8.875	4.313	1.388	0.950	0.988	0.825	0.463	0.550	236.6
1,000	**E**	2833	10.48	6.963	3.388	1.325	0.625	0.513	0.763	0.713	0.675	270.2
1,100	**W**	2667	13.21	10.41	4.938	1.563	0.825	0.825	0.788	0.712	0.313	201.8
1,100	**E**	2850	11.70	7.388	3.763	1.400	0.863	0.613	0.563	0.487	0.550	243.6
1,300	**W**	2808	12.51	9.363	4.000	0.900	0.538	0.413	0.400	0.425	0.225	224.4
1,300	**E**	2808	11.22	6.800	2.950	0.663	0.325	0.125	0.150	0.138	0.063	250.2
1,485	**W**	2408	12.45	9.150	4.088	1.150	0.588	0.388	0.500	0.488	0.413	193.4
1,485	**E**	2825	11.22	7.813	3.863	1.538	0.788	0.500	0.550	0.450	0.438	251.7
1,585	**W**	2767	12.50	9.288	4.000	1.400	1.050	0.925	0.913	0.762	0.625	221.3
1,585	**E**	2758	11.85	7.850	4.188	1.875	1.100	0.663	0.563	0.400	0.350	232.8
1,685	**W**	2892	14.61	11.42	5.300	1.475	1.013	0.800	0.750	0.600	0.563	197.9
1,685	**E**	2783	12.97	8.750	4.350	1.900	1.163	0.875	0.875	0.800	0.800	214.5
1,785	**W**	2800	12.90	9.350	4.038	1.038	0.600	0.338	0.350	0.313	0.375	217.1
1,785	**E**	2792	11.81	7.813	3.438	1.150	0.537	0.238	0.413	0.475	0.388	236.3
1,885	**W**	2750	13.91	10.57	4.700	1.225	0.788	0.625	0.675	0.700	0.600	197.7
1,885	**E**	2858	12.36	7.363	2.925	0.913	0.400	0.363	0.450	0.550	0.325	231.2
2,000	**W**	2833	14.33	11.27	4.700	1.725	1.288	1.100	0.938	0.725	0.688	197.6
2,000	**E**	2583	13.33	7.563	3.838	1.238	0.538	0.288	0.350	0.425	0.088	193.7

Note: D30 – deflection (in mils) at an offset of 30 inches

Table F3: Weight Drop Device - Deflection data – Lanes SN & SS

Passes (x 1,000)	Point	Load (lbs)	D0	D6	D12	D21	D30	D39	D48	D57	D66	K0
Lane SN												
0	**W**	1883	4.638	2.613	1.288	0.538	0.313	0.275	0.238	0.238	0.138	406.1
0	**E**	1933	4.763	2.475	0.975	0.375	0.288	0.188	0.200	0.288	0.225	405.9
200	**W**	2692	15.92	10.07	4.438	1.550	0.875	0.663	0.663	0.525	0.338	169.0
200	**E**	2792	15.93	9.413	3.538	1.088	0.650	0.600	0.538	0.638	0.388	175.2
300	**W**	2775	15.62	9.063	3.675	1.088	0.788	0.525	0.500	0.638	0.400	177.6
300	**E**	2642	16.11	9.638	3.925	0.988	0.838	0.513	0.338	0.463	0.263	164.0
400	**W**	2592	14.01	7.925	3.138	0.700	0.513	0.500	0.675	0.863	0.600	185.0
400	**E**	2725	16.92	10.40	4.113	1.100	0.950	1.075	0.813	0.550	0.275	161.0
500	**W**	2892	13.35	9.475	4.325	1.588	0.663	0.425	0.550	0.637	0.225	216.6
500	**E**	2692	15.20	9.375	3.813	1.113	0.913	0.750	0.450	0.488	0.200	177.1
700	**W**	2783	14.71	7.275	2.988	1.238	0.825	0.725	0.950	0.775	0.600	189.2
700	**E**	2725	15.53	9.963	4.800	1.613	1.038	0.600	0.538	0.450	0.350	175.4
800	**W**	2767	15.06	9.100	3.763	1.400	0.788	0.463	0.363	0.325	0.313	183.7
800	**E**	2775	16.95	11.45	5.025	1.338	0.913	0.638	0.388	0.438	0.288	163.7
Lane SS												
0	**W**	2825	14.16	6.625	2.400	0.825	0.750	0.775	0.788	0.763	0.463	199.5
0	**E**	2908	16.35	10.75	4.425	1.350	0.938	0.713	0.762	0.625	0.500	177.9
45	**W**	1858	4.950	3.475	1.700	0.538	0.375	0.363	0.463	0.425	0.363	375.4
45	**E**	1858	4.838	3.163	1.338	0.463	0.450	0.400	0.288	0.288	0.288	384.2

Note: *D30 – deflection (in mils) at an offset of 30 inches*

DEPARTMENT OF TRANSPORTATION
AD ASTRA PER ASPERA
KANSAS